SUDAS

Design Manual

Iowa
SUDAS

Statewide Urban Design and Specifications Program

NOTICE TO USERS OF
SUDAS DESIGN MANUAL

Before using the design standards in this manual, the Project Engineer must check with the Jurisdictional Engineer of the Jurisdiction in which the project will be designed and/or constructed for any special provisions that modify the standards contained herein. The Jurisdiction will review all submittals for compliance with the specific local design criteria, procedures, and regulations. The Jurisdiction shall have no obligation to verify the certified engineering calculations, method of design, and as-built drawings required to be submitted. Acceptance of plans and issuance or approval of any permit should not be interpreted as guaranteeing the performance of the engineering documents or alleviating the Project Engineer of being responsible for the accuracy and adequacy of the plan. Approval by the Jurisdiction does not relieve the Project Engineer from the responsibility of ensuring that the calculations, design, and plans are accurate and are in compliance with this manual as may be modified by the local Jurisdiction standards and fit the needs of a particular project based on sound engineering principles.

Foreword

In the late 1980s, sixteen central Iowa public agencies, including the City of Des Moines, surrounding cities, and two counties, began meeting to discuss developing common urban design standards and construction specifications.

Developing common standards among several jurisdictions was breaking new ground in Iowa, and the group made slow but deliberate progress.

Their efforts came into focus when, in 1995, Governor Terry Branstad assembled the “Blue Ribbon Task Force on Transportation” to investigate ways to use Iowa’s Road Use Tax Fund more efficiently. One of the task force’s recommendations was that agencies “adopt common standards for construction specifications . . . .” By 1998, the central Iowa group (then known as the Central Iowa Committee) had expanded to 34 Iowa jurisdictions, including several communities outside the Des Moines area, and had published their design guidelines and standard specifications.

In 2000, the effort was underway to further expand the number of cities using the Central Iowa Committee’s manuals and to convert them to statewide manuals, eventually known as the Statewide Urban Design and Specifications (SUDAS) program.

A statewide steering committee, comprised of various stakeholder groups, including Iowa’s cities and counties, the Iowa DOT, engineering consultants, and industry representatives, was organized in 2002 to oversee the new SUDAS program. Iowa State University’s Center for Transportation Research and Education (CTRE) was chosen to manage the program.

In 2004, a new nonprofit entity was created to establish a mechanism for statewide ownership: the Iowa SUDAS Corporation. The Board of Directors for the corporation consisted of members who formerly served on the statewide steering committee, with the addition of a few others.

On February 17, 2005, the Central Iowa Committee acted to officially transfer ownership of the manuals to the Iowa SUDAS Corporation. Statewide ownership of the manuals makes them truly the statewide standards for urban public works improvements. The program is funded through the Iowa DOT and the state transportation planning agencies.

The SUDAS Standard Specifications were revised and reissued with the 2009 Edition. This version represented the most extensive revisions since the original manual was published in 1998. Since it had been six years since the last full printing of the SUDAS Standard Specifications, another full printing with the 2015 Edition was issued so users could be assured they had a fully updated manual. With the 2015 Edition, demolition was moved to Division 10 and a new Division 11 (Miscellaneous) was developed.

The SUDAS Design Manual was reissued with the 2013 Edition, which included rewriting and revising 13 of the 14 chapters. This extensive work was accomplished through the SUDAS technical and district committees, the SUDAS Board of Directors, and engineering consultants. This task was completed within a 2 year period, and represented the most extensive revisions since the 2001 Edition.

Iowa State University’s Institute for Transportation (InTrans, formerly CTRE) continues to manage the SUDAS program.

Contributors and Acknowledgments

In 2016, SUDAS staff held many meetings to accomplish the various revisions reflected in the 2017 versions of the SUDAS manuals. These revisions would not have been possible without the efforts of the SUDAS technical committee members. The SUDAS program's success is also due to the dedication of the district committees and Board of Directors. Keeping the SUDAS manuals current is an ongoing, cooperative effort, involving hundreds of people who volunteer their time and expertise. It is not possible to acknowledge each of these volunteers individually, but we appreciate them all.

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General Provisions

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General Conditions

A. Purpose

The SUDAS Design Manual has been prepared as a mechanism to implement uniform design standards, procedures, and regulations for the preparation of urban public improvement construction plans. Public improvements are those that meet any of the following:

1. Are initiated, designed, and constructed by or under the supervision of the Jurisdiction as a public improvement and maintained by the Jurisdiction.

2. Are initiated, designed, and constructed by the private owner/developer's private engineer and contractor. Upon acceptance of the improvements in the local Jurisdiction system, the improvements are maintained by the Jurisdiction.

Those improvements that require review and approval by the Jurisdiction, but will remain under private ownership, may be required to follow the SUDAS Design Manual. Each jurisdiction will decide on their own if these types of improvements are to follow the SUDAS Design Manual.

B. Intent of the SUDAS Design Manual

The values contained herein are considered fundamental concepts of basic design criteria that will serve as a framework for satisfactory design on new public improvements. The Project Engineer is encouraged to develop the design based on this framework and tailored to particular situations that are consistent with the general purpose and intent of the design criteria through the exercise of sound engineering judgment. Situations do arise that require special considerations. Therefore, to eliminate hardships or problems, the Jurisdiction may choose to vary the design criteria, procedures, and regulations. Should variances from the SUDAS Design Manual be required, the reason for the variance should be documented and evaluated on a case-by-case basis.

The design standards as described for new public improvements may not be attainable for restoration and rehabilitation projects. Each project of this type must be considered individually to determine if these design standards apply.

The SUDAS Design Manual and the Jurisdiction’s supplemental design standards should be used for the preparation of all design plans for new improvements or major reconstruction submitted by the Project Engineer for Jurisdictional review. The Jurisdiction will review all submittals for general compliance with the specific design criteria, procedures, and regulations. Approval by the Jurisdiction does not relieve the Project Engineer from the responsibility of ensuring that the calculations, design, and plans are accurate or comply with the SUDAS Design Manual and fit the needs of a particular project.

The technical criteria not specifically addressed in the SUDAS Design Manual should follow the provisions of each jurisdiction's own policy or criteria.

C. Organization of the Manual

The SUDAS Design Manual is organized into fourteen chapters: General Provisions, Stormwater, Sanitary Sewers, Water Mains, Roadway Design, Geotechnical, Erosion and Sediment Control, Recreational Trails & Sidewalk, Utilities, Street Tree Criteria, Street Lighting, Parking Lots, Traffic Signals, and Trenchless Construction. The chapters include general information, report documentation, plan design, and federal and state requirements. The manual provides a compilation of readily available literature relevant to the design of urban facilities. The chapters are designed so that revisions can be made by updating the effected sections, as necessary, to reflect up-to-date engineering practices and changes in technology.

D. Jurisdiction and Agencies

The SUDAS Design Manual applies to participating local governments except where superseded by state and federal requirements.

E. Amendment and Revisions

The standards and criteria will be amended as new technology is developed and/or experience gained in the use of SUDAS Design Manual indicates a need for revision. The revisions will be adopted and jurisdiction engineers will monitor the performance and effectiveness of the design standards and will recommend changes and/or amendments through the SUDAS program as needed.

F. Enforcement Responsibility

Each jurisdiction is responsible for enforcing the adopted provisions of the SUDAS Design Manual.

G. Interpretation

The Jurisdiction will make the interpretation and application of the SUDAS Design Manual. The following classification of improvements and definitions are provided for a clearer understanding of general policy.

H. Innovation

Nothing in the SUDAS Design Manual limits the designer's use of new and innovative technology. Each alternative proposed utilizing new or unproven technology must receive approval prior to implementation. Materials meeting the technical specifications should be allowed unless specifically prohibited by the Jurisdiction.

Classifications of Improvements and Definitions

A. Jurisdictional Engineer

The local Jurisdiction's authorized representative who is appointed to carry out the provisions of the SUDAS Design Manual and the SUDAS Standard Specifications (referred to as the SUDAS Specifications).

B. Project Engineer

The person, firm, or corporation who is legally responsible for the design and/or administration of the project. The local jurisdiction may require designating a specific person as the Project Engineer.

C. Inspector or Construction Observer

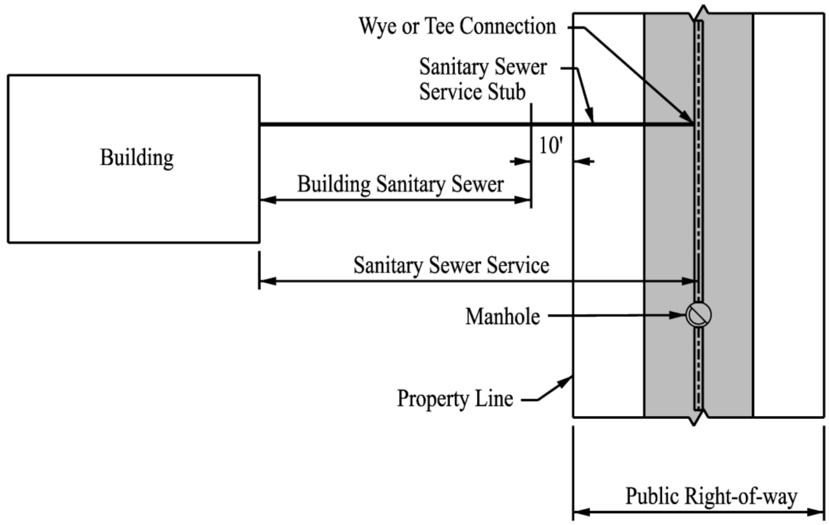
The Project Engineer or the Jurisdictional Engineer may appoint inspectors to inspect all materials used and all work done. Such inspection may extend to any or all parts of the work and to the preparation or manufacture of the materials to be used. The inspectors will not be authorized to revoke, alter, enlarge, or relax the provisions of the specifications. When an inspector is placed on a project, the inspector will keep the Project Engineer or the Jurisdictional Engineer informed as to the progress and quality of the work and the manner in which it is being done.

D. Sanitary Sewer Service Stub

The portion of the sanitary sewer service that is within the public right-of-way to a designated point beyond the right-of-way line (normally 10 feet) as specified by the Jurisdictional Engineer. The sanitary sewer stub may be constructed in conjunction with the sanitary sewer construction. Check with the local jurisdiction to determine if the sanitary sewer service stub is public or private and the exact permit and construction requirements.

Construction Standard: Jurisdiction plumbing code; Jurisdiction plumbing permit required where applicable; SUDAS Specifications.

Figure 1B-1.01: Example of Sanitary Sewer Service

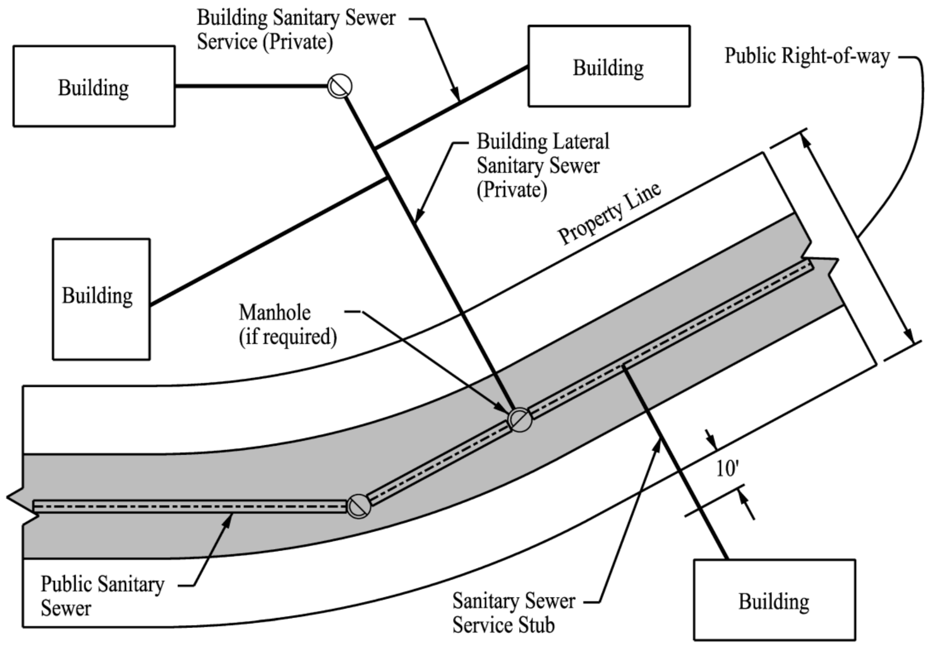


E. Private Lateral Sanitary Sewer

A sewer used to convey sanitary sewage from one or more sanitary sewer services. This sewer is limited to providing service to one owner or homeowner's association. This sewer is to be owned and maintained by a single person or entity and constructed on private property controlled by the owner or homeowner's association. For location of private lateral sanitary sewer, see Figure 1B-1.02.

Construction Standard: SUDAS Specifications; local agency plumbing permit and Iowa DNR permit may be required.

Figure 1B-1.02: Example of Lateral Sanitary Sewer



F. Main (Trunk) Sanitary Sewer

A sewer used to receive and convey sanitary sewage to another trunk sewer or a sanitary interceptor sewer. This sewer is owned and maintained by the Jurisdiction and is constructed on public property or on private property with an easement held by the Jurisdiction.

Construction Standard: SUDAS Specifications; Iowa DNR permit required.

G. Sanitary Sewer Lift Station

A facility used to convey sanitary sewage from one or more sanitary sewers that cannot be conveyed by gravity flow to the public sewer system. This facility is owned and maintained by the Jurisdiction. Warning alarms may be required to automatically communicate to locations designated by the Jurisdiction. This facility is constructed on property deeded to the Jurisdiction or on private property with an easement held by the Jurisdiction.

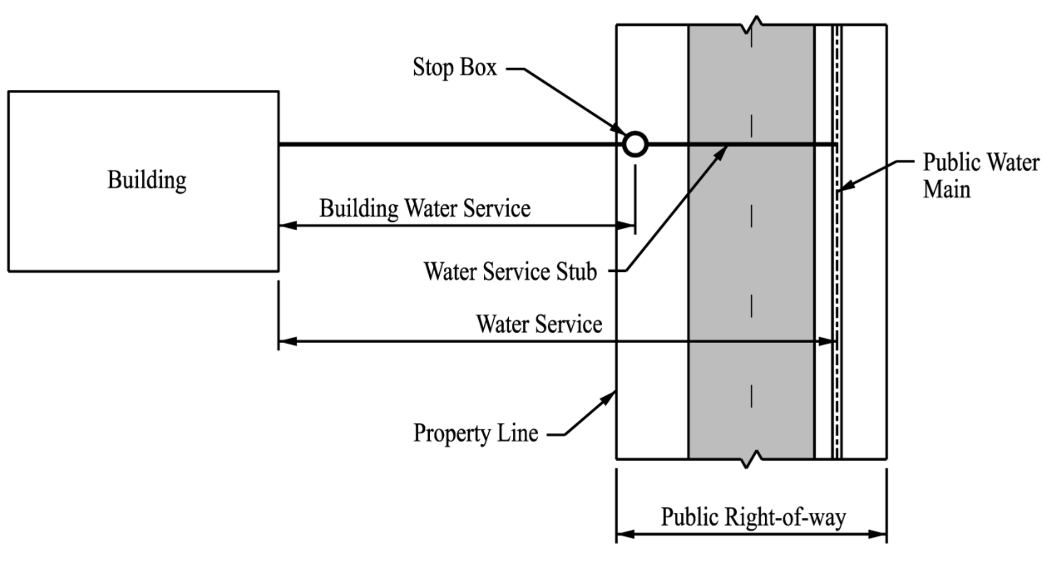
Construction Standard: SUDAS Specifications; Iowa DNR permit required.

H. Water Service Stub

The water service stub is comprised of the piping and related appurtenances including the corporation, installed from the public water main to the stop box or as specified by the Jurisdictional Engineer. For location of the water service stub, see Figure 1B-1.03.

Construction Standard: SUDAS Specifications. Jurisdiction plumbing permit required where applicable.

Figure 1B-1.03: Example of Water Service



I. Private Water Main

A private water main is used to distribute water for domestic and fire fighting purposes to only one owner or homeowner's association. This private water main is to be owned and maintained by only one party and constructed on private property controlled by the owner or homeowner's association. Approval for the use of private water mains must be obtained from the Jurisdiction. Metering of water flowing through the private water main will be subject to Jurisdiction’s water metering requirements.

Construction Standard: SUDAS Specifications; Jurisdiction Water Works and/or Rural Water Association Standards; Iowa DNR and Jurisdiction plumbing permit where applicable.

J. Water Main

A water main is used to distribute water to consumers for domestic, industrial, and fire fighting purposes. The main is owned by the Jurisdiction, water works, or an approved public/private water utility corporation or association.

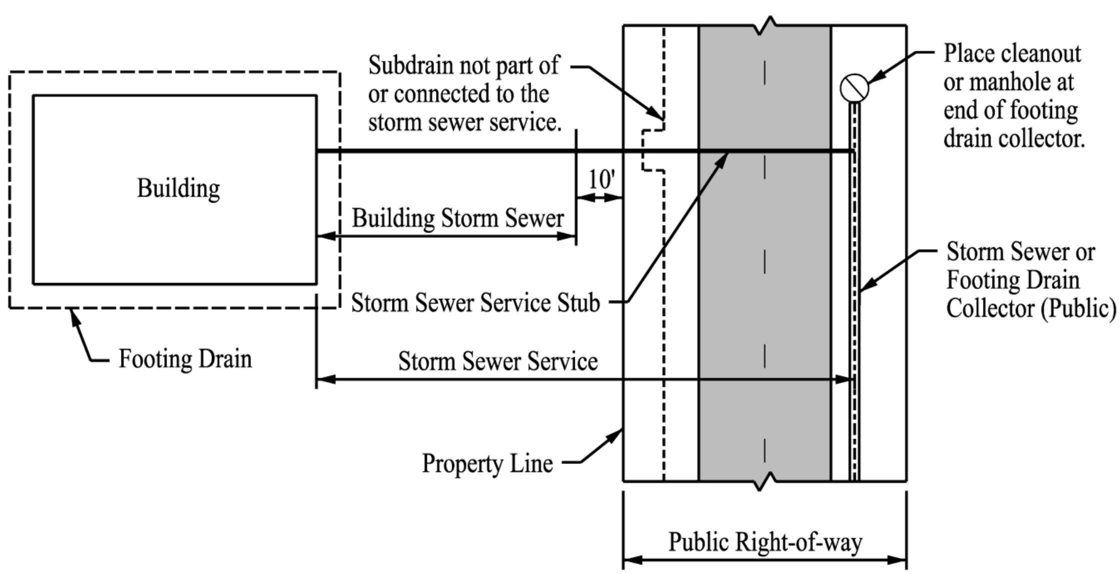
Construction Standard: SUDAS Specifications; Iowa DNR permit required.

K. Storm Sewer Service Stub

The portion of the storm sewer service that is within the public right-of-way to a designated point beyond the right-of-way line (normally 10 feet) as specified by the Jurisdictional Engineer. The storm sewer service stub may be public or private. Verify with the Jurisdiction. The storm sewer service stub may be constructed in conjunction with the footing drain collector or storm sewer construction. For location of the storm sewer service stub, see Figure 1B-1.04.

Construction Standard: SUDAS Specifications; Jurisdiction plumbing code; Jurisdiction plumbing permit may be required where applicable.

Figure 1B-1.04: Example of Storm Sewer Service Stub



L. Private Storm Sewer

A private storm sewer is used to convey stormwater from private property to a public storm sewer, natural drainage way, or other acceptable outlet. These sewers should be designed to fit within the Jurisdiction's overall drainage system. Easements are to be obtained when crossing other private property. Drainage area limits for private storm sewers of large sites will be examined on a case by case basis by the Jurisdiction. This sewer is located on private property and maintained by only one party or homeowner's association. For location of private storm sewer, see Figure 1B-1.05.

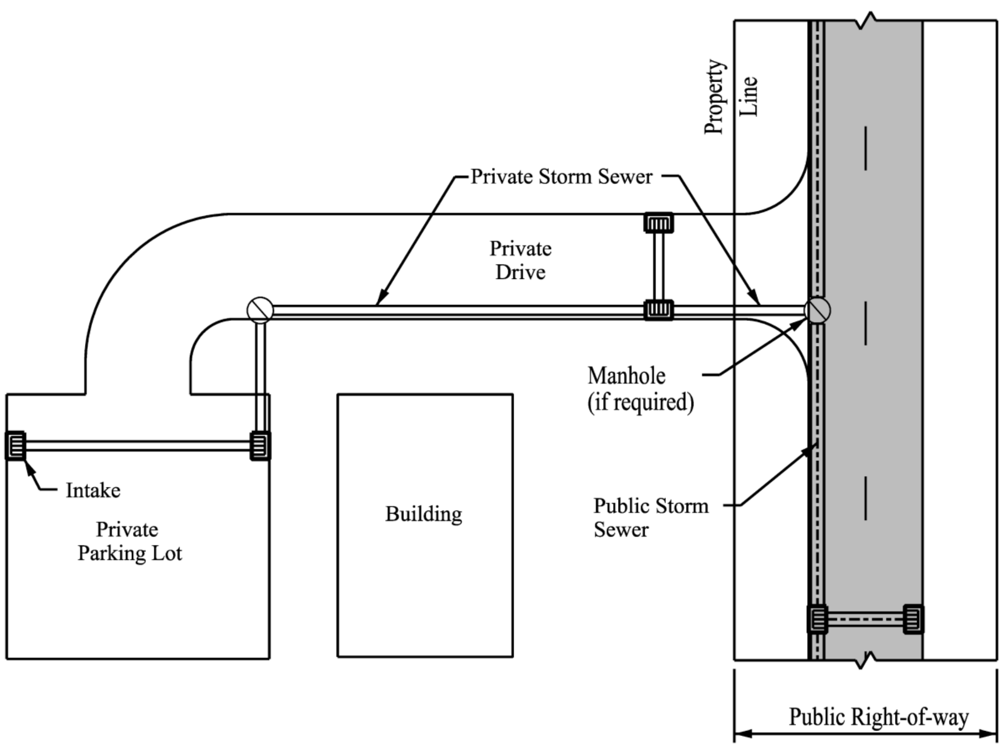
<u>Construction Standard:</u> SUDAS Specifications; Jurisdiction plumbing permit may be required; federal and state permits may be required.

M. Storm Sewer

A storm sewer is used to convey stormwater runoff to an acceptable outlet. This sewer is owned and maintained by the Jurisdiction and constructed on public property or on private property with an easement held by the Jurisdiction. For location of storm sewer, see Figure 1B-1.05.

<u>Construction Standard:</u> SUDAS Specifications; Federal and State permits may be required.

Figure 1B-1.05: Example of Public and Private Storm Sewers

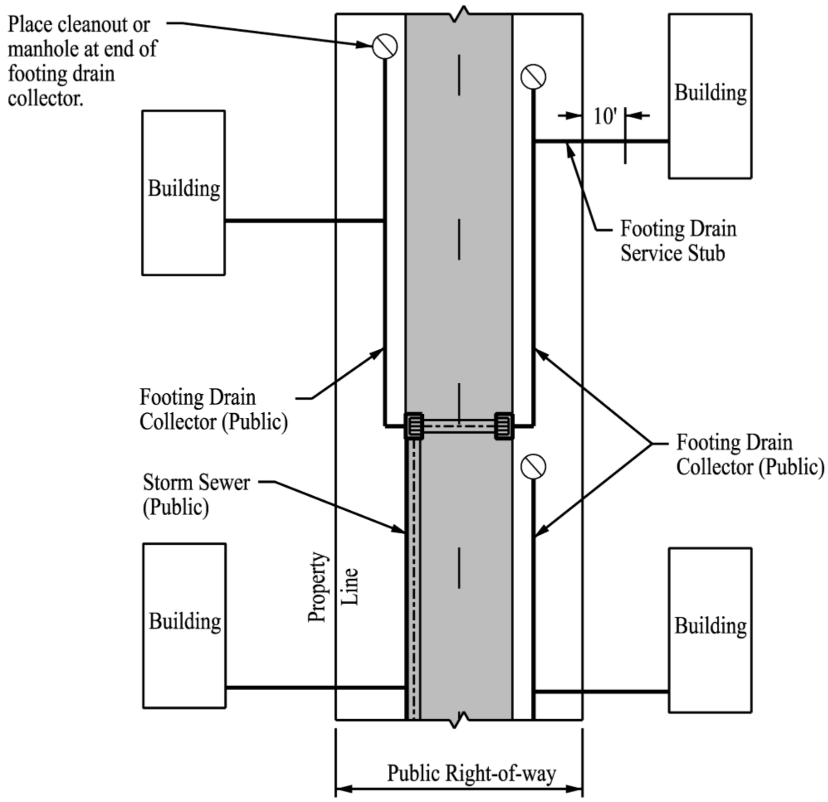


N. Footing Drain Collector

A footing drain collector is used to convey ground water from private footing drains to a public storm sewer or drainage way. This footing drain collector is owned and maintained by the Jurisdiction and constructed on public property or on private property with an easement held by the Jurisdiction. For location of footing drain collector, see Figure 1B-1.06.

Construction Standard: SUDAS Specifications.

Figure 1B-1.06: Example of Footing Drain Sewer

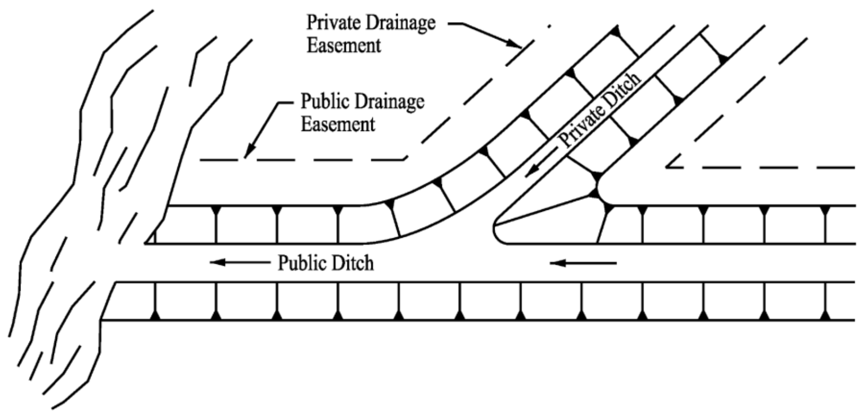


O. Private Ditch

An open drainage way, swale, or manmade channel used to convey stormwater drainage to the public drainage system. Private ditches may be allowed on a case-by-case basis. The channel should be designed to accommodate the Jurisdiction’s overall drainage system needs with respective easements that will serve more than one property and will be located on private property and maintained by one party or homeowner's association. Take care to provide good grades to prevent low points, but also do not create erosion. The ditch may discharge directly into a stream or other waterway. For location of private ditch, see Figure 1B-1.07.

Construction Standard: SUDAS Specifications; Federal and State permits may be required.

Figure 1B-1.07: Example of Private Ditch and Public Ditch



P. Ditch

A natural channel improvement or manmade channel required by the Jurisdiction as a component of a planned drainage system that conveys stormwater drainage across public property or public easement. Public ditches should be designed to accommodate the Jurisdiction's overall drainage systems needs. The use of buried storm sewer in or nearby the private ditch that will accommodate low flows of minor storms is encouraged. Public ditches are owned by the Jurisdiction or within an easement held by the Jurisdiction. For location of ditch, see Figure 1B-1.07.

<u>Construction Standard:</u> SUDAS Specifications; contact Iowa DNR for potential 401 Water Quality and NPDES permit requirements; U.S. Army Corps of Engineers for 404 permit.

Q. Private Runoff Detention

A basin used for on-site stormwater runoff storage and controlled release. The detention facility should be designed to accommodate the Jurisdiction’s overall drainage system needs with the intent to not increase the existing rate of discharge from the site. (See Chapter 2 for details).

<u>Construction Standard:</u> SUDAS Specifications - Jurisdictional Engineer's Approval; Iowa DNR permit may be required.

R. Runoff Detention

A basin used to meet the Jurisdiction’s stormwater management plan goals. These facilities should be designed to accommodate the Jurisdiction’s overall drainage system needs. This detention basin is located on public or private property (with easements) and is maintained by the Jurisdiction.

<u>Construction Standard:</u> SUDAS Specifications; Federal and Iowa DNR permits may be required.

S. Entrances

Access to private property is the responsibility of the property owner. Any change in existing property use that requires a modification to the entrances will be the responsibility of the owner to obtain an entrance permit.

<u>Construction Standard:</u> SUDAS Specifications; Jurisdiction permit required.

T. Private Street

A street that is restricted to use by only one owner or homeowner's association and is available for use by emergency vehicles. This classification of street is located on private property and maintained by only one party or homeowner's association. Private streets should meet all applicable geometric requirements for the given operating speed and pavement thickness requirements for the type of traffic, but may be deficient in other elements, such as right-of-way width. (See Chapter 5 for details). Approval for the use of private streets must be obtained from the Jurisdiction.

Construction Standard: SUDAS Specifications; Jurisdiction permit may be required.

U. Public Street

This classification of street is owned and maintained by the Jurisdiction and constructed on dedicated street right-of-way. (See Chapter 5 for detailed description of each roadway system element).

Construction Standard: SUDAS Specifications.

V. Franchise Utility

A Jurisdiction may grant a franchise to erect, maintain, and operate underground and overhead plant and systems. These systems could be for electric light and power, heating, telephone, cable television, water works, gas, or other utilities within the Jurisdiction. Construction of said facilities could be in the public right-of-way. Location of franchised utilities should take into account the future right-of-way needs based on the ultimate classification of the street. If easements are obtained for the utilities, it is recommended these easements be obtained in the name of the jurisdiction. All franchise utility installations should abide by the same design and construction requirements as other improvements.

W. Public and Non-franchised Utility

The Jurisdiction may allow the installation of public and non-franchised utilities in public right-of-way upon review of the proposed improvements and approval by the Jurisdiction. Such improvements may include, but not be limited to, water mains constructed by a water board, electric facilities constructed by an electric board, stormwater facilities, storm sewers, fiber optic lines, communication lines, irrigation systems, and other miscellaneous installations.

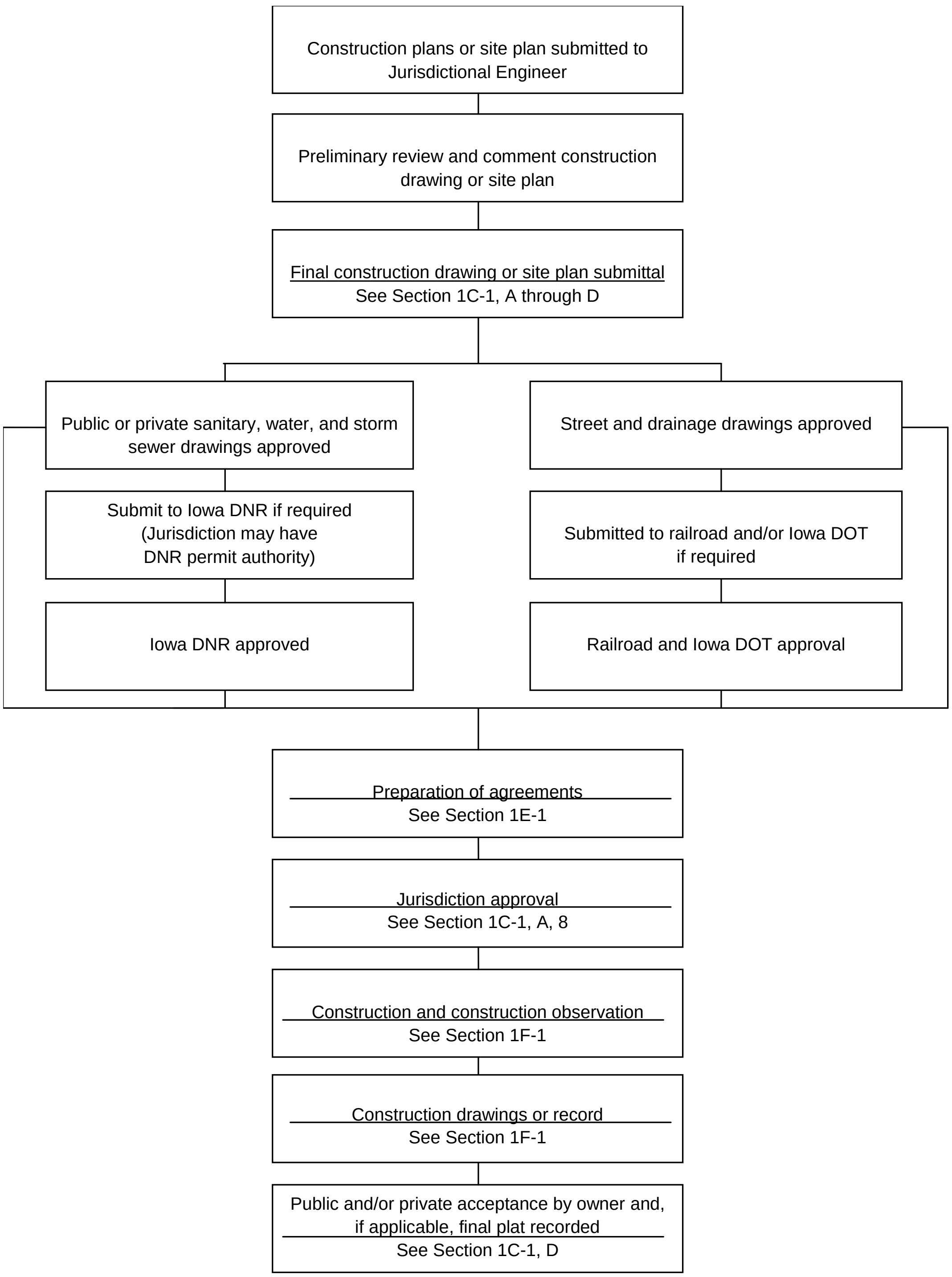
Ensure the installation of such facilities in public right-of-way does not damage or infringe on the usefulness of existing facilities. Upon receipt of a written notice from the Jurisdiction, the owner of a public and non-franchised utility must remove the utility from the Jurisdiction's right-of-way or relocate it within the right-of-way.

X. Utility Conflicts

Franchised, public, and non-franchised utilities are expected to cooperate in relocation of facilities that are in conflict. It is critical that the utilities be given as much advance notice as possible. The Project Engineer should coordinate with each utility agency or company to determine location and elevation of all utilities located within the project area. If any existing utilities conflict with the proposed project, the Project Engineer should contact the utility company and work to resolve the conflict in order to keep the project on schedule. If the conflicts are unable to be resolved, the Project Engineer should bring the matter to the attention of the Jurisdictional Engineer.

Submittal Procedures

Figure 1C-1.01: Submittal Flow Chart



A. Construction Plans and Specifications Submittal Procedure

1. **General:** Consulting engineers and developers seeking approval and acceptance of civil engineering reports, construction plans, and site plans are required to follow the procedures as established by each Jurisdiction. These procedures are generally outlined in this section and Figure 1C-1.01. The adherence to these procedures will assist in an efficient review of engineering plans and reports. Each jurisdiction reserves the right to modify certain procedures to fit their unique situation.

2. **Pre-submittal Meetings:** Each Jurisdiction may conduct pre-submittal meetings at which developers may ask questions and obtain direction and/or information from the Jurisdiction's staff. These meetings may be used by the developer to obtain very basic information about procedures, practices, or standards as a basis on which to begin development planning. Alternatively, the applicant may use the meeting as a final check by staff to verify a specific type application is complete.

3. **Submittal of Public Improvement and Development Plan Application:** The development plan application, site plans, revised site plans, and other public improvements submitted to the Jurisdiction for any project, subdivision, or planned unit development, whether residential, retail, commercial, or industrial, should include adequate concept drawings for public improvements including any impact reports.

4. **Engineering Review Objective:** The objective of the Jurisdictional Engineer is to complete the initial review and issue comments according to the schedule prescribed by the Jurisdiction to prevent delaying further review by other agencies or impact any other scheduling, such as subdivision platting.

5. **Results of Engineering Review:** After the review is completed, the check prints and comments report will be returned to the Project Engineer.

6. **Revision of Engineering Plans and Reports:** The Project Engineer will make all the revisions requested on the original plans/report and re-submit. Seriously deficient plans may require several reviews prior to approval.

7. **Revision of Plans and Reports:** When submitting revised plans, drawings, or reports to the Jurisdictional Engineer, the re-submittal must contain the following.

   a. The revised plans for review.

   b. All check prints from previous reviews with copies of the previous plans. Notations should be made after each comment if the correction was made or justification why a comment is not valid.

   c. If fees are applicable, they must accompany the application.

   If all of the above are not submitted, the re-submittal may be returned without further action until such time as the submittal is complete.

8. **Approved Plans:** When plans or reports have been conditionally approved by the Jurisdictional Engineer, the designer should submit reproducible copies of original plans on stable plastic film or other media as designated by the Jurisdiction for approval. The reproducible copies should be accompanied by three blueline or blackline copies for use by the Jurisdiction. If the project relates to a development, original engineering plans for public improvements may be approved by

the Jurisdictional Engineer, only after the approval of the preliminary plat, the land dedication, and the subdivision improvements agreement associated with property.

9. **Resubmittal of Plans:** The objective of the Jurisdictional Engineer is to complete resubmittal reviews and issue comments/approval according to the schedule prescribed by the Jurisdiction to prevent delaying further review by other agencies or impact any other scheduling, such as subdivision platting.

10. **Order of Processing:** The following policy regarding order of processing (priority) will be used for all submittals. Applications are normally processed on a first come basis.

    a. Mylars or final media for approval.

    b. Resubmittal, complete package.

    c. Initial submittal, complete package.

    Complete submittals include all drawings and supporting reports.

    When plans are returned to the Project Engineer for lack of adequate information, or in the event of re-platting or major site plan revisions after the initial review, the re-submittal will be considered a new submittal rather than a return. A thorough technical review will be started by the Jurisdiction when adequate information is provided.

B. Updates to Previously Approved Plans

1. Construction plans, pavement design reports, drainage reports, site plans, and other documents are approved initially for 12 months. If not constructed during this time period they automatically become void and must be updated to current criteria before any further permits can be issued. The Jurisdictional Engineer may grant an extension to the construction plans, pavement design reports, and drainage report validity period; provided a) the development plan, construction plans, or reports have not substantially changed, and b) that other conditions affecting the development site have not substantially changed or do not require a modification to approved plans or specifications.

2. Whenever updates or revisions to previously approved construction plans, specifications, or drainage reports are necessary, the Project Engineer will submit updates or revisions through the normal document submittal process. After all Jurisdictional Engineer comments and revisions have been incorporated, the construction plans or reports containing revisions may be submitted for approval by the developer.

3. Requests for updates and revisions will be considered only if there are no revisions to the original development plan(s) or drainage report. The Jurisdiction will review the original development plan(s) or drainage report for compliance with current standards under normal review procedures (requests for updates will be considered a resubmittal), and if found in compliance with current standards, the construction plan(s) or drainage report(s) will be approved.

C. Submittal Checklist

1. The following documents should be submitted for review and approval when preparing final construction plans for public subdivision improvements or other public improvements.

    a. Street plan and profile.

    b. Storm sewer plan and profile, including details for all structures and material specifications.

    c. Culvert plan, profile, and construction detail for structures.

    d. Permanent traffic signing and striping plan.

    e. Pavement design where required with supporting geotechnical report.

    f. Grading and erosion control plan.

    g. Sanitary sewer plan and profile including details for all structures, material specifications and sewer treatment agreement.

    h. Water construction plans as approved by the governing Jurisdiction or utility with a water supply agreement. If these plans represent lines to be installed with the proposed roadways, the plans must be approved by the Jurisdictional Engineer.

    i. Plan for traffic control during construction.

    j. Engineering review and approval fee.

    k. All appropriate permits from the Jurisdiction, State, and Federal agencies.

D. Final Acceptance

Upon completion of construction of a public project, the improvements will be accepted by the Jurisdiction and/or Water Board into the public system upon submittal of the following, if applicable:

1. Approval final plat.

2. Title opinions.

3. Consent to plat by owner and any persons having an ownership interest in the property to be platted.

4. Easements and deeds dedicated to Jurisdiction (Plat of Acquisition required for any property that is not included in the subject plat or property). Declaration of Value and Groundwater Hazard and/or Restrictive Covenants Statement should accompany any property that is deeded to Jurisdiction.

5. Supplemental agreements as required.

6. Maintenance bonds for improvements.

7. Performance bonds for uncompleted work.

8. Certified statement from the County Treasurer that the taxes are paid in full on the property to be subdivided.

9. Certified statement from the County Clerk and County Recorder stating the property to be subdivided is free from liens and all other encumbrances.

10. Submit required certifications that improvements have been constructed according to the approved plans and specifications as required by the Jurisdiction.

11. Submit a certified "as-built" set of plans.

Certain jurisdictions may require additional fees (i.e. sewer, park, etc.) and may require the submittal of the items listed above before construction commences.

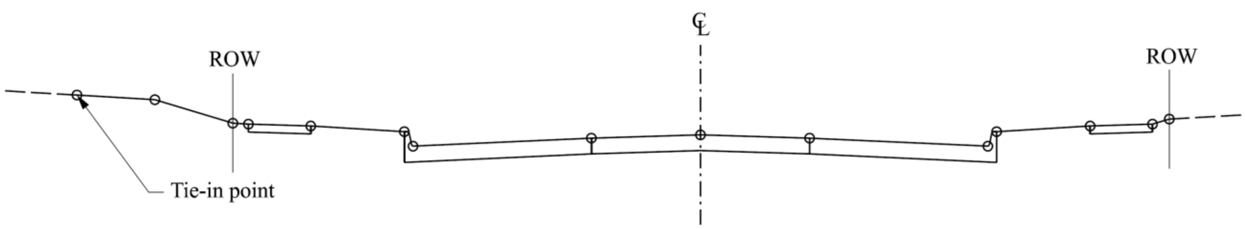
Detailed Plans for Construction of Public Improvements

A. Public Improvement Plan Sheet Requirements

Detailed reproducible plans, certified by a licensed professional engineer in the State of Iowa, should be filed with the Jurisdiction for all work involved in Public Improvement Contracts and/or agreements.

When providing Computer Aided Design (CAD) files, ensure they contain all break lines used to develop a 3-D file showing coordinates (x,y,z) needed to accurately represent the paper design plans. Break lines should be shown according to the cross-section below. In addition, break lines within the 3-D file should indicate all locations within the project limits where there is a change of slope.

The 3-D file should be available to potential bidders at the same time that the paper plans are available to the bidders and filed with the Contracting Authority. A disclaimer statement should also be included that indicates the paper copy on file with the agency is the official copy and the contractors are responsible for constructing the project to those plans.



Detailed plans should comply with the following general requirements.

1. **Plan Organization:** It is important, if for no other reason than uniformity for contractors, that the plan sheets are arranged consistently from one plan set to another. In general, the sheets should be arranged in the following order, which is consistent with Iowa DOT plans, where possible.

Page Number	Description
A.01	Title sheet
A.02	Legend sheet
A.03, A.04, …	Map sheets (if needed)
A.0*, … (*whatever number follows the previous A sheets)	Revision sheets (if needed)
B.01, B.02, …	Typical cross-sections
C.01, C.02, …	Estimate of quantities, general information, and erosion control and SWPPP sheets
D.01, D.02, …	Plan and profile sheets - mainline
E.01, E.02, …	Plan and profile sheets - side road and channel change
F.01, F.02, …	Plan and profile sheets - detour
G.01, G.02, …	Reference ties and bench marks
H.01, H.02, …	Right-of-way sheets (urban)
J.01, J.02, …	Staging and traffic control sheets
K.01, K.02, …	Interchange geometric staking, jointing, and edge profiles
L.01, L.02, …	Intersection geometric staking, jointing, and edge profiles
M.01, M.02, …	Storm sewer sheets
MIT.1, MIT.2, …	Wetland sheets
MSA.1, MSA.2, …	Sanitary sewer sheets
MWM.1, MWM.2, …	Water main sheets
N.01, N.02, …	Traffic signal, permanent pavement markings, and permanent signage sheets
P.01, P.02, …	Lighting layout sheets
Q.01, Q.02, …	Soils and soils stabilities sheets
R.01, R.02, …	Removal sheets
S.01, S.02, …	Sidewalk sheets
T.01, T.02, …	Tabulation of earthwork quantities
U.01, U.02, …	500 series, modified standards, and special details
V.01, V.02, …	Bridge and culvert situation plans
W.01, W.02, …	Cross-sections - mainline
X.01, X.02, …	Cross-sections - side roads and channel change
Y.01, Y.02, …	Cross sections - ramps and loops
Z.01, Z.02, …	Cross sections - borrows

All of the above mentioned sheets will not necessarily occur in every plan, but those that do should remain in the same relative order and use the letter designation listed above.

2. **Plan Sheet Material:** Plans filed with the Jurisdiction should be on media designated by the Jurisdiction.

3. **Plan Sheet Size:** Check with the Jurisdiction for appropriate plan sheet sizes.

4. **Title Sheet:** The following information should be shown when applicable.

    a. Project name title and location.

    b. Jurisdiction’s name.

    c. Small scale vicinity map showing project location.

d. Index (a complete sheet index is to be shown).

e. File number/project number (to be filled in by the Jurisdiction).

f. Engineer's firm name and address.

g. Signature line for Jurisdiction Authority.

Sample:

REVIEWED:

Jurisdiction Authority	Title	Date

h. Sheet number and total number of sheets.

i. Project Engineer's certification, registration number, and date certified. All official plans should be certified according to the requirements set forth by the Iowa Engineering and Land Surveying Examining Board.

   A professional engineer or other licensed professional must certify every sheet in a plan. This is done by indicating on the signature block which sheet numbers are covered by the licensee's seal. The engineer's signature and date must be handwritten in ink that contrasts in color with the plan's print.

   Parts of the plan that are designed under the direction of other licensed professionals should have separate signature blocks that related to their part of the project. These signature blocks should be placed on the first plan sheet that is covered by their seal.

   Each signature block should list the sheet numbers that are covered by the licensed professional's seal. Plan information prepared under the direction of separate licensees should not be placed on the same sheet. In other words, two licensed professionals should not be responsible for the same plan sheet.

   In addition to the separate signature blocks, an index of seals should be tabulated on the title sheet if parts of the plan are being certified by other licensed professionals. The tabulation would identify the sheet number where any other signature blocks are located, as well as the licensee's name and area of design.

j. Note that projects should be constructed according to the SUDAS Standard Specifications.

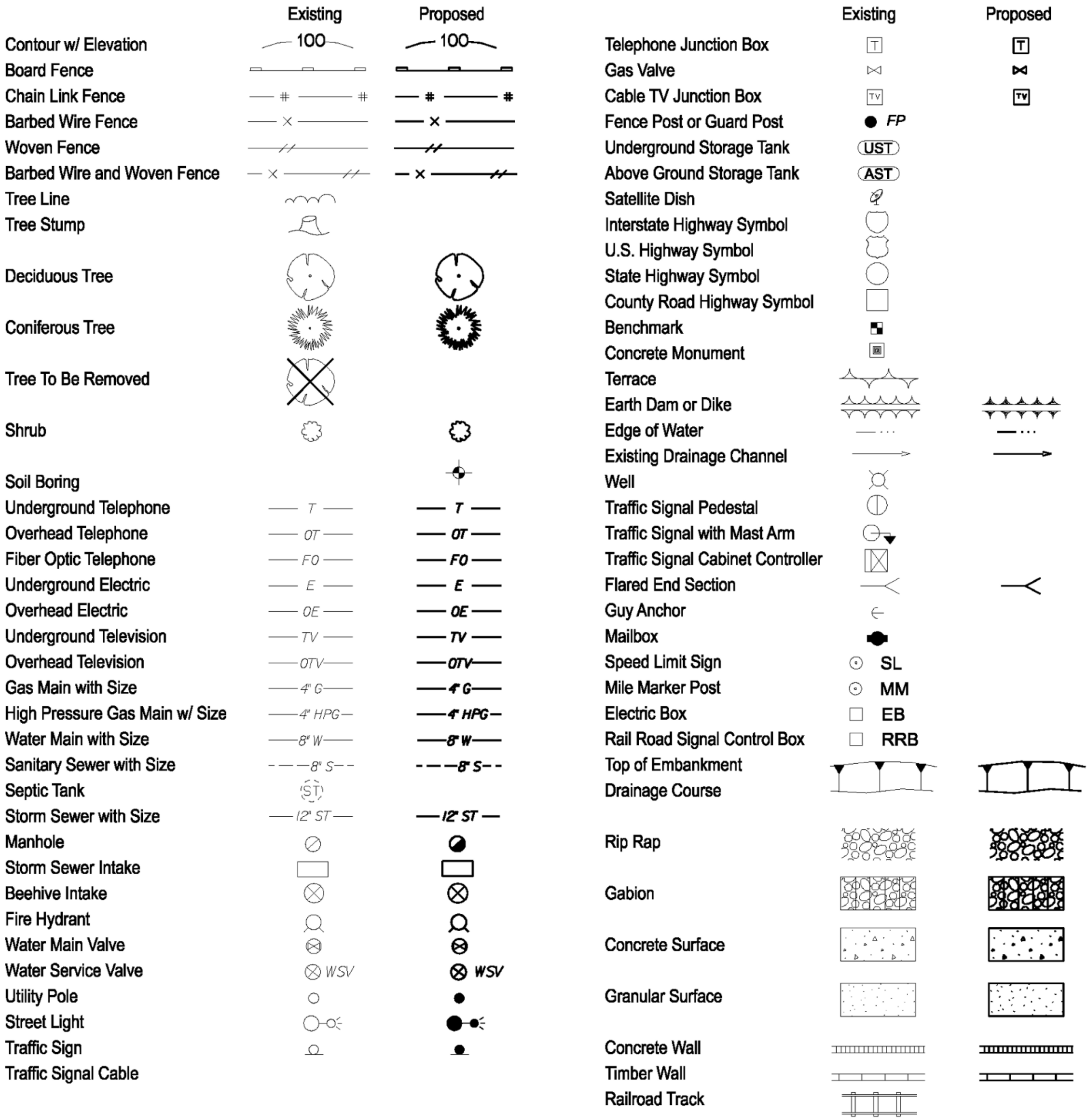
k. Listing of standards (if applicable).

l. Owner/developer.

m. Legend (see Figure 1D-1.01 for sample legend).

   The Jurisdictional Engineer may require different legends depending on the designated design software package. The Project Engineer should ensure that the completed design plan complies with the Jurisdiction's requirements for symbols and the design information to be placed on specific layers within the software program.

Figure 1D-1.01: Sample Legend



5. **Title Block:** A title block listing the following information.

   a. The name of the project.

   b. Owner and Project Engineer along with sheet title (including address and phone number).

   c. Date.

   d. Sheet number and total number of sheets.

   e. Space that denotes revisions.

   f. Place in lower right corner or right edge of each sheet so it can be read from the bottom or right side. Check with the Jurisdiction involved for correct location.

   g. Insert page numbers in the lower right-hand corner.

   h. All persons designing, detailing, and checking plans should legibly place their names in the title block on the title sheet in a space provided for this purpose.

6. **Plan Scale:** Vertical scale: 1" = 5'
   Minimum horizontal scale: 1" = 50'

   or 1" = 20' or larger if details for sanitary sewer, storm sewer, paving, and/or sidewalk are on same plans.

   Changes to above scale to be approved by the Jurisdictional Engineer.

   A bar scale is required on each drawing.

   Prepare drawings and lettering of such a scale as to be reproducible to 1/2 scale.

B. General Information to be Shown on the Construction Plans

1. Beginning (B.O.P.) and ending (E.O.P.) of project.

2. Street names.

3. Right-of-way widths and legal descriptions as required.

4. Legend as part of title sheet requirements.

5. Adequate witnesses and horizontal and vertical controls so surveyor can lay out project plans. Show all controls at actual locations on the plans. Benchmarks and ties.

6. Lot numbers, subdivision names, and project numbers, as applicable.

7. Lot dimensions (along right-of-way or easements).

8. North arrow up or to the right, when applicable.

9. Existing and proposed utilities, including type, size, and location.

10. Proposed improvement locations, dimensions, and stations.

11. Station Bar (all improvements shall be referenced to same stationing). Stationing from left to right or bottom to top.

12. Existing trees, fences, walks, drainage structures, ditches, pavements, buildings, and other obstacles or improvements that could reasonably affect the work area.

13. Survey line or reference line shown on plan view with stations increasing from west to east or south to north, when practical.

14. Quantity estimate - separate sanitary sewer, storm sewer, and paving quantities shown if they are detailed on same plan. Include estimate reference information listing any special requirements for each bid item.

15. Easements, both temporary and permanent.

16. Cross-sections - for subdivisions, existing and proposed finished contours may also be used.

17. Special details and special notes when required.

18. Plan view and profile. Profile should line up with plan stations whenever possible.

19. Plans for development work should contain a general note to construct the project according to the SUDAS Standard Specifications.

20. Symbols and abbreviations used on plans if different from those shown in Standard Specifications.

21. Make reference to soils report.

22. Traffic control signs and markings will follow the latest edition of the Manual on Uniform Traffic Control Devices (MUTCD). When it is required by the Jurisdiction to maintain traffic during construction, show stage construction and special requirements on the plans. If required, show signing, street closures, and/or detours on traffic control sheet.

23. Permanent signing.

24. Stormwater pollution prevention plan; temporary and permanent erosion control measures proposed.

25. Other information deemed necessary by the Engineer certifying the plans.

C. Detailed Sanitary and Storm Sewer Plans

1. Stationing, location, and type of all manholes, intakes, or other structures.

    a. Show structure designation on the plans.

    b. Show location on the plans and reference survey line or centerline.

    c. Comply with the SUDAS Standard Specifications for the type of structure required.

2. Details should be shown for all structures that are not standard in the SUDAS Standard Specifications.

3. Plan and profiles of all sewer lines and ground line above sewer.

4. Size, length, and grade of sewers in profile.

5. Type of pipe materials and strengths, if different from SUDAS Standard Specifications, or if specific materials are required.

6. Invert elevations at all intakes, manholes, and other structures in profile.

7. Location, size, and type of all sewer stubs, wyes, or tees. Reference stub locations to lot corners. When risers are to be installed, show riser location and size.

8. Estimates should include all length of pipe stubbed out from structures.

9. Rim elevations of manholes.

10. Ensure all castings comply with the Jurisdictional requirements on sewers to be maintained by the Jurisdiction.

11. Manholes should be identified with a numbering system on plan and profile. Structure sizes and casting sizes to be included by schedule or note on the plans.

12. Class of pipe bedding.

13. Existing utilities or other underground features that could reasonably affect the construction and maintenance of the sewer.

14. Storm sewer design calculations need to be submitted showing drainage area, flow patterns, and flows for design storms. (Hydraulic grade line data).

D. Detailed Drainage Ditch and Drainageway Plans

1. Stationing and flow line elevation at beginning and end of ditch construction.

2. Plan and profile of drainage ditch.

3. Size, type, length, and grade of ditch and alignment.

4. Typical sections showing ditch dimensions, backslopes, and invert and slope treatment.

5. Invert elevations at all structures.

6. All special structures detailed on plans.

7. Criteria for hydraulic design data and elevations.

8. Cross-sections and contour map showing existing ground and finished grade.

9. Permanent and temporary erosion controls.

E. Detailed Paving Plans

1. Minimum 100 feet station intervals and profile elevations at a minimum of 50 feet intervals on tangents and 25 feet intervals at curves. Show station of the centerline of all intersecting streets.

2. Show street profiles and existing ground elevations in the profile view and the curb line in the plan view. The profile should show top of curb tangent grades, vertical curve data, and break grade data.

3. Pavement width (back-to-back).

4. All radii at returns (may be specified in general note if all radii are same).

5. Expansion joint locations, if applicable, on plan view.

6. Horizontal curve data should include centerline PC, PT, PI, delta angle, arc length, degree of curve, tangent length, and radius.

7. Typical cross-section showing referenced profile, subgrade treatment, pavement thickness, jointing, sidewalk, parking slope, foreslopes, backslopes, cross-slopes, any break in ground line or grade, right-of-way line, and dimension of the location of the roadway with the right-of-way line.

8. Vertical curve data should include station and elevation of PI, PC, PT, K-value, low point, and length of curve. Elevations should be given on curves at 25 foot spacings.

9. Break grade - station and elevation.

10. Intersection details showing drainage and typical joint patterns, if applicable.

11. Location and type of standard sidewalk ramps.

12. Special subgrade or pavement treatment.

13. Location of existing pavement, including elevation and grades.

F. Grading Plans/Erosion Control Plans

1. Survey control data.
2. Cross-sections and/or existing and proposed contours, as required.
3. Bar scale (north arrow).
4. Storm sewer/detention appurtenances.
5. Vicinity map showing haul routes with dates, if any, and borrow areas.
6. Total site area (disturbed area) with construction staging to minimize the area disturbed at any one time.
7. Stationing as it relates to paving plans, sewer, or drainageway plans.
8. Geometric dimensions.
9. Soils data and soil boring location when applicable.
10. Erosion control information and location of any special erosion control measures such as silt fences, silt traps and basins, rip rap or gabions, vegetation and trees to remain, stockpile areas, terraces, contour furrows, temporary diversions, grading phases, etc. See Chapter 7 for a detailed listing of the required contents of Iowa DNR Stormwater Pollution Prevention Plan. Also include the name and 24 hour telephone number of the individual responsible for maintaining erosion control measures.
11. Top soil stockpile and stabilization measures and vegetation areas to be preserved.

G. Water Main Plans

The plans for water mains and appurtenances should show all appropriate physical features adjacent to the proposed water mains along with horizontal and vertical controls and hydrant coverage. Other utilities such as sanitary and storm sewers, manholes, etc. should be shown on the plans with horizontal and vertical separation distances. Design details for other utilities that do not affect the water main will not be shown on water main plans.

1. Stationing, location, and type of all fittings, valves, and fire hydrants.
2. Details should be shown for all items that are not standard in the SUDAS Standard Specifications.
3. Plan and profiles of all water lines and the ground line above the water main.
4. Size, length, and grade of water mains in profile.
5. Type of pipe materials and strengths if different from the SUDAS Standard Specifications or if specific materials and fire hydrants are required.

6. Elevations at all structures in profile.

7. Location, size, and type of all water service stubs. Stub locations should be referenced to lot corners.

8. Estimates should include length of pipe stubbed out from valves.

9. Fire hydrants should be identified with numbering system on plan and profile.

10. Class of pipe bedding if different than the SUDAS Standard Specifications.

11. Existing utilities or other underground features that could reasonably affect the construction and maintenance of the water main.

H. Railroad Crossings

If a railroad crossing is within the project limits, the Project Engineer should notify the railroad with a copy of the plans and specifications a minimum of 4 months prior to the project letting. If the project limits contain construction of railroad facilities that will be performed by the railroad's forces, the Project Engineer will state this in the contract documents. The contract documents will state the Contractor's limits of responsibility and allow sufficient time in the schedule for the work to be accomplished by the railroad; and that the Contractor must coordinate its activities with the railroad. The Contractor must be made aware of any permit requirements imposed by the railroad.

The Project Engineer should notify the railroad of the following, immediately after awarding the contract:

1. Federal Railroad Administration (FRA) crossing number*

2. LPA project number

3. Contractor's name, mailing information, and phone number

4. Contractor's contact person

5. Anticipated start date

6. Number of working days

7. Number of days it is believed the Contractor will impact the railroad.

8. Date of preconstruction meeting

* For help in identifying the FRA number, see Iowa DOT Office of Rail Transportation's Highway-Railroad Crossing Identifiers webpage.

I. Items to be Specified on Plans or in Contract Documents

The SUDAS Standard Specifications specify many items and methods that can be used for the construction of improvements. Following is a list of items in the SUDAS Standard Specifications that are to be noted on the construction drawings and/or in the special provisions whenever there is to be a deviation from the standard requirements of the specifications. This information may include specifying pipe sizes and materials, who is responsible for providing compaction testing, as well as many others.

The Project Engineer should review the following list and the SUDAS Standard Specifications to make sure all items that are necessary to construct the project are specified on the plans and/or in the special provisions. Please note - this list is not all-inclusive.

Section 2010 - Earthwork, Subgrade, and Subbase

2010, 1.08 D, 1, a	Specify whenever the depth of cut for stripping and salvaging topsoil is other than 8 inches.
2010, 1.08, E	Specify the class of excavation as Class 10, Class 12, or Class 13.
2010, 1.08, E, 1, b, 2)	When the truck count method is to be used for measuring Class 10 or Class 13 excavation, specify if the shrinkage factor is other than 1.35.
2010, 1.08, E, 4	Specify whenever stripping, salvaging, and spreading 8 inches of topsoil is NOT a pay item and is included in the payment of Class 10, Class 12, or Class 13 Excavation.
2010, 1.08, F, 1	Specify whenever below grade excavation (core out) will NOT be measured and paid as extra work.
2010, 1.08, J, 3	Specify whenever removal of pipe and conduits will include capping.
2010, 1.08, L	Specify when the Contractor is responsible for compaction testing.
2010, 2.01	Specify use of compost-amended or off-site topsoil if on-site topsoil is NOT to be used.
2010, 2.02, C, 3	Specify the limits of Class 13 excavation.
2010, 2.04, C, 5	Specify whenever Type 2 geogrid is to be used in lieu of Type 1.
2010, 3.03, F, 1	Specify the desired depth for removal of unsuitable or unstable materials.
2010, 3.04, D	Specify whenever Type A compaction is to be used in lieu of compaction with moisture and density control.
2010, 3.05	Specify whenever and where unsuitable soils will be allowed in the right-of-way.
2010, 3.06, A	Specify if granular stabilization materials or subgrade treatment is to be used in lieu of select subgrade materials.

I. Items to be Specified on Plans or in Contract Documents (Continued)

2010, 3.07	Specify the type of subgrade treatment (lime, cement, fly ash, asphalt, geogrid, or geotextiles) to be used.
2010, 3.07, A, 1	Specify the depth and rate of incorporation of the subgrade treatment material (lime, cement, fly ash, or asphalt).
2010, 3.07, A, 2	Specify the areas requiring subgrade treatment.
2010, 3.08, B	Specify the type and depth of subbase.
2010, 3.09, A	Specify when the Contractor is responsible for compaction testing.
Figure 2010.102	Specify whenever Type A compaction is desired in lieu of compaction with moisture and density control.

Section 3010 - Trench Excavation and Backfill

3010, 1.08, F	Specify when the Contractor is responsible for trench compaction testing.
3010, 2.03, B	Specify whenever Class V material can be used as other than topsoil.
3010, 3.05, B, 1, a	Specify if granular bedding material is to be used for pressure pipes.
3010, 3.05, B, 3, a, 1)	Specify if concrete, flowable mortar, or CLSM is to be used in lieu of other bedding materials.
3010, 3.05, C, 3, a, 1)	Specify if concrete, flowable mortar, or CLSM is to be used in lieu of other bedding materials.
3010, 3.05, D, 4, a, 1)	Specify if concrete, flowable mortar, or CLSM is to be used in lieu of other bedding materials.
Figure 3010.101	Specify when over-excavation and foundation stone will be required.
Figure 3010.105	Specify when and where to install a waterstop.

Section 3020 - Trenchless Construction

3020, 2.02, A	Specify the wall thickness of casing pipe. See Section 9C-1.
3020, 2.02, C	Specify inside diameter of casing pipe.
3020, 2.05, B	Specify where special fill materials will be used.
3020, 3.04, A, 2, b	Specify the installation deviation tolerances of casing pipe if different than those included.
3020, 3.04, A, 2, b, 2), b)	Specify the minimum depth of pressurized pipe.
3020, 3.04, C, 8	Specify when to fill the annular space between the carrier and casing pipe with flowable mortar or CLSM.

I. Items to be Specified on Plans or in Contract Documents (Continued)

Section 4010 - Sanitary Sewers

4010, 1.08, E	Specify the distance beyond the right-of-way line that the sanitary sewer service stub is to extend, if other than 10 feet.
4010, 1.08, H, 3	For removal of sanitary sewer, specify if capping is required.
4010, 2.01, A, 1	For solid wall PVC pipe, 8 inch to 15 inch, specify if SDR 35 may be used.
4010, 2.01, C, 2, a	For corrugated PVC, 8 inch to 10 inch, specify if a minimum pipe stiffness of 46 psi may be used.
4010, 2.02, A	Specify when joint restraints for ductile iron pipe force mains are required.
4010, 2.02, B	Specify when restrained joints are required for PVC force mains.
4010, 2.02, E, 2	Specify the color of plastic post used for tracer wire station.
4010, 3.02, B, 7	Specify the location for installation of wye or tee service fitting.
4010, 3.05, B, 2	Specify the location for any installation of a tracer wire station in addition to each end of the force main.
4010, 3.06, A	Specify the locations for installation of sanitary sewer service stub.
4010, 3.06, C	Specify the distance beyond the right-of-way line that the sanitary sewer service stub is to extend, if other than 10 feet.
4010, 3.06, C, 3	Specify the depth of sanitary sewer service stub at its termination, if other than 10 to 12 feet.
4010, 3.06, C, 5	Specify method of marking the end of the sanitary sewer service line.
4010, 3.08, B, 2	Specify when to fill an abandoned sanitary sewer with flowable mortar or controlled low strength material (CLSM).
4010, 3.10	Specify where to provide sanitary sewer cleanouts.

Section 4020 - Storm Sewers

4020, 1.08, C, 3	Specify if capping is required for removal of storm sewer.
4020, 2.01, A, 3	Specify when to use a rubber O-ring or profile gasket in lieu of a tongue and groove joint wrapped with engineering fabric.
4020, 2.01, B, 3	Specify when to use a rubber O-ring or profile gasket in lieu of a tongue and groove joint wrapped with engineering fabric.

I. Items to be Specified on Plans or in Contract Documents (Continued)

4020, 2.01, C, 3	Specify when to use a rubber O-ring or profile gasket in lieu of a tongue and groove joint wrapped with engineering fabric.
4020, 2.01, G, 1, d	Specify gage of corrugated metal pipe, if other than Iowa DOT Standard Road Plan DR-104.
4020, 2.01, I, 2	Specify gage of coated corrugated metal pipe, if other than Iowa DOT Standard Road Plan DR-104.
4020, 3.04, B, 2	Specify the use of a rubber O-ring or profile gasket.
4020, 3.07, B, 2	Specify when to fill a line to be abandoned with flowable mortar or CLSM.

Section 4030 - Pipe Culverts

4030, 2.01, C, 5	Specify gage of the structural plate culverts, if other than Iowa DOT Standard Road Plan DR-104.
4030, 3.02, A	Specify the locations to install pipe aprons.
4030, 3.02, B	Specify the locations to install apron footings.
4030, 3.02, E	Specify the locations to install apron guards.
Figure 4030.225	Specify when to extend the bottom cross bar through the apron.

Section 4040 - Subdrains and Footing Drains

4040, 1.08, A, 3	Specify the use of engineering fabric.
4040, 1.08, E	Specify the distance beyond the right-of-way that the storm sewer service stub is to extend, if other than 10 feet.
4040, 3.01, A, 1	Excavate trench and provide pipe bedding and backfill as shown on the figures. Install engineering fabric if specified in the contract documents.
4040, 3.02, B	Specify the use of engineering fabric.
4040, 3.03, A	Specify the locations to install footing drain service stubs.
4040, 3.03, C	Specify the distance beyond the right-of-way that the footing drain service stub is to extend, if other than 10 feet.
Figure 4040.231	For Type 1 subdrains, specify Case A, B, or C. For Type 2 subdrains, specify Case D or E and the pipe diameter. When using Case A or Case D, specify the distance from back of curb. For both types, specify when engineering fabric is to be used.
Figure 4040.232	Specify the type of subdrain cleanout to be used.

I. Items to be Specified on Plans or in Contract Documents (Continued)

Figure 4040.233	Specify when to use a CMP outlet.

Section 4050 - Pipe Rehabilitation

4050, 1.07, C	Specify who will provide water for installation of cured-in-place pipe if not the owner.
4050, 2.01, A, 3	Specify the maximum outside diameter and SDR of polyethylene or polyolefin pipe for sliplining.
4050, 2.06, B, 1	Specify the nominal internal diameter and length of existing pipe.
4050, 2.06, B, 5	Specify the minimum SDR wall thickness for DRP-HDPE.
4050, 2.07, B, 1	Specify the nominal internal diameter and length of existing pipe.
4050, 2.07, B, 5	Specify the minimum SDR wall thickness for FFP-PVC pipe lining.
4050, 2.09, B	Specify materials to be used for pipe replacement (spot repairs).
4050, 3.08	Specify the installation process for DRP-HDPE or FFP-PVC, if other than manufacturer’s recommendations.
4050, 3.08, C, 1	Specify the material used to replace pipe of the same nominal size as the existing pipe.

Section 4060 - Cleaning, Inspection, and Testing of Sewers

4060, 2.01, B, 3	Specify the type of recording media that will be used to record the inspection.
4060, 3.03, A, 1	Specify whenever video inspection of storm sewers is <u>not</u> desired.

Section 5010 - Pipe and Fittings

5010, 1.08, C	Specify whether measurement of fittings will be made by count or by weight.
5010, 2.01, A, 1, b	Specify the minimum wall thickness for PVC pipe sizes over 24 inches.
5010, 2.01, A, 2	Specify joint type for PVC pipe if other than push-on.
5010, 2.01, B, 1, b	Specify the minimum wall thickness for DIP sizes over 24 inches.
5010, 2.01, B, 4	Specify joint type for DIP if other than push-on.
5010, 2.04, C	Specify when thrust blocks will be used for pipe sizes greater than 16 inches in diameter.
5010, 2.07, B	Specify the materials to use for water service pipe and appurtenances.

I. Items to be Specified on Plans or in Contract Documents (Continued)

5010, 3.01, A, 3	Specify the lines and grades to install pipe with fittings.
5010, 3.01, A, 8	For pipes larger than 16 inches, specify when concrete thrust blocks are required in addition to restrained joints.
5010, 3.06, E	Specify the locations to install ground rods if other than adjacent to connections to existing piping.
5010, 3.07, B	Specify where to construct utility line supports.
5010, 3.08	Specify when the change of piping material is to be on the inside of the structure wall.
Figure 5010.101	Specify when to use the alternate method of thrust blocks at dead ends.

Section 5020 - Valves, Fire Hydrants, and Appurtenances

5020, 2.01, A, 2	Specify whenever the opening direction for valves is clockwise.
5020, 2.01, D, 7	Specify the locations to use tapping valve assemblies.
5020, 2.02, B	Specify allowable manufacturer(s) of fire hydrant assemblies.
5020, 2.02, C, 5	Specify whenever the opening direction for fire hydrant assemblies is clockwise.
5020, 2.02, C, 6	For fire hydrant assemblies, specify the operating nut, pumper nozzle, nozzle threads, and main valve nominal opening sizes.
5020, 2.03, A	Specify the type of flushing device (blowoff) to be used.
5020, 2.03, B, 2	Specify the allowable manufacturer(s) for valve boxes.
5020, 3.02	Specify where to install and how to construct flushing device (blowoff).
5020, 3.04, D	Specify if exterior of a new fire hydrant barrel section will be painted a color other than matching the existing fire hydrant.

Section 6010 - Structures for Sanitary and Storm Sewers

6010, 2.05, B, 2, b	Specify the use of engineering fabric.
6010, 2.06, B	Specify when to use a concentric cone on sanitary sewer manholes.
6010, 2.11, B, 1	Specify if sanitary sewer manhole exterior is to be coated.
6010, 2.11, B, 2	Specify whenever sanitary sewer manhole lining is required.
6010, 2.13, A	Specify if steps are to be provided for structures other than circular, precast manholes. Specify if steps are NOT to be provided in circular, precast manholes.

I. Items to be Specified on Plans or in Contract Documents (Continued)

6010, 3.01, J	Specify the type of casting to use for manholes and intakes, except for intakes that have a specific casting type identified on the figures. Specify if casting frame is to be attached to the structure with bolts.
6010, 3.02, B, 2	Specify if reinforcing steel is to lap other than 36 diameters.
6010, 3.04, A, 1	Specify when to install casting extension rings.
6010, 3.04, B, 3	Specify when existing casting may be reinstalled for minor adjustment of existing manhole or intake.
6010, 3.04, C, 4	Specify when existing casting may be reinstalled for major adjustment of existing manhole or intake.
6010, 3.05, C, 1, a	Specify whenever a knockout opening is allowed in lieu of a cored opening.
6010, 3.05, C, 1, b	Specify if sanitary sewer service is NOT required to be maintained at all times when connecting a sanitary sewer to existing manhole or intake.
6010, 3.05, C, 3	Specify whenever a knockout opening is allowed in lieu of a cored opening.
6010, 3.06, A	Specify if removal of manhole or intake is other than to a minimum of 10 feet below top of subgrade in paved areas or 10 feet below finished grade in other areas.
6010, 3.06, B, 3	Specify when to fill abandoned pipe line with flowable mortar or controlled low strength material.
Figure 6010.501	Specify when Type Q grate is to be used in lieu of Type R.
Figure 6010.502	Specify when Type Q grate is to be used in lieu of Type R.
Figure 6010.603	Specify when Type Q grate is to be used in lieu of Type R.

Section 6020 - Rehabilitation of Existing Manholes

6020, 2.02, A	Specify the thickness of the in-situ manhole replacement wall.
6020, 2.02, C	Specify whenever the Contractor is required to provide a PVC or PE plastic liner for in-situ manhole replacement.
6020, 3.01, C	Specify when the use of a urethane chimney seal is allowed.
6020, 3.02, B, 3	Specify whenever a plastic liner is to be installed in an in-situ manhole replacement.

I. Items to be Specified on Plans or in Contract Documents (Continued)

Section 6030 - Cleaning, Inspection, and Testing of Structures

6030, 3.04, A, 1	Specify when exfiltration testing is required for new sanitary sewer manholes in lieu of vacuum testing.
6030, 3.04, C, 1	Specify when exfiltration testing is required for new sanitary sewer manholes in lieu of vacuum testing.

Section 7010 - Portland Cement Concrete Pavement

7010, 2.03, A, 2, a	Specify the type of coarse aggregate to be used (crushed limestone or gravel).
7010, 3.02, H, 5, a	Specify when a textured finished surface other than an artificial turf or burlap drag is desired (i.e. surface tining).
7010, 3.02, H, 5, b	Specify when surface tining is required. *Note - longitudinal tining is listed as the default.*
7010, 3.02, I, 1, a	Specify when the use of a linseed oil solution is required.
7010, 3.02, J, 1, a	Specify the type and locations for construction of joints.
7010, 3.02, J, 2, i	Specify when to use wet sawing for dust control.
7010, 3.02, J, 3, a	Specify the location of longitudinal and transverse construction joints.
7010, 3.02, J, 4, a	Specify the location of expansion joints.
7010, 3.04, A, 1, b, 1)	Specify the depth to scarify existing HMA surface to receive a PCC bonded overlay.
7010, 3.04, B, 1	Specify the location to trim high spots in the existing asphalt surface prior to constructing unbonded overlays.
7010, 3.04, B, 2, b	Specify when to place an HMA stress relief course over existing PCC pavement.
7010, 3.08, C, 2, a	Specify when the use of a profilograph for pavement smoothness is required.
Figure 7010.101, sheet 4	Specify when to use Detail D-1, D-2, or D-3.

Section 7020 - Hot Mix Asphalt Pavement

7020, 1.08, A & B	Specify if measurement of HMA pavement or overlay is by ton or square yard.
7020, 1.08, C & D	Specify if measurement of HMA base widening is by ton or square yard.

I. Items to be Specified on Plans or in Contract Documents (Continued)

7020, 3.05, B, 1	Specify when the use of profilograph for pavement smoothness is required.
7020, Table 7020.05	Specify if the field laboratory air voids target value is other than 4%.

Section 7030 - Sidewalks, Shared Use Paths, and Driveways

7030, 1.08, H, 2	Specify whether granular surfacing for driveways will be computed in square yards or tons.
7030, 1.08, I, 1	Specify whenever the Contractor will be responsible for concrete compression or HMA density testing.
7030, 2.03, A	Specify color and surface texture of clay brick pavers, or select from samples submitted by the Contractor.
7030, 2.03, B	If concrete brick pavers are to be used, specify the material requirements.
7030, 3.01, A-C	Specify removal limits of sidewalks, shared use paths, driveways, bricks, and curbs.
7030. 3.01, E	Specify the locations to grind or saw existing curbs to install sidewalks, shared use paths, and driveways.
7030, 3.04, D	Specify when curing is required.
7030, 3.04, F, 2, a, 1)	Specify the spacing for transverse joints in shared use paths, if other than equal to the width of the shared use paths.
7030, 3.06, A, 2	Specify the cross-section and patterns to use for brick sidewalks with a sand base.
7030, 3.06, B, 1, b	Specify the cross-section and patterns to use for brick sidewalks with a concrete base.
7030, 3.11, A	Specify when testing will be the Contractor's responsibility.
Figure 7030.101	Specify the radius for commercial and industrial driveways. Specify when a 'B' joint is to be provided at the back of curb. Specify the driveway width. Specify when a 5 foot sidewalk is to be constructed through the driveway.
Figure 7030.102	Specify the radius for commercial and industrial driveways. Specify the driveway width. Specify when a 5 foot sidewalk is to be constructed through the driveway.
Figure 7030.104	Specify parking grading slope and property slope if different than 4:1.
Figure 7030.201	If a special grade is required for parking slopes, specify the grade. Specify the width of the sidewalk.

I. Items to be Specified on Plans or in Contract Documents (Continued)

Figure 7030.202	Specify one of the curb details for Class A sidewalk.
Figure 7030.203	Specify the brick sidewalk pattern. Specify the jointing of the concrete base.
Figure 7030.205	Specify the use of a BT-3, KT-2, or expansion joint.
	Section 7040 - Pavement Rehabilitation
7040, 2.01, A, 1	Specify if patches are <u>not</u> constructed as standard patches.
7040, 2.01, A, 2	Specify the use of calcium chloride in high early strength patching.
7040, 2.01, B	Specify if an HMA mixture other than a minimum Low Traffic (LT) mixture is desired.
7040, 2.01, C, 5	Specify the use of soil sterilant for crack and joint filler material.
7040, 2.01, G	Specify if a subbase material other than modified subbase is desired.
7040, 3.01, C	Specify the dimensions of full depth and partial depth patches.
7040, 3.01, F	Specify seeding or sodding the area outside the pavement.
7040, 3.02, A, 1	Specify when a second saw cut is required.
7040, 3.02, C, 6	Specify the locations of joints.
7040, 3.03, A, 4	Specify if a vertical face is <u>not</u> desired.
7040, 3.04, J	Specify when pavement smoothness testing is required.
7040, 3.05, B	Specify the depth to mill the pavement area.
7040, 3.05, D	Specify if materials removed are <u>not</u> the property of the Contractor.
7040, 3.06, B, 3	Specify when to clean wet sawn joints.
7040, 3.06, C, 2	Specify the level to heat, handle, and apply joint filler material.
7040, 3.07, A, 3	Specify when to apply soil sterilant.
7040, 3.07, B, 2	For cracks wider than 1 inch, specify when to utilize additional methods to clean cracks of old crack filler.
7040, 3.07, C, 2	For cracks 1/4 inch to 1 inch in width, specify when to utilize additional methods to clean cracks of old crack filler.
Figure 7040.102	Specify the use of a ‘CD’ joint.
Figure 7040.105	Specify the use of filter fabric. Specify the type of subbase.

I. Items to be Specified on Plans or in Contract Documents (Continued)

Section 7050 - Asphalt Stabilization

7050, 1.02	Specify the crown of the pavement.
7050, 2.01, B	Specify the type of aggregate required.
7050, 3.03, A	Specify the depth of existing roadway surface to reclaim, if other than 4 inches.
7050, 3.07	Specify the type of surface treatment to apply.

Section 7060 - Bituminous Seal Coat

7060, 1.08 A & B	Specify measurement of bituminous seal coat is in area or units.
7060, 2.01, A	Specify the cover aggregate size.
7060, 2.01, B	Specify bituminous material if different than CRS-2P.
7060, 3.02, A, 1	Specify when to patch and joint fill hard surfaced streets.
7060, 3.04, B	Specify the application rate for spreading binder bitumen, if other than shown in the table.
7060, 3.04, D	Specify the application rate for spreading cover aggregate, if other than shown in the table.
7060, 3.06, B, 2	Specify the rate for spreading binder bitumen for two course seal coats.
7060, 3.06, B, 3	Specify the size of aggregate and the rate for spreading cover aggregate for two course seal coats.
7060, 3.07	Specify if sweeping of rural pavements is <u>not</u> necessary.

Section 7070 - Emulsified Asphalt Slurry Seal

7070, 1.02, B	Specify the application of fine or coarse slurry mixtures.
7070, 2.01, B	Specify when to use crushed aggregates.
7070, 2.02, A	Specify the amount of asphalt emulsion to blend with the aggregate.
7070, 3.01, B, 1, b	Specify the width of slurry mixture application.
7070, 3.02, A	Specify when to complete pavement patches and joint or crack filling for surface preparation.
7070, 3.02, C	Specify if water flushing for surface preparation is <u>not</u> allowed.

I. Items to be Specified on Plans or in Contract Documents (Continued)

7070, 3.03, C	Specify the rate of applying the slurry seal, if other than 10 to 18 pounds per square yard for fine aggregate and 15 to 22 pounds per square yard for coarse aggregate.
7070, 3.03, F	Specify when to apply a burlap drag.
7070, 3.05, E	Specify if strip slurry treatment is to be placed in two separate operations.

Section 7080 - Permeable Interlocking Pavers

7080, 2.02, A	Specify either slotted or perforated underdrain pipes.
7080, 2.02, B	Specify the size of collector pipe if other than 6 inch diameter is desired.
7080, 2.03, C	Specify the size of lateral pipe if other than 4 inch diameter is desired.
7080, 3.02, A	Specify the elevation and grade for the excavation area.
7080, 3.02, B	Specify the use and location of underdrains.
7080, 3.03, A	Specify the use of engineering fabric over completed subgrade.
7080, 3.04, A, 5	Specify cleanout locations.
7080, 3.04, A, 7	Specify the use of underdrain cleanout pipes and observation wells.
7080, 3.04, B, 1	Specify underdrain lateral pipe locations.
7080, 3.05, A	Specify the thickness of storage aggregate.
7080, 3.05, C	Specify the storage aggregate elevation.
7080, 3.09	Specify the installation pattern of the pavers.

Section 8010 - Traffic Control

8010, 2.01, A, 1, c	Specify if a message besides “TRAFFIC SIGNAL” will be required on the handhole cover.
8010, 2.01, B, 3, a, 2)	Specify solvent welded, socket type fittings for use other than PVC conduit and fittings.
8010, 2.01, C, 6, a	Specify the mode type, size, and number of fibers for fiber optic cable required.
8010, 2.01, C, 6, p	Specify the type of fiber distribution panel if a panel other than one capable of terminating a minimum of 24 fibers is desired.
8010, 2.01, C, 6, t	Specify the use of fusion splice continuous fiber runs or branch circuit connections in splice enclosures.

I. Items to be Specified on Plans or in Contract Documents (Continued)

8010, 2.02, B, 2, c	Specify the voice message to be used for accessible pedestrian signal push button stations.
8010, 2.02, D, 9	Specify the type of mounting for microwave vehicle detectors.
8010, 2.03, A	Specify the use of traffic monitoring systems.
8010, 2.03, B	Specify the use of fiber optic hub cabinet.
8010, 2.03, C, 2, b	Specify the location to mount the antenna for a wireless interconnect network, if other than near the top of the signal pole nearest the controller cabinet.
8010, 2.04, A, 2, b	Specify dimensions and type of aluminum cabinet riser to be used.
8010, 2.04, A, 2, g	Specify accommodations of phasing and expansibility of cabinet back panel positions.
8010, 2.04, C	Specify the use of emergency vehicle preemption system.
8010, 2.05, A, 1, a	Specify the color of vehicle traffic signal head assembly housing.
8010, 2.05, B, 1, a	Specify the color of pedestrian traffic signal head assembly housing.
8010, 2.05, C, 1, a	Specify the mast arm length and vertical pole height.
8010, 2.05, C, 1, f	Specify where to use a combination street lighting/signal pole. Specify if the luminaire arm is to be mounted somewhere other than the same vertical plane as the signal arm.
8010, 2.05, D, 1, a	Specify the vertical pole height of the traffic signal pedestal pole.
8010, 2.05, F, 3	Specify the street name sign dimensions, letter height and font, and sheeting.
8010, 3.01, B, 3, c	Specify if boring pits are allowed to be closer than 2 feet to the back of curb.
8010, 3.01, C, 9, c	Specify if the conduit cables could be pulled through intermediate junction boxes, handholes, pull boxes, pole bases, or any conduit opening.
8010, 3.01, C, 9, g	Specify how much cable slack to provide in each handhole, junction box, and cabinet.
8010, 3.01, C, 9, h	Specify installation of fiber optic accessories.
8010, 3.01, D, 1	Specify the foundation excavation size, shape, and depth.
8010, 3.02, C	Specify the installation of video detection camera system.

I. Items to be Specified on Plans or in Contract Documents (Continued)

8010, 3.03, A	Specify the installation of traffic monitoring system.
8010, 3.03, B	Specify the installation of fiber optic hub cabinet.
8010, 3.04, A, 1	Specify the installation of controller cabinet and auxiliary equipment.
8010, 3.04, B	Specify the installation of controller.
8010, 3.04, C	Specify the installation of UPS battery backup system.
8010, 3.04, D	Specify the installation of emergency vehicle preemption system.
8010, 3.06	Specify construction of temporary traffic signal.
Figure 8010.104	Specify the length of rectangular detector loop.
Figure 8010.105	Specify the number of signals, signs, and spacing.

Section 8020 - Pavement Markings

8020, 3.02, A, 3, c	Specify lane widths.
8020, 3.02, B, 2	Specify if pavement surface will not be cleaned with a rotary broom or street sweeper.
8020, 3.02, D	Specify if pavement is to be grooved prior to placing marking tape.
8020, 3.02, G, 2	Specify when to place pavement markings in a groove cut into the pavement surface.

Section 9010 - Seeding

9010, 2.01, B	Specify PLS, which shall <u>not</u> be less than the accumulated total.
9010, 2.02	Specify seed mixture in the contract documents.
9010, 2.03, A, 2	Specify if fertilizer is <u>not</u> to be applied for temporary conventional seeding.
9010, 3.01, A	Specify when aerial application of seed and fertilizer is desired.
9010, 3.01, M	Specify the use of a no-till attachment if desired.
9010, 3.04, E, 4, a	Specify if winter dormant seeding is required.
9010, 3.10, B	Specify when a warranty for seeding is required.

Section 9020 - Sodding

9020, 2.04	Specify when contractor is <u>not</u> to provide water and watering equipment.

I. Items to be Specified on Plans or in Contract Documents (Continued)

Section 9030 - Plant Material and Planting

9030, 1.03, E	Specify when the contractor is to submit a schedule of unit prices for each size and variety of tree, shrub, and ground cover plant.
9030, 2.01, A, 4	Specify whenever plants in rows do not need to be matched in form or size.
9030, 2.01, E, 1	Specify where to use bare root plants.
9030, 3.05	Specify when tree drainage wells are needed.
9030, 3.08, A	Specify when tree wrapping is required.
9030, 3.12, B	Specify when a warranty for plants is required.
Figure 9030.102	Specify when tree wrapping is required.

Section 9040 - Erosion and Sediment Control

9040, 1.08, A, 1	Specify if the Contractor will be responsible for the SWPPP preparation.
9040, 1.08, A, 2	Specify if the Contractor will be responsible for the SWPPP management.
9040, 1.08, B	Specify thickness for compost blankets.
9040, 1.08, E, 1	Specify the width of temporary RECP.
9040, 1.08, I	Specify if level spreaders are not to be removed.
9040, 1.08, L, 1, c	Specify the use of anti-seep collars.
9040, 1.08, O	Specify measurement for stabilized construction entrance in square yards or tons.
9040, 2.02, B	Specify the use of filter berms or compost blankets.
9040, 2.03	Specify the use of filter material in areas other than filter socks and filter berms.
9040, 2.06, A	Specify diameter for open weave, degradable netting if other than 9 inches is required.
9040, 2.07, A, 2	Specify if using RECP for permeable check dam.
9040, 2.08, A	Specify length of pressure-treated timber for level spreaders.
9040, 2.11, A	Specify class of concrete if not Class C.
9040, 2.11, B	Specify riser diameter for sediment basin outlet structures.

I. Items to be Specified on Plans or in Contract Documents (Continued)

9040, 2.11, C, 1	Specify the number, diameter, and elevation of the holes in the riser of the dewatering device in sediment basin outlet structures.
9040, 2.11, D	Specify barrel diameter of the sediment basin outlet structures.
9040, 2.11, E	Specify riser diameter for anti-vortex device.
9040, 3.02, D	Specify if weekly erosion and sediment control site inspections are not required as a part of SWPPP management.
9040, 3.05, B	Specify depth of compost blankets.
9040, 3.06, A	Specify when the filter berm is not to be installed along the contour.
9040, 3.06, C	Specify when a vegetated berm is required.
9040, 3.07, A, 1	Specify the size and length of filter sock.
9040, 3.07, A, 3	Specify when the filter sock is not to be installed along the contour.
9040, 3.07, B	Specify when to remove the filter sock.
9040, 3.08, A, 2	Specify if placement of seed and fertilizer is to be accomplished before installation of temporary rolled erosion control products.
9040, 3.08, A, 3	Specify if placement of seed and fertilizer is to be accomplished on the anchor trench.
9040, 3.08, B, 1	Specify if placement of seed and fertilizer is to be accomplished before installation of temporary rolled erosion control products.
9040, 3.09, B	Specify when to remove the wattle.
9040, 3.10, A, 2	Specify when to provide an RECP under the check dam.
9040, 3.10, D	Specify when to remove check dams.
9040, 3.12, C	Specify the excavated depth behind the level spreader.
9040, 3.12, E	Specify the minimum depth of depression before accumulated sediment is removed.
9040, 3.13	Specify the quantity of rip rap (revetment stone or erosion stone).
9040, 3.15, B, 1	Specify the number, diameter, and configuration of holes in the riser section of sediment basin outlet structures.
9040, 3.17	Specify the size and elevations of sediment traps.
9040, 3.18, A, 1	Specify when the silt fence material is not to be installed along the contour.

I. Items to be Specified on Plans or in Contract Documents (Continued)

9040, 3.19, E	Specify when to install subgrade stabilization fabric prior to placing crushed stone.
9040, 3.19, F	Specify the thickness and dimensions of crushed stone for stabilized construction entrance.
Figure 9040.101	Specify if compost blankets are vegetated or unvegetated.
Figure 9040.102	Specify size of berm if slope is steeper than 3:1. Specify berm placement locations in uncompacted windrow perpendicular to the slope. Specify filter sock diameter.
Figure 9040.105	Specify diameter of wattle. Specify space between wattles.
Figure 9040.107	Specify height between engineering fabric and crest on the rock check dam.
Figure 9040.108	Specify total height of diversion.
Figure 9040.109	Specify excavated depression depth.
Figure 9040.110	Specify the rock thickness (T), width (W), and length (L) for rip rap apron for pipe outlet onto flat ground.
Figure 9040.111	Specify the rock thickness (T), width (W), and length (L) for rip rap apron for pipe outlet into channel.
Figure 9040.112	Specify diameter of pipe for temporary pipe slope drain. Specify A, B, and C anchoring options.
Figure 9040.113	Specify barrel length and diameter for sediment basin without emergency spillway. Specify when anti-seep collars are required.
Figure 9040.114	Specify barrel length and diameter for sediment basin with emergency spillway. Specify when anti-seep collars are required.
Figure 9040.115	Specify elevations and dimensions for sediment basin dewatering device. Specify perforation configurations. Specify diameter of discharge pipe barrel.
Figure 9040.116	Specify riser diameter for anti-vortex device.
Figure 9040.117	Specify when anti-seep collars are required.
Figure 9040.118	Specify width of sediment trap.
Figure 9040.119	Specify spacing of post installation for silt fence.

Section 9050 - Gabions and Revet Mattresses

9050, 1.08, A, 3	Specify PVC coating for gabions.
9050, 1.08, B, 3	Specify PVC coating for revet mattresses.

I. Items to be Specified on Plans or in Contract Documents (Continued)

9050, 2.01	Specify when double twisted wire baskets are <u>not</u> required.
9050, 2.02	Specify when to use welded wire baskets.
9050, 2.05	Specify when to use anchor stakes. Specify the length of anchor stakes.
9050, 3.01, A	Specify when to cut and reshape the area behind a proposed gabion wall to allow for placement of the wall.
9050, 3.01, E	Specify the placement, compaction, and dimensions of granular subbase materials.
9050, 3.04, A	Specify special details of gabion wall installation including height, slope of wall, gabion setback, special backfill materials, and tieback requirements.

Section 9060 - Chain Link Fence

9060, 1.08, A, 3	Specify PVC coating for chain link fence.
9060, 1.08, B, 3	Specify the use of barbed wire for gates.
9060, 1.08, C, 3	Specify the type of barbed wire supporting arm.
9060, 2.01, D, 2	Specify the PVC coating color.
9060, 2.02, A, 2	Specify the nominal diameter of fence height for post use, if other than shown in the table.
9060, 2.05, A	Specify the type of arm configuration for barbed wire supporting arms.
9060, 2.07, A	Specify the type, height, and width of gates.
9060, 3.01, A	Specify fence location and height.
9060, 3.01, B, 2, a	Specify post holes dimensions.
9060, 3.01, B, 2, e	Specify the required brace-post assembly.
9060, 3.01, G	Specify when to use barbed wire.
9060, 3.01, G, 1	Specify the installation of barbed wire, if other than 3 parallel wires on each barbed wire supporting arm on the outside of the area being secured.
9060, 3.01, H	Specify the installation requirements for gates.
9060, 3.01, I, 1	Specify the installation of electrical grounds.
9060, 3.02	Specify when all fences, including posts and footings, are <u>not</u> to be removed from within work areas.

I. Items to be Specified on Plans or in Contract Documents (Continued)

9060, 3.03, A	Specify the height of temporary fence.
Figure 9060.101	Specify the fence fabric width. Specify when to install fence on the roadway side of the right-of-way.
Figure 9060.103	Specify the length of the sidewalk.

Section 9070 - Landscape Retaining Walls

9070, 2.01, B	Specify the depth of limestone slabs, if other than 8 inches.
9070, 3.01, B	Specify the excavation line and grade.

Section 9071 - Segmental Block Retaining Walls

9071, 3.01, B	Specify the excavation line and grade.
9071, 3.02, B	Specify leveling pad materials.
9071, 3.02, C	Specify the elevation and orientation.
9071, 3.02, D, 1	Specify the use of subdrains.

Section 9072 - Combined Concrete Sidewalk and Retaining Wall

9072, 2.01, A, 3	Specify the type of expansion joint, if resilient filler is <u>not</u> desired.
9072, 3.01, B	Specify the excavation line and grade.
9072, 3.04	Specify the formation of rustications.

Section 9080 - Concrete Steps, Handrails, and Safety Rail

9080, 2.04, B	Specify when to galvanize handrail and safety rail.
9080, 2.04, C	Specify when to apply powder coat to steel, galvanized steel, or aluminum handrail and safety rail.
9080, 3.02, A, 1	Specify the length of rail.
Figure 9080.103	Specify the field painting of safety rail.

Section 10,010 - Demolition

10,010, 1.07, A	Specify when the use of explosives is allowed.
10,010, 3.08, D	Specify when the removal and disposal of all brush, shrubs, trees, logs, downed timber, and other yard waste on the site is <u>not</u> desired.
10,010, 3.08, E	Specify when the removal of all retaining walls is <u>not</u> desired.

I. Items to be Specified on Plans or in Contract Documents (Continued)

10,010, 3.11	Specify what materials are required to be recycled from the demolition site.

Section 11,010 - Construction Survey

11,010, 1.01, I	Specify any additional items to be included in construction survey work.
11,010, 3.02, D	Specify if property limits are to be marked.
11,010, 3.04	Specify which land corners, property corners, permanent reference markers, and benchmarks are to be replaced.

Section 11,040 - Temporary Sidewalk Access

11,040, 3.02, A	Specify locations to construct temporary granular sidewalks.
11,040, 3.03, B	Specify locations to locate temporary longitudinal channelizing devices.
Figure 11,040.102	Specify when to install orange construction safety fence between the top of the bottom rail and the bottom of the top rail.

Section 11,050 - Concrete Washout

11,050, 3.02, A	Specify locations of temporary granular sidewalks.

J. Incidental or Included Items

Items that are necessary to properly complete construction, including work and materials, and are not pay items. The following is a list of items in the SUDAS Standard Specifications that are considered incidental to other work unless specified as a pay item on the plans or in the contract documents. Please note - this list is not all-inclusive.

Section 2010 - Earthwork, Subgrade, and Subbase

2010, 1.08, A, 3	<u>Clearing and Grubbing (by units)</u> Placement of backfill in area where roots have been removed, and removal and disposal of all materials.
2010, 1.08, B, 3	<u>Clearing and Grubbing (by area)</u> Removal and disposal of all materials and placement of backfill in area where roots have been removed.
2010, 1.08, D, 2, c	<u>Topsoil, Compost-amended</u> Furnishing and incorporating compost.
2010, 1.08, E, 3	<u>Excavation, Class 10, Class 12, or Class 13</u> a. Site preparation for, and the construction of, embankment, fills, shoulder backfill, and backfill behind curbs. b. Overhaul. c. Finishing the soil surface, including roadways, shoulders, behind curbs, side ditches, slopes, and borrow pits. d. Repair or replacement of any fences that have been unnecessarily damaged or removed. e. Compaction testing, as specified in the contract documents.
2010, 1.08, F, 3	<u>Below Grade Excavation (Core Out)</u> Equipment, tools, labor, disposal of unsuitable materials, dewatering, drying, furnishing, and placement of foundation materials as required by the Engineer, compaction and finishing of the excavated area, and all incidental work as may be required.
2010, 1.08, G, 3	<u>Subgrade Preparation</u> Excavating, manipulating, replacing, compacting, and trimming to the proper grade.
2010, 1.08, H, 3	<u>Subgrade Treatment</u> Furnishing, placing, and incorporating the subgrade treatment material (cement, asphalt, fly ash, lime, geogrid, or geotextiles).
2010, 1.08, I, 3	<u>Subbase</u> Furnishing, placing, compacting, and trimming to the proper grade.
2010, 1.08, J, 1, c	<u>Removal of Structures</u> Removal and disposal of structures.
2010, 1.08, J, 2, a, 3)	<u>Removal of Known Box Culverts</u> Removal and disposal of known box culverts.

J. Incidental or Included Items (Continued)

2010, 1.08, J, 2, c, 3)	Removal of Known Pipe Culverts Removal and disposal of known pipe culverts.
2010, 1.08, J, 3, a, 3)	Removal of Known Pipes and Conduits Removal, disposal, and plugging, if specified, of pipes and conduits.

Section 3010 - Trench Excavation and Backfill

3010, 1.08, A	General 1. Standard trench excavation. 2. Removal and disposal of unsuitable backfill material encountered during standard trench excavation. 3. Removal of abandoned private utilities encountered during trench excavation. 4. Furnishing and placing granular bedding material. 5. Placing and compacting backfill material. 6. Dewatering. 7. Sheeting, shoring, and bracing. 8. Adjusting the moisture content of excavated backfill material to the range specified for placement and compaction.
3010, 1.08, C, 3	Trench Foundation Removal and disposal of over-excavated material required to stabilize trench foundation; and furnishing, hauling, and placing stabilization material.
3010, 1.08, D, 3	Replacement of Unsuitable Backfill Material Furnishing, hauling, and placing backfill material.
3010, 1.08, E, 3	Special Pipe Embedment or Encasement Furnishing and placing all required special pipe embedment or encasement materials.

Section 3020 - Trenchless Construction

3020, 1.08	All items of work contained in this section are incidental to the underground utility pipe being installed and will not be paid for separately.

Section 4010 - Sanitary Sewers

4010, 1.08, A, 1, c	Sanitary Sewer Gravity Main, Trenched Trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, wyes and other fittings, pipe joints, pipe connections, testing, and inspection.
4010, 1.08, A, 2, c	Sanitary Sewer Gravity Main, Trenchless Furnishing and installing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; pipe connections; testing; and inspection.

J. Incidental or Included Items (Continued)

4010, 1.08, B, 1, c	Sanitary Sewer Gravity Main with Casing Pipe, Trenched Furnishing and installing both carrier pipe and casing pipe, trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, furnishing and installing annular space fill material, casing spacers, pipe connections, testing, and inspection.
4010, 1.08, B, 2, c	Sanitary Sewer Gravity Main with Casing Pipe, Trenchless Furnishing and installing both carrier pipe and casing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; casing spacers; furnishing and installing annular space fill material; pipe connections; testing; and inspection.
4010, 1.08, C, 1, c	Sanitary Sewer Force Main, Trenched Trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, wyes and other fittings, pipe joints, testing, and inspection.
4010, 1.08, C, 2, c	Sanitary Sewer Force Main, Trenchless Furnishing and installing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; pipe connections; testing; and inspection.
4010, 1.08, D, 1, c	Sanitary Sewer Force Main with Casing Pipe, Trenched Furnishing and installing both carrier pipe and casing pipe, trench excavation, dewatering, placing bedding and backfill material, furnishing and installing annular space fill material, casing spacers, pipe connections, testing, and inspection.
4010, 1.08, D, 2, c	Sanitary Sewer Force Main with Casing Pipe, Trenchless Furnishing and installing both carrier pipe and casing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; casing spacers; furnishing and installing annular space fill material; pipe connections; testing; and inspection.
4010, 1.08, E, 3	Sanitary Sewer Service Stub Trench excavation, furnishing bedding material, placing bedding and backfill material, tap, fittings, testing, and inspection.
4010, 1.08, F, 3	Sanitary Sewer Service Relocation Removal of existing pipe, trench excavation, furnishing new pipe and bedding material, placing bedding and backfill material, connection back to existing service, compaction, testing, and inspection.
4010, 1.08, G, 3	Sewage Air Release Valve and Pit Excavation, furnishing bedding material, placing bedding and backfill material, compaction, and testing.
4010, 1.08, H, 3	Removal of Sanitary Sewer Removal, disposal, and capping (if specified) of pipe.

J. Incidental or Included Items (Continued)

4010, 1.08, I, 3	Sanitary Sewer Cleanout Plug at the end of the main, fittings, riser pipe, cap with screw plug, casting, and concrete casting encasement.
4010, 1.08, K, 1	Plugging sanitary sewers is incidental to other work and will not be paid for separately.

Section 4020 - Storm Sewers

4020, 1.08, A, 1, c	Storm Sewer, Trenched Trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, joint wrapping, wyes and other fittings, pipe joints, pipe connections, testing, and inspection. The length of elbows and tees of the pipes installed will be included in the length of pipe measured.
4020, 1.08, A, 2, c	Storm Sewer, Trenchless Furnishing and installing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; pipe connections; testing; and inspection.
4020, 1.08, B, 1, c	Storm Sewer with Casing Pipe, Trenched Furnishing and installing both carrier pipe and casing pipe, trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, furnishing and installing annular space fill material, casing spacers, pipe connections, testing, and inspection.
4020, 1.08, B, 2, c	Storm Sewer with Casing Pipe, Trenchless Furnishing and installing both carrier pipe and casing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; casing spacers; furnishing and installing annular space fill material; pipe connections; testing; and inspection.
4020, 1.08, C, 3	Removal of Storm Sewer Removal, disposal, and capping (if specified) of pipe.
4020, 1.08, E, 1	Plugging storm sewers is incidental to other work and will not be paid for separately.

Section 4030 - Pipe Culverts

4030, 1.08, A, 1, c	Pipe Culvert, Trenched Trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, connectors, testing, and inspection. The length of elbows and tees of the pipes installed will be included in the length of pipe measured.
4030, 1.08, A, 2, c	Pipe Culvert, Trenchless Furnishing and installing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill materials; pipe connections; testing; and inspection.

J. Incidental or Included Items (Continued)

4030, 1.08, B, 3	Pipe Apron Trench excavation, furnishing bedding material, placing bedding and backfill material, connectors, and other appurtenances.
4030, 1.08, C, 3	Footings for Concrete Pipe Aprons Excavation, reinforcing steel, and concrete.

Section 4040 - Subdrains and Footing Drain Collectors

4040, 1.08, A, 3	Subdrain Trench excavation, furnishing and placing bedding and backfill material, engineering fabric (when specified), connectors, and elbows and tees. The length of elbows and tees of the pipes installed will be included in the length of pipe measured.
4040, 1.08, B, 3	Footing Drain Collector Trench excavation, pipe, wyes, tap, fittings, and furnishing and placing bedding and backfill material.
4040, 1.08, D, 3	Subdrain or Footing Drain Outlets and Connections Pipe, non-shrink grout, coupling bands, and rodent guards for pipes 6 inches or smaller.
4040, 1.08, E, 3	Storm Sewer Service Stub Trench excavation, furnishing bedding material, placing bedding and backfill material, tap, fittings, and plugs.

Section 4050 - Pipe Rehabilitation

4050, 1.08, A, 3	Pipe Lining Removal of internal obstructions, pipe cleaning, inspection, and all costs associated with the public information and notification program.
4050, 1.08, B, 3	Building Sanitary Sewer Service Reconnection Removal of internal obstructions, pipe cleaning, and all costs associated with the public information and notification program.
4050, 1.08, C, 1, c	Spot Repairs (by Pipe Replacement) Uncovering and removing existing pipe, placing backfill material for replacement pipe, and restoring the surface.
4050, 1.08, C, 2, c	Spot Repairs (by Linear Foot) Furnishing and installing replacement pipe and connections.

4060 - Cleaning, Inspection, and Testing of Sewers

4060, 1.08	Cleaning, inspecting, and testing sanitary sewers, storm sewers, pipe culverts, and rehabilitated pipes (including video inspection) are incidental to other project costs and will not be paid for separately.

J. Incidental or Included Items (Continued)

Section 5010 - Pipe and Fittings

5010, 1.08, A, 1, c	Water Main, Trenched Trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, tracer system, testing, disinfection, and polyethylene wrap for ductile iron pipe and for fittings.
5010, 1.08, A, 2, c	Water Main, Trenchless Furnishing and installing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; tracer system; testing; and disinfection.
5010, 1.08, B, 1, c	Water Main with Casing Pipe, Trenched Furnishing and installing both carrier pipe and casing pipe, trench excavation, dewatering, furnishing bedding material, placing bedding and backfill material, casing spacers, furnishing and installing annular space fill material, tracer system, testing, and disinfection.
5010, 1.08, B, 2, c	Water Main with Casing Pipe, Trenchless Furnishing and installing both carrier pipe and casing pipe; trenchless installation materials and equipment; pit excavation, dewatering, and placing backfill material; casing spacers; furnishing and installing annular space fill material; tracer system; testing; and disinfection.
5010, 1.08, C, 1, c	Fitting (by count) Restrained joints and thrust blocks.
5010, 1.08, C, 2, c	Fitting (by weight) Restrained joints and thrust blocks.
5010, 1.08, D, 3	Water Service Stub (by each) Water service corporation, service pipe, curb stop, stop box, trench excavation, dewatering, furnishing bedding material, installation of tracer wire system for non-metallic service pipe, and placing bedding and backfill material.
5010, 1.08, E, 1, c	Water Service Stub (by length), Water Service Pipe Trench excavation, dewatering, furnishing bedding material, installation of tracer wire system for non-metallic service pipe, and placing bedding and backfill material.

Section 5020 - Valves, Fire Hydrants, and Appurtenances

5020, 1.08, A, 3	Valve (Butterfly or Gate) All components attached to the valve or required for its complete installation, including underground or above ground operator, square valve operating nut, valve box and cover, valve box extension, and valve stem extension.
5020, 1.08, B, 3	Tapping Valve Assembly Tapping sleeve, tapping valve, the tap, valve box and cover, valve box extension, and valve stem extension.

J. Incidental or Included Items (Continued)

5020, 1.08, C, 3	Fire Hydrant Assembly The fire hydrant, barrel extensions sufficient to achieve proper bury depth of anchoring pipe and height of fire hydrant above finished grade, and components to connect the fire hydrant to the water main, including anchoring pipe, fittings, thrust blocks, pea gravel or porous backfill material, and fire hydrant gate valve and appurtenances, except tapping valve assembly if used.
5020, 1.08, E	Measurement and payment for minor adjustment of an existing valve box by raising or lowering the adjustable valve box is incidental.
5020, 1.08, G, 3	Valve Box Replacement Removal of existing valve box; excavation; furnishing and installing new valve box; backfill; compaction; and all other necessary appurtenances.
5020, 1.08, H, 3	Fire Hydrant Adjustment Removal and reinstallation of the existing fire hydrant; furnishing and installing the extension barrel section and stem; and all other necessary appurtenances.

Section 5030 - Testing and Disinfection

5030, 1.08	Testing and disinfection of water systems is incidental to the construction of pipe and fittings.

Section 6010 - Structures for Sanitary and Storm Sewers

6010, 1.08, A, 3	Manhole Excavation, furnishing bedding material, placing bedding and backfill material, compaction, base, structural concrete, reinforcing steel, precast units (if used), infiltration barriers (sanitary sewer manholes only), castings, and adjustment rings.
6010, 1.08, B, 3	Intake Excavation, furnishing bedding material, placing bedding and backfill material, compaction, base, structural concrete, reinforcing steel, precast units (if used), inverts, pipe connections, castings, and adjustment rings.
6010, 1.08, C, 3	Drop Connection The connection to the manhole and all pipe, fittings, concrete encasement, and bedding and backfill material.
6010, 1.08, E, 3	Manhole or Intake Adjustment, Minor Removing existing casting and existing adjustment rings, furnishing and installing adjustment rings, furnishing and installing new casting, and installing new infiltration barrier (sanitary sewer manholes only).
6010, 1.08, F, 3	Manhole or Intake Adjustment, Major Removal of existing casting, adjustment rings, top sections, and risers; excavation; concrete and reinforcing steel or precast sections; furnishing and installing new casting; installing new infiltration barrier (sanitary sewer manholes only); placing backfill material; and compaction.

J. Incidental or Included Items (Continued)

6010, 1.08, G, 3	Connection to Existing Manhole or Intake Coring or cutting into the existing manhole or intake, pipe connectors, grout, and waterstop (when required).
6010, 1.08, H, 3	Remove Manhole or Intake Removal of casting, concrete, and reinforcement; plugging pipes; filling remaining structure with flowable mortar; and placing compacted fill over structure to finished grade.

Section 6020 - Rehabilitation of Existing Manholes

6020, 1.08, A, 1, c	Infiltration Barrier, Rubber Chimney Seal All necessary compression or expansion bands and extension sleeves as necessary to complete chimney seal.
6020, 1.08, A, 2, c	Infiltration Barrier, Molded Shield Sealant.
6020, 1.08, B, 3	In-situ Manhole Replacement, Cast-in-place Concrete Handling of sewer flows as required to properly complete the installation, invert overlay as recommended by the manufacturer, replacement of existing casting with a new casting, and testing the manhole upon completion.
6020, 1.08, C, 3	In-situ Manhole Replacement, Cast-in-place Concrete with Plastic Liner Handling of sewer flows as required to properly complete the installation, invert overlay as recommended by the manufacturer, replacement of existing casting with a new casting, sealing at the frame and cover, sealing pipe penetrations as recommended by the manufacturer, and testing the manhole upon completion.
6020, 1.08, D, 3	Manhole Lining with Centrifugally Cast Cementitious Mortar Liner with Epoxy Seal Handling of sewer flows during lining operations as required to properly complete the installation, and replacement of the existing casting with a new casting.

Section 6030 - Cleaning, Inspection, and Testing of Structures

6030, 1.08	Cleaning, inspection, and testing of structures are incidental to construction of structures and will not be paid for separately.

Section 7010 - Portland Cement Concrete Pavement

7010, 1.08, A, 3	Pavement, PCC Final trimming of subgrade or subbase, integral curb, bars and reinforcement, joints and sealing, surface curing and pavement protection, safety fencing, concrete for rigid headers, boxouts for fixtures, and pavement smoothness testing.

J. Incidental or Included Items (Continued)

7010, 1.08, E, 3	Curb and Gutter Final subgrade/subbase preparation, bars and reinforcement, joints and sealing, surface curing and pavement protection, and boxouts for fixtures.
7010, 1.08, F, 3	Beam Curb Final subgrade/subbase preparation, bars and reinforcement, joints and sealing, surface curing and pavement protection, and boxouts for fixtures.
7010, 1.08, G, 3	Concrete Median Final subgrade/subbase preparation, bars and reinforcement, joints and sealing, surface curing and pavement protection, and boxouts for fixtures.
7010, 1.08, I, 3	PCC Pavement Samples and Testing Certified plant inspection, pavement thickness cores, profilograph pavement smoothness measurement (when required by the contract documents), and maturity testing.
7010, 1.08, K, 3	PCC Pavement Widening Final subgrade/subbase preparation, integral curb, bars and reinforcement, joints and sealing, surface curing and pavement protection, safety fencing, concrete for rigid headers, boxouts for fixtures, and pavement smoothness.
7010, 1.08, L, 1, c	PCC Overlay, Furnish Only Furnishing the concrete mixture and delivery to the project site.
7010, 1.08, L, 2, c	PCC Overlay, Place Only Integral curb, bars and reinforcement, joints and sealing, surface curing and pavement protection, safety fencing, concrete for rigid headers, boxouts for fixtures, and pavement smoothness testing.
7010, 1.08, L, 3, c	Surface Preparation for Bonded PCC Overlay Sandblasting, shot blasting, scarification, and surface cleaning.
7010, 1.08, L, 4, c	Surface Preparation for Unbonded PCC Overlay Scarification and surface cleaning.
7010, 1.08, L, 5, c	HMA Stress Relief Course for Unbonded PCC Overlay HMA mix, including binder, and placement.

Section 7020 - Hot Mix Asphalt Pavement

7020, 1.08, A, 3	Pavement or Overlay, HMA (by ton) Asphalt mix with asphalt binder, tack coats between layers, construction zone protection, and quality control.

J. Incidental or Included Items (Continued)

7020, 1.08, B, 3	Pavement or Overlay, HMA (by square yard) Asphalt mix with asphalt binder, tack coats between layers, construction zone protection, and quality control.
7020, 1.08, C, 3	HMA Base Widening (by ton) Asphalt mix with asphalt binder, tack coats between layers, construction zone protection, and quality control.
7020, 1.08, D, 3	HMA Base Widening (by square yard) Asphalt mix with asphalt binder, tack coats between layers, construction zone protection, and quality control.
7020, 1.08, H, 3	HMA Pavement Samples and Testing Certified plant inspection, pavement thickness cores, density analysis, profilograph pavement smoothness measurement (when required by the contract documents), and air void testing.

Section 7030 - Sidewalks, Shared Use Paths, and Driveways

7030, 1.08, A, 3	Removal of Sidewalk, Shared Use Path, or Driveway Sawing, hauling, and disposal of materials removed.
7030, 1.08, B, 3	Removal of Curb Hauling and disposal of materials removed.
7030, 1.08, C, 3	Shared Use Paths Subgrade preparation, jointing, sampling, smoothness testing and correction, and testing.
7030, 1.08, D, 3	Special Subgrade Preparation for Shared Use Paths Water required to bring subgrade moisture content to within the required limits.
7030, 1.08, E, 3	Sidewalk, PCC Minor grade adjustments at driveways and other intersections, subgrade preparation, formwork, additional thickness at thickened edges, jointing, sampling, smoothness testing and correction, and testing.
7030, 1.08, F, 1, c	Brick Sidewalk with Sand Base Subgrade preparation, brick edge restraints, furnishing and placing compacted sand base, and sand/cement joint filler.
7030, 1.08, F, 2, c	Brick Sidewalk with Concrete Base Subgrade preparation, concrete base, HMA setting bed, neoprene asphalt adhesive for asphalt setting bed, and sand/cement joint filler.
7030, 1.08, G, 3	Detectable Warning Steel bar supports and manufactured detectable warning panels.
7030, 1.08, H, 1, c	Driveway, Paved Excavation, subgrade preparation, jointing, sampling, and testing.

J. Incidental or Included Items (Continued)

7030, 1.08, H, 2, c	Driveway, Granular Excavation and preparation of subgrade.

Section 7040 - Pavement Rehabilitation

7040, 1.08, A, 3	Full Depth Patches Sawing, removing, and disposing of existing pavement and reinforcing; restoring the subgrade; furnishing and installing tie bars and dowel bars; furnishing and placing the patch material, including the asphalt binder and tack coat; forming and constructing integral curb; surface curing and pavement protection; joint sawing and filling; and placing backfill and restoring disturbed surfaces.
7040, 1.08, B, 3	Subbase Over-excavation Removal of existing subbase or subgrade, disposal of materials removed, furnishing and placing subbase material, and any additional excavation required for subbase placement.
7040, 1.08, C, 3	Partial Depth Patches Sawing, removing, and disposing of existing pavement; furnishing tack coat or bonding agent; furnishing and placing the patch material; curing; joint filling (PCC patches only); placing backfill; and restoring disturbed surfaces.
7040, 1.08, D, 3	Crack and Joint Cleaning and Filling, Hot Pour Furnishing crack and joint filler material and routing, sawing, cleaning, and filling joints or cracks.
7040, 1.08, E, 1, c	Crack Cleaning and Filling, Emulsion Furnishing emulsified crack filler material, cleaning cracks, placing soil sterilant, and filling cracks.
7040, 1.08, E, 2, c	Hot Mix Asphalt for Crack Filling Cleaning, applying tack coat, and furnishing and placing HMA for crack filling.
7040, 1.08, F, 3	Diamond Grinding Diamond grinding pavement, testing for smoothness according to the contract documents, and removal of slurry and residue from the project site.
7040, 1.08, G, 3	Milling Milling pavement; furnishing water; and salvaging, stockpiling, and removing cuttings and debris.
7040, 1.08, H, 3	Pavement Removal Sawing, breaking, removing, and disposing of existing pavement and reinforcing steel.
7040, 1.08, I, 3	Curb and Gutter Removal Sawing, breaking, removing, and disposing of existing curb and gutter.

J. Incidental or Included Items (Continued)

7040, 1.08, J	Required sampling and testing for pavement repair and rehabilitation work is incidental to other project costs and will not be paid for separately.

Section 7050 - Asphalt Stabilization

7050, 1.08, A, 3	Asphalt Stabilization Furnishing and spreading imported material, applying and incorporating asphalt stabilization, blending of the materials, grading and compacting the blended materials, and final clean up.

Section 7060 - Bituminous Seal Coat

7060, 1.08, A, 3	Bituminous Seal Coat (by area) Surface preparation including protection of street fixtures; furnishing and placing of materials, including fillets at intersecting streets, driveways, and turnouts; and final clean up.
7060, 1.08, B, 1, c	Bituminous Seal Coat (by units), Cover Aggregate Surface preparation including protection of street fixtures; furnishing and placing of materials, including fillets at intersecting streets, driveways, and turnouts; and final clean up.
7060, 1.08, B, 2, c	Bituminous Seal Coat (by units), Binder Bitumen Furnishing and placing of materials, including fillets at intersecting streets, driveways, and turnouts; and final clean up.

Section 7070 - Emulsified Asphalt Slurry Seal

7070, 1.08, A, 3	Emulsified Asphalt Slurry Seal (by area) Surface preparation and furnishing and placing of materials, including fillets at intersecting streets, driveways, and turnouts.
7070, 1.08, B, 1, c	Emulsified Asphalt Slurry Seal (by units), Aggregate Surface preparation and furnishing and placing of materials, including fillets at intersecting streets, driveways, and turnouts.
7070, 1.08, B, 2, c	Emulsified Asphalt Slurry Seal (by units), Asphalt Emulsion Surface preparation and furnishing and placing of materials, including fillets at intersecting streets, driveways, and turnouts.

Section 7080 - Permeable Interlocking Pavers

7080, 1.08, B, 3	Engineering Fabric Placing and securing filter fabric and any overlapped areas.
7080, 1.08, C, 3	Underdrain Furnishing and placing pipe, cleanouts, observation wells, and pipe fittings.

J. Incidental or Included Items (Continued)

7080, 1.08, D, 3	Storage Aggregate Furnishing, hauling, placing, and compacting storage aggregate.
7080, 1.08, E, 3	Filter Aggregate Furnishing, hauling, placing filter, and compacting aggregate.
7080, 1.08, F, 3	Permeable Interlocking Pavers Testing, placement of bedding course, installing permeable interlocking pavers, placing joint/opening fill material, refilling joint after 6 months, and pavement protection.
7080, 1.08, G, 3	PCC Edge Restraint Final trimming of subgrade or subbase, bars and reinforcement, joints and sealing, surface curing and pavement protection, safety fencing, and boxouts for fixtures.

Section 8020 - Pavement Markings

8020, 1.08, B, 3	Painted Pavement Markings, Solvent/Waterborne Reflectorizing spheres, layout, surface preparation, and application of marking paint.
8020, 1.08, C, 3	Painted Pavement Markings, Durable Layout, surface preparation, and application of marking paint.
8020, 1.08, D, 3	Painted Pavement Markings, High-Build Layout, surface preparation, and application of marking paint.
8020, 1.08, E, 3	Permanent Tape Markings Layout, surface preparation, and application of marking tape.
8020, 1.08, F, 3	Wet, Retroreflective Removable Tape Markings Layout, surface preparation, application, and removal.
8020, 1.08, G, 3	Painted Symbols and Legends Layout, surface preparation, and application of each symbol and legend.
8020, 1.08, H, 3	Precut Symbols and Legends Layout, surface preparation, and application of each symbol and legend.
8020, 1.08, I, 3	Temporary Delineators Installation and removal of delineators.
8020, 1.08, J, 3	Raised Pavement Markers Installation and removal of pavement markers.
8020, 1.08, K, 3	Pavement Markings Removed Pavement marking removal and waste material collection, removal, and disposal.

J. Incidental or Included Items (Continued)

8020, 1.08, L, 3	Symbols and Legends Removed Symbol and legend marking removal and waste material collection, removal, and disposal.
8020, 1.08, M, 3	Grooves Cut for Pavement Markings Layout, cutting grooves, collection and disposal of removed material, and additional groove width and transition length beyond the pavement marking dimensions.
8020, 1.08, N, 3	Grooves Cut for Symbols and Legends Layout, cutting grooves, and collection and disposal of removed material.

Section 9010 - Seeding

9010, 1.08, A, 1, c	Conventional Seeding, Seeding Removal of rock and other debris from the area; repairing rills and washes; preparing the seedbed; furnishing and placing seed, including any treatment required; furnishing and placing fertilizer and mulch; and furnishing water and other care during the care period, unless these items are bid separately.
9010, 1.08, B, 3	Hydraulic Seeding , Seeding, Fertilizing, and Mulching Removal of rock and other debris from the area; repairing rills and washes; preparing the seedbed; furnishing and placing seed, including any treatment required; furnishing and placing fertilizer and mulch; and furnishing water and other care during the care period, unless these items are bid separately.
9010, 1.08, C, 3	Pneumatic Seeding , Seeding, Fertilizing, and Mulching Removal of rock and other debris from the area; repairing rills and washes; preparing the seedbed; furnishing and placing seed, including any treatment required; furnishing and placing fertilizer and mulch; and furnishing water and other care during the care period, unless these items are bid separately.
9010, 1.08, E, 3	Warranty All work required to correct any defects in the original placement of the seeding for the period of time designated.

Section 9020 - Sodding

9020, 1.08, A, 3	Sod Preparation of sod and sodbed, stakes, fertilizing, watering, maintenance, and clean-up. Also includes any necessary sod replacements during maintenance period.

J. Incidental or Included Items (Continued)

Section 9030 - Plant Material and Planting

9030, 1.08, A, 3	<u>Plants (by count)</u> Delivery, excavation, installation, watering, placing backfill material, mulching, wrapping, staking or guying, herbicide, maintenance during the establishment period, and replacements.
9030, 1.08, B, 3	<u>Plants (by count), With Warranty</u> Delivery, excavation, installation, watering, placing backfill material, mulching, wrapping, staking or guying, herbicide, maintenance during the establishment and warranty periods, and replacements.
9030, 1.08, C, 3	<u>Plants (by lump sum)</u> Delivery, excavation, installation, watering, placing backfill material, mulching, wrapping, staking or guying, herbicide, maintenance during the establishment period, and replacements.
9030, 1.08, D, 3	<u>Plants (by lump sum), With Warranty</u> Delivery, excavation, installation, watering, placing backfill material, mulching, wrapping, staking or guying, herbicide, maintenance during the establishment and warranty period, and replacements.
9030, 1.08, E, 3	<u>Tree Drainage Wells</u> Excavation, furnishing and placing rock, engineering fabric, and placing backfill material.

Section 9040 - Erosion and Sediment Control

9040, 1.07, C	When applicable, conduct all operations in compliance with the Iowa DNR NPDES General Permit No. 2. Labor, equipment, or materials not included as a bid item, but necessary to prevent stormwater contamination from construction related sources, are considered incidental. Incidental work related to compliance with the permit may include, but is not limited to: hazardous materials protection, fuel containment, waste disposal, and providing employee sanitary facilities.
9040, 1.08, A, 1, c	<u>SWPPP Preparation</u> Development of a SWPPP by the Contractor meeting local and state agency requirements, filing the required public notices, filing a Notice of Intent for coverage of the project under the Iowa DNR NPDES General Permit No. 2, and payment of associated NPDES permit fees.
9040, 1.08, A, 2, c	<u>SWPPP Management</u> All work required to comply with the administrative provisions of the Iowa DNR NPDES General Permit No. 2; including record keeping, documentation, updating the SWPPP, filing the Notice of Discontinuation, etc. Item also includes weekly inspections required to satisfy the provisions of General Permit No. 2, unless otherwise specified in the contract documents.

J. Incidental or Included Items (Continued)

9040, 1.08, D, 1, c	Filter Socks, Installation Anchoring stakes.
9040, 1.08, D, 2, c	Filter Socks, Removal Restoration of the area to finished grade and off-site disposal of filter socks and accumulated sediment.
9040, 1.08, E, 3	Temporary RECP Excavation, staples, anchoring devices, and material for anchoring slots.
9040, 1.08, F, 1, c	Wattles, Installation Anchoring stakes.
9040, 1.08, F, 2, c	Wattles, Removal Restoration of the area to finished grade and off-site disposal of wattle and accumulated sediment.
9040, 1.08, G, 1, c	Check Dams, Rock Engineering fabric.
9040, 1.08, G, 2, a, 3)	Check Dams, Manufactured, Installation Anchoring stakes.
9040, 1.08, G, 2, b, 3)	Check Dams, Manufactured, Removal Restoration of the area to finished grade and off-site disposal of manufactured check dam and accumulated sediment.
9040, 1.08, H, 3	Temporary Earth Diversion Structures Removal of the structure upon completion of the project.
9040, 1.08, I, 3	Level Spreaders Maintaining the spreader during the period of construction and removal upon completion of the project, unless otherwise specified in the contract documents.
9040, 1.08, J, 3	Rip Rap Engineering fabric.
9040, 1.08, K, 3	Temporary Pipe Slope Drains Excavation, furnishing and installing pipe and pipe aprons, grading, and removal of the slope drain upon completion of the project.
9040, 1.08, L, 1, c	Sediment Basin, Outlet Structure Concrete base, dewatering device, anti-vortex device, outlet pipe, and anti-seep collars (if specified).
9040, 1.08, L, 2, c	Sediment Basin, Removal of Sediment Dewatering and removal and off-site disposal of accumulated sediment.

J. Incidental or Included Items (Continued)

9040, 1.08, L, 3, c	Sediment Basin, Removal of Outlet Structure Dewatering and off-site disposal of the outlet structure, concrete base, emergency spillway, and accumulated sediment.
9040, 1.08, M, 1, c	Sediment Trap Outlet, Installation Engineering fabric.
9040, 1.08, M, 2, c	Sediment Trap Outlet, Removal of Sediment Dewatering and removal and off-site disposal of accumulated sediment.
9040, 1.08, M, 3, c	Sediment Trap Outlet, Removal of Device Dewatering and off-site disposal of sediment trap outlet and accumulated sediment.
9040, 1.08, N, 1, c	Silt Fence or Silt Fence Ditch Check, Installation Anchoring posts.
9040, 1.08, N, 2, c	Silt Fence or Silt Fence Ditch Check, Removal of Sediment Anchoring posts.
9040, 1.08, N, 3, c	Silt Fence or Silt Fence Ditch Check, Removal of Device Restoration of the area to finished grade and off-site disposal of fence, posts, and accumulated sediment.
9040, 1.08, O, 1, c	Stabilized Construction Entrance (by Square Yard) Subgrade stabilization fabric.
9040, 1.08, O, 2, c	Stabilized Construction Entrance (by Ton) Subgrade stabilization fabric.
9040, 1.08, P, 1, c	Dust Control, Water Furnishing, transporting, and distributing water to the haul road.
9040, 1.08, R, 3	Turf Reinforcement Mats (TRM) Excavation, staples, anchoring devices, and material for anchoring slots.
9040, 1.08, T, 1, c	Inlet Protection Device, Installation Removal of the device upon completion of the project.
9040, 1.08, T, 2, c	Inlet Protection Device, Maintenance Removal and off-site disposal of accumulated sediment.
9040, 1.08, U, 3	Flow Transition Mat Anchoring devices.

Section 9050 - Gabions and Revet Mattresses

9050, 1.08, A, 3	Gabions Furnishing and assembling wire mesh baskets, PVC coating (if specified in the contract documents), fasteners, furnishing and placing gabion stone, engineering fabric, and anchor stakes.

J. Incidental or Included Items (Continued)

9050, 1.08, B, 3	Revet Mattresses Furnishing and assembling wire mesh baskets, PVC coating (if specified in the contract documents), fasteners, furnishing and placing mattress stone, engineering fabric, and anchor stakes.

Section 9060 - Chain Link Fence

9060, 1.08, A, 3	Chain Link Fence Posts, fabric, rails, braces, truss rods, ties, tension wire, tension bands, tension bars, grounds, fittings, PVC coating (if specified in the contract documents), excavation of post holes, and concrete encasement of posts.
9060, 1.08, B, 3	Gates Gate rails, fabric, stretcher bars, braces, vertical stay, hinges, latches, keepers, drop bar lock, center gate stop, and barbed wire (if specified).
9060, 1.08, C, 3	Barbed Wire Furnishing and installing all necessary strands of barbed wire, anchors, and barbed wire supporting arms.
9060, 1.08, D, 3	Removal and Reinstallation of Existing Fence Removing vegetation; removing all fence fabric, appurtenances, posts, and gates; removal of concrete encasement from posts; storage of the removed fencing materials to prevent damage; reinstallation of the posts, gates, and fabric, including all appurtenances; and replacement of any fence parts that are not able to be salvaged and reinstalled. Replace items damaged from Contractor's operations with new materials, at no additional cost to the Contracting Authority.
9060, 1.08, E, 3	Removal of Fence Off-site disposal of fence (including posts, concrete encasement of posts, gates, grounds, and barbed wire) and placing and compacting backfill material in post holes.
9060, 1.08, F, 3	Temporary Fence Furnishing, installing, and removing posts, fabric, ties, and fittings.

Section 9070 - Landscape Retaining Walls

9070, 1.08, A, 3	Modular Block Retaining Wall Excavation, foundation preparation, furnishing and placing wall units, geogrid (if necessary), leveling pad, subdrain, porous backfill material for subdrain, engineering fabric for subdrain, granular backfill material, suitable backfill material, and shoring as necessary.
9070, 1.08, B, 3	Limestone Retaining Wall Excavation, foundation preparation, furnishing and placing leveling pad, limestone, subdrain, porous backfill material for subdrain, engineering fabric for subdrain, suitable backfill material, and shoring as necessary.

J. Incidental or Included Items (Continued)

9070, 1.08, C, 3	<u>Landscape Timbers</u> Excavation, foundation preparation, furnishing and placing leveling pad, landscape timbers, spikes, reinforcing bar, subdrain, porous backfill material for subdrain, engineering fabric for subdrain, suitable backfill material, and shoring as necessary.

Section 9071 - Segmental Block Retaining Walls

9071, 1.08, A, 3	<u>Segmented Block Retaining Wall</u> Design by a Licensed Professional Engineer in the State of Iowa, excavation, foundation preparation, furnishing and placing wall units, geogrid, leveling pad, subdrain, porous backfill material for subdrain, engineering fabric for subdrain, suitable backfill material, and shoring as necessary.
9071, 1.08, C, 3	<u>Granular Backfill Material</u> Furnishing, transporting, placing, and compacting material.

Section 9072 - Combined Concrete Sidewalk and Retaining Walls

9072, 1.08, A, 3	<u>Combined Concrete Sidewalk and Retaining Wall</u> Excavation; foundation preparation; furnishing and placing concrete and reinforcing steel; joint material; subdrain; porous backfill material; suitable backfill material; finishing disturbed areas; and shoring as necessary.

Section 9080 - Concrete Steps, Handrails, and Safety Rail

9080, 1.08, A, 3	<u>Concrete Steps</u> Reinforcement, expansion joint material, and preparation of subgrade.
9080, 1.08, B, 3	<u>Handrail</u> Posts, mounting hardware or concrete grout, and finishing (painted, galvanized, or powder coated).
9080, 1.08, C, 3	<u>Safety Rail</u> Posts, pickets, mounting hardware, epoxy grout, and finishing (painted, galvanized, or powder coated).

Section 10,010 - Demolition

10,010, 1.08, A, 3	<u>Demolition Work</u> Removal of trees, brush, vegetation, buildings, building materials, contents of buildings, appliances, trash, rubbish, basement walls, foundations, sidewalks, steps, and driveways from the site; disconnection of utilities; furnishing and compaction of backfill material; furnishing and placing topsoil; finish grading of disturbed areas; placing and removing safety fencing; removal of fuel and septic tanks and cisterns; seeding; and payment of any permit or disposal fees.

J. Incidental or Included Items (Continued)

10,010, 1.08, B, 3	<u>Plug or Abandon Well</u> Obtaining all permits; plug or abandon private wells according to local, state, and federal regulations.

Section 11,010 - Construction Survey

11,010, 1.08, A, 3	<u>Construction Survey</u> The costs of resetting project control points, re-staking, and any additional staking requested beyond the requirements of this section.
11,010, 1.08, B, 3	<u>Monument Preservation and Replacement</u> Property research and documentation, locating monuments prior to construction, replacement of disturbed monuments, and preparation and filing of the monument preservation certificate.

Section 11,020 - Mobilization

11,020, 1.07, B	When the proposal form does not include a bid item for mobilization, all costs incurred by the contractor for mobilization are incidental to other work and no separate payment will be made.
11,020, 1.08, A, 3	<u>Mobilization</u> The movement of personnel, equipment, and supplies to the project site; the establishment of offices, buildings, and other facilities necessary for the project; and bonding, permits, and other expenses incurred prior to construction.

Section 11,040 - Temporary Sidewalk Access

11,040, 1.08, A, 3	<u>Temporary Pedestrian Residential Access</u> Supplying and placing granular material, continuous maintenance of granular surface, removal of temporary granular sidewalk, and restoring disturbed surfaces to a condition equal to that which existed prior to construction.
11,040, 1.08, B, 3	<u>Temporary Granular Sidewalk</u> Excavation, grading, timber edging, supplying and placing granular material, continuous maintenance of granular surface, removal of temporary granular sidewalk, and restoring disturbed surfaces to a condition equal to that which existed prior to construction.
11,040, 1.08, C, 3	<u>Temporary Longitudinal Channelizing Device</u> Construction, placement, maintenance, and removal of the device.

Section 11,050 - Concrete Washout

11,050, 1.08, A, 3	<u>Concrete Washout</u> Providing concrete washwater containment, collection, and disposal.

K. Bid Items

The following is a list of standard bid items listed in the SUDAS Standard Specifications. The following are suggested bid items. This list may not be all-inclusive. The Engineer may make modifications as necessary.

Item Number	Bid Item	Unit
	Section 2010 - Earthwork, Subgrade, and Subbase	
2010-108-A-0	Clearing and Grubbing	UNIT
2010-108-B-0	Clearing and Grubbing	AC
2010-108-C-0	Clearing and Grubbing	LS
2010-108-D-1	Topsoil, On-site	CY
2010-108-D-2	Topsoil, Compost-amended	CY
2010-108-D-3	Topsoil, Off-site	CY
2010-108-E-0	Excavation, Class 10, Class 12, or Class 13	CY
2010-108-G-0	Subgrade Preparation	SY
2010-108-H-0	Subgrade Treatment, ____ (Type)	SY
2010-108-I-0	Subbase, ____ (Type)	SY
2010-108-J-1	Removal of Structure, ____ (Type)	EA
2010-108-J-2-a	Removal of Known Box Culvert, ____ (Type), ____ (Size)	LF
2010-108-J-2-c	Removal of Known Pipe Culvert, ____ (Type), ____ (Size)	LF
2010-108-J-3-a	Removal of Known Pipe and Conduit, ____ (Type), ____ (Size)	LF
2010-108-K-1	Filling and Plugging of Known Pipe Culverts, Pipes, and Conduits, ____ (Type), ____ (Size)	LF
2010-108-L-0	Compaction Testing	LS
	Section 3010 - Trench Excavation and Backfill	
3010-108-B-0	Rock Excavation	CY
3010-108-C-0	Trench Foundation	TON
3010-108-D-0	Replacement of Unsuitable Backfill Material	CY
3010-108-E-0	Special Pipe Embedment or Encasement	LF
3010-108-F-0	Trench Compaction Testing	LS
	Section 4010 - Sanitary Sewers	
4010-108-A-1	Sanitary Sewer Gravity Main, Trenched, ____ (Type), ____ (Size)	LF
4010-108-A-2	Sanitary Sewer Gravity Main, Trenchless, ____ (Type), ____ (Size)	LF
4010-108-B-1	Sanitary Sewer Gravity Main with Casing Pipe, Trenched, ____ (Type), ____ (Size)	LF
4010-108-B-2	Sanitary Sewer Gravity Main with Casing Pipe, Trenchless, ____ (Type), ____ (Size)	LF
4010-108-C-1	Sanitary Sewer Force Main, Trenched, ____ (Type), ____ (Size)	LF
4010-108-C-2	Sanitary Sewer Force Main, Trenchless, ____ (Type), ____ (Size)	LF
4010-108-D-1	Sanitary Sewer Force Main with Casing Pipe, Trenched, ____ (Type), ____ (Size)	LF

K. Bid Items (Continued)

Item Number	Bid Item	Unit
4010-108-D-2	Sanitary Sewer Force Main with Casing Pipe, Trenchless, ____ (Type), ____ (Size)	LF
4010-108-E-0	Sanitary Sewer Service Stub, ____ (Type), ____ (Size)	LF
4010-108-F-0	Sanitary Sewer Service Relocation	EA
4010-108-G-0	Sewage Air Release Valve and Pit	EA
4010-108-H-0	Removal of Sanitary Sewer, ____ (Type), ____ (Size)	LF
4010-108-I-0	Sanitary Sewer Cleanout	EA
4010-108-K-2	Sanitary Sewer Abandonment, Fill and Plug	LF
	Section 4020 - Storm Sewers	
4020-108-A-1	Storm Sewer, Trenched, ____ (Type), ____ (Size)	LF
4020-108-A-2	Storm Sewer, Trenchless, ____ (Type), ____ (Size)	LF
4020-108-B-1	Storm Sewer with Casing Pipe, Trenched, ____ (Type), ____ (Size)	LF
4020-108-B-2	Storm Sewer with Casing Pipe, Trenchless, ____ (Type), ____ (Size)	LF
4020-108-C-0	Removal of Storm Sewer, ____ (Type), ____ (Size)	LF
4020-108-E-2	Storm Sewer Abandonment, Fill and Plug	LF
	Section 4030 - Pipe Culverts	
4030-108-A-1	Pipe Culvert, Trenched, ____ (Type), ____ (Size)	LF
4030-108-A-2	Pipe Culvert, Trenchless, ____ (Type), ____ (Size)	LF
4030-108-B-0	Pipe Apron, ____ (Type), ____ (Size)	EA
4030-108-C-0	Footing for Concrete Pipe Apron, ____ (Type), ____ (Size)	EA
4030-108-D-0	Pipe Apron Guard	EA
	Section 4040 - Subdrains and Footing Drain Collectors	
4040-108-A-0	Subdrain, ____ (Type), ____ (Size)	LF
4040-108-B-0	Footing Drain Collector, ____ (Type), ____ (Size)	LF
4040-108-C-0	Subdrain Cleanout, ____ (Type), ____ (Size)	EA
4040-108-C-0	Footing Drain Cleanout, ____ (Type), ____ (Size)	EA
4040-108-D-0	Subdrain Outlets and Connections, ____ (Type), ____ (Size)	EA
4040-108-D-0	Footing Drain Outlets and Connections, ____ (Type), ____ (Size)	EA
4040-108-E-0	Storm Sewer Service Stub, ____ (Type), ____ (Size)	LF
	Section 4050 - Pipe Rehabilitation	
4050-108-A-0	Pipe Lining, ____ (Type), ____ (Size)	LF
4050-108-B-0	Building Sanitary Sewer Service Reconnection	EA
4050-108-C-1	Spot Repairs by Pipe Replacement	EA
4050-108-C-2	Spot Repairs by Pipe Replacement	LF

K. Bid Items (Continued)

Item Number	Bid Item	Unit
	Section 5010 - Pipe and Fittings	
5010-108-A-1	Water Main, Trenched, ____ (Type), ____ (Size)	LF
5010-108-A-2	Water Main, Trenchless, ____ (Type), ____ (Size)	LF
5010-108-B-1	Water Main with Casing Pipe, Trenched, ____ (Type), ____ (Size)	LF
5010-108-B-2	Water Main with Casing Pipe, Trenchless, ____ (Type), ____ (Size)	LF
5010-108-C-1	Fitting, ____ (Type), ____ (Size)	EA
5010-108-C-2	Fitting, ____ (Type), ____ (Size)	LB
5010-108-D-0	Water Service Stub, ____ (Type), ____ (Size)	EA
5010-108-E-1	Water Service Pipe, ____ (Type), ____ (Size)	LF
5010-108-E-2	Water Service Corporation, ____ (Type), ____ (Size)	EA
5010-108-E-3	Water Service Curb Stop and Box, ____ (Type), ____ (Size)	EA
	Section 5020 - Valves, Fire Hydrants, and Appurtenances	
5020-108-A-0	Valve, ____ (Type), ____ (Size)	EA
5020-108-B-0	Tapping Valve Assembly, ____ (Size)	EA
5020-108-C-0	Fire Hydrant Assembly	EA
5020-108-D-0	Flushing Device (Blowoff), ____ (Size)	EA
5020-108-F-0	Valve Box Extension	EA
5020-108-G-0	Valve Box Replacement	EA
5020-108-H-0	Fire Hydrant Adjustment	EA
	Section 6010 - Structures for Sanitary and Storm Sewers	
6010-108-A-0	Manhole, ____ (Type), ____ (Size)	EA
6010-108-B-0	Intake, ____ (Type), ____ (Size)	EA
6010-108-C-0	Drop Connection	EA
6010-108-D-0	Casting Extension Ring	EA
6010-108-E-0	Manhole Adjustment, Minor	EA
6010-108-E-0	Intake Adjustment, Minor	EA
6010-108-F-0	Manhole Adjustment, Major	EA
6010-108-F-0	Intake Adjustment, Major	EA
6010-108-G-0	Connection to Existing Manhole	EA
6010-108-G-0	Connection to Existing Intake	EA
6010-108-H-0	Remove Manhole	EA
6010-108-H-0	Remove Intake	EA
	Section 6020 - Rehabilitation of Existing Manholes	
6020-108-A-0	Infiltration Barrier, ____ (Type)	EA
6020-108-B-0	In-situ Manhole Replacement, Cast-in-place Concrete	VF
6020-108-C-0	In-situ Manhole Replacement, Cast-in-place Concrete with Plastic Liner	VF
6020-108-D-0	Manhole Lining with Centrifugally Cast Cementitious Mortar Liner with Epoxy Seal	VF

K. Bid Items (Continued)

Item Number	Bid Item	Unit
	Section 7010 - Portland Cement Concrete Pavement	
7010-108-A-0	Pavement, PCC, ____ (Thickness)	SY
7010-108-E-0	Curb and Gutter, ____ (Width), ____ (Thickness)	LF
7010-108-F-0	Beam Curb	LF
7010-108-G-0	Concrete Median	SY
7010-108-I-0	PCC Pavement Samples and Testing	LS
7010-108-K-0	PCC Pavement Widening, ____ (Thickness)	SY
7010-108-L-1	PCC Overlay, Furnish Only	CY
7010-108-L-2	PCC Overlay, Place Only	SY
7010-108-L-3	Surface Preparation for Bonded PCC Overlay	SY
7010-108-L-4	Surface Preparation for Unbonded PCC Overlay	SY
7010-108-L-5	HMA Stress Relief Course for Unbonded PCC Overlay	SY
	Section 7020 - Hot Mix Asphalt Pavement	
7020-108-A-0	Pavement or Overlay, HMA	TON
7020-108-B-0	Pavement or Overlay, HMA, ____ (Thickness)	SY
7020-108-C-0	HMA Base Widening	TON
7020-108-D-0	HMA Base Widening, ____ (Thickness)	SY
7020-108-H-0	HMA Pavement Samples and Testing	LS
	Section 7030 - Sidewalks, Shared Use Paths, and Driveways	
7030-108-A-0	Removal of Sidewalk	SY
7030-108-A-0	Removal of Shared Use Path	SY
7030-108-A-0	Removal of Driveway	SY
7030-108-B-0	Removal of Curb	LF
7030-108-C-0	Shared Use Path, ____ (Type), ____ (Thickness)	SY
7030-108-D-0	Special Subgrade Preparation for Shared Use Path	SY
7030-108-E-0	Sidewalk, PCC, ____ (Thickness)	SY
7030-108-F-1	Brick Sidewalk with Sand Base	SY
7030-108-F-2	Brick Sidewalk with Concrete Base	SY
7030-108-G-0	Detectable Warning	SF
7030-108-H-1	Driveway, Paved, ____ (Type), ____ (Thickness)	SY
7030-108-H-2	Driveway, Granular	SY or TON
7030-108-I-0	Sidewalk Assurance Testing	LS
7030-108-I-0	Shared Use Path Assurance Testing	LS
7030-108-I-0	Driveway Assurance Testing	LS

K. Bid Items (Continued)

Item Number	Bid Item	Unit
	Section 7040 - Pavement Rehabilitation	
7040-108-A-0	Full Depth Patches	SY
7040-108-B-0	Subbase Over-excavation	TON
7040-108-C-0	Partial Depth Patches	SF
7040-108-D-0	Crack and Joint Cleaning and Filling, Hot Pour	LF
7040-108-E-1	Crack Cleaning and Filling, Emulsion	LF
7040-108-E-2	Hot Mix Asphalt for Crack Filling	TON
7040-108-F-0	Diamond Grinding	SY
7040-108-G-0	Milling	SY
7040-108-H-0	Pavement Removal	SY
7040-108-I-0	Curb and Gutter Removal	LF
	Section 7050 - Asphalt Stabilization	
7050-108-A-0	Asphalt Stabilization	SY
	Section 7060 - Bituminous Seal Coat	
7060-108-A-0	Bituminous Seal Coat	SY
7060-108-B-1	Cover Aggregate, ____ (Size)	TON
7060-108-B-2	Binder Bitumen	GAL
	Section 7070 - Emulsified Asphalt Slurry Seal	
7070-108-A-0	Emulsified Asphalt Slurry Seal	SY
7070-108-B-1	Aggregate, ____ (Size)	TON
7070-108-B-2	Asphalt Emulsion	GAL
	Section 7080 - Permeable Interlocking Pavers	
7080-108-B-0	Engineering Fabric	SY
7080-108-C-0	Underdrain, ____ (Type), ____ (Size)	LF
7080-108-D-0	Storage Aggregate	TON
7080-108-E-0	Filter Aggregate	TON
7080-108-F-0	Permeable Interlocking Pavers, ____ (Type)	SY
7080-108-G-0	PCC Edge Restraint, ____ (Type), ____ (Size)	LF
	Section 8010 - Traffic Control	
8010-108-A-0	Traffic Signal	LS
8010-108-B-0	Temporary Traffic Signal	LS
	Section 8020 - Pavement Markings	
8020-108-B-0	Painted Pavement Markings, Solvent/Waterborne	STA
8020-108-C-0	Painted Pavement Markings, Durable	STA
8020-108-D-0	Painted Pavement Markings, High-Build	STA

K. Bid Items (Continued)

Item Number	Bid Item	Unit
8020-108-E-0	Permanent Tape Markings	STA
8020-108-F-0	Wet, Retroreflective Removable Tape Markings	STA
8020-108-G-0	Painted Symbols and Legends	EA
8020-108-H-0	Precut Symbols and Legends	EA
8020-108-I-0	Temporary Delineators	EA
8020-108-J-0	Raised Pavement Markers	EA
8020-108-K-0	Pavement Markings Removed	STA
8020-108-L-0	Symbols and Legends Removed	EA
8020-108-M-0	Grooves Cut for Pavement Markings	STA
8020-108-N-0	Grooves Cut for Symbols and Legends	EA
	Section 9010 - Seeding	
9010-108-A-0	Conventional Seeding, Seeding, Fertilizing, and Mulching	AC
9010-108-B-0	Hydraulic Seeding, Seeding, Fertilizing, and Mulching	AC
9010-108-C-0	Pneumatic Seeding, Seeding, Fertilizing, and Mulching	AC
9010-108-D-0	Watering	MGAL
9010-108-E-0	Warranty	LS
	Section 9020 - Sodding	
9020-108-A-0	Sod	SQ
	Section 9030 - Plant Material and Planting	
9030-108-A-0	Plants, ____ (Type)	EA
9030-108-B-0	Plants with Warranty, ____ (Type)	EA
9030-108-C-0	Plants	LS
9030-108-D-0	Plants with Warranty	LS
9030-108-E-0	Tree Drainage Wells	EA
	Section 9040 - Erosion and Sediment Control	
9040-108-A-1	SWPPP Preparation	LS
9040-108-A-2	SWPPP Management	LS
9040-108-B-0	Compost Blanket, ____ (Thickness)	SF
9040-108-C-0	Filter Berm, ____ (Size)	LF
9040-108-D-1	Filter Sock, ____ (Size)	LF
9040-108-D-2	Filter Sock, Removal	LF
9040-108-E-0	Temporary RECP, ____ (Type)	SY
9040-108-F-1	Wattle, ____ (Type), ____ (Size)	LF
9040-108-F-2	Wattle, Removal	LF
9040-108-G-1	Check Dam, Rock	TON
9040-108-G-2-a	Check Dam, Manufactured, ____ (Type), ____ (Size)	LF
9040-108-G-2-b	Check Dam, Manufactured, Removal, ____ (Type)	LF
9040-108-H-0	Temporary Earth Diversion Structure, ____ (Type), ____ (Size)	LF
9040-108-I-0	Level Spreader	LF
9040-108-J-0	Rip Rap, ____ (Type)	TON

K. Bid Items (Continued)

Item Number	Bid Item	Unit
9040-108-K-0	Temporary Pipe Slope Drain, ____ (Type), ____ (Size)	LF
9040-108-L-1	Sediment Basin, Outlet Structure, ____ (Size)	EA
9040-108-L-2	Sediment Basin, Removal of Sediment	EA
9040-108-L-3	Sediment Basin, Removal of Outlet Structure	EA
9040-108-M-1	Sediment Trap Outlet	TON
9040-108-M-2	Sediment Trap Outlet, Removal of Sediment	EA
9040-108-M-3	Sediment Trap Outlet, Removal of Device	EA
9040-108-N-1	Silt Fence or Silt Fence Ditch Check	LF
9040-108-N-2	Silt Fence or Silt Fence Ditch Check, Removal of Sediment	LF
9040-108-N-3	Silt Fence or Silt Fence Ditch Check, Removal of Device	LF
9040-108-O-1	Stabilized Construction Entrance	SY
9040-108-O-2	Stabilized Construction Entrance	TON
9040-108-P-1	Dust Control, Water	MGAL
9040-108-P-2	Dust Control, Product	SY
9040-108-Q-1	Erosion Control Mulching, Conventional	AC
9040-108-Q-2	Erosion Control Mulching, Hydromulching	AC
9040-108-R-0	Turf Reinforcement Mats, ____ (Type)	SQ
9040-108-S-0	Surface Roughening	SF
9040-108-T-1	Inlet Protection Device, ____ (Type)	EA
9040-108-T-2	Inlet Protection Device, Maintenance	EA
9040-108-U-0	Flow Transition Mat	SF
	Section 9050 - Gabions and Revet Mattresses	
9050-108-A-0	Gabions, ____ (Type)	CY
9050-108-B-0	Revet Mattresses, ____ (Type)	CY
	Section 9060 - Chain Link Fence	
9060-108-A-0	Chain Link Fence, ____ (Type), ____ (Size)	LF
9060-108-B-0	Gates, ____ (Type), ____ (Size)	EA
9060-108-C-0	Barbed Wire, ____ (Type of Supporting Arm)	LF
9060-108-D-0	Removal and Reinstallation of Existing Fence, ____ (Type), ____ (Size)	LF
9060-108-E-0	Removal of Fence	LF
9060-108-F-0	Temporary Fence, ____ (Type), ____ (Size)	LF
	Section 9070 - Landscape Retaining Walls	
9070-108-A-0	Modular Block Retaining Wall	SF
9070-108-B-0	Limestone Retaining Wall	SF
9070-108-C-0	Landscape Timbers	SF
	Section 9071 - Segmental Block Retaining Walls	
9071-108-A-0	Segmental Block Retaining Wall	SF
9071-108-C-0	Granular Backfill Material	TON

K. Bid Items (Continued)

Item Number	Bid Item	Unit
	Section 9072 - Combined Concrete Sidewalk and Retaining Wall	
9072-108-A-0	Combined Concrete Sidewalk and Retaining Wall	CY
	Section 9080 - Concrete Steps, Handrails, and Safety Rail	
9080-108-A-0	Concrete Steps, ____ (Type)	SF
9080-108-B-0	Handrail, ____ (Type)	LF
9080-108-C-0	Safety Rail	LF
	Section 10,010 - Demolition	
10,010-108-A	Demolition Work	LS
10,010-108-B	Plug or Abandon Well	EA
	Section 11,010 - Construction Survey	
11,010-108-A	Construction Survey	LS
11,010-108-B	Monument Preservation and Replacement	LS
	Section 11,020 - Mobilization	
11,020-108-A	Mobilization	LS
	Section 11,030 - Temporary Services During Construction	
11,030-108-A-0	Maintenance of Postal Service	LS
11,030-108-B-0	Maintenance of Solid Waste Collection	LS
	Section 11,040 - Temporary Sidewalk Access	
11,040-108-A-0	Temporary Pedestrian Residential Access	SY
11,040-108-B-0	Temporary Granular Sidewalk	SY
11,040-108-C-0	Temporary Longitudinal Channelizing Device	LF
	Section 11,050 - Concrete Washout	
11,050-108-A-0	Concrete Washout	LS

Public Improvement Contracts

A. General

Public improvements contracts should be used to ensure construction of all public improvements to the standards provided by the Jurisdiction. These contracts may also be used between the developer, contractor, and the Jurisdiction for private subdivision or site developments. After the plans and the contract have been given Jurisdictional approval, changes should not be made in the design or scope of work without addenda or a change order approved by the Jurisdiction.

If the change involves engineering details shown on the plans, the original plans (depending on the Jurisdiction's requirements, plans may be held by the Project Engineer or Jurisdiction) should be modified by the Project Engineer and should accompany a change order. Work on portions of the project involved in the change order should not be performed until the change order is approved by the Jurisdiction.

B. Contract Documents

The Project Engineer should use the contract documents required by the Jurisdiction. Sample contract document forms are available on the SUDAS website at www.iowasudas.org.

The following items are typically included in the contract documents:

1. Notice to Bidders and Notice to Public Hearing

2. Instructions to Bidders

3. Proposal
   - Part A - Scope of Wok
   - Part B - Acknowledgement of Addenda
   - Part C - Bid Items, Quantities, and Prices
   - Part D - General
   - Part E - Additional Requirements
   - Part G - Identity of Bidder
   - Proposal Attachments

4. Bid Bond

5. Contract and Contract Attachment

6. Performance, Payment, and Maintenance Bond

Plans of Record

A. General

Plans of record (as-built) information should be added to the original plan tracing or media. These tracings may be checked out by the Project Engineer and the record information should be clearly shown thereon. No original design data should be removed from these tracings or media. The Project Engineer returns the tracings to the Jurisdiction Engineer for filing as plans of record.

B. Information to be Shown on Plans of Record

1. General:

a. Final quantities.

b. Plans of record certification or label.

c. Any other information deemed necessary by the Project Engineer.

d. Location and elevation of any drainage tiles or other utilities encountered.

2. Paving Plans:

a. Pavement width and all radii at returns.

b. Stationing from the beginning to the end of the construction, stationing of intakes, manholes, and centerline of intersecting streets.

c. Cross-sections will generally not be required. However, if the Jurisdiction has reason to believe that the plans do not accurately reflect the field conditions, the Jurisdiction may require record cross-sections.

3. Sewer Plans:

a. Invert elevations of all pipes at manholes, structures, inlets, outlets, and rims.

b. Lengths, type, and sizes of all pipes.

c. Stationing, location, and type of all structures and begin and end construction.

d. Location of all wyes, tees, or stubs and riser lengths.

e. Manhole number system to be labeled for each manhole.

4. Drainage Ditch Plans:

a. Invert elevation and, if required, cross-sections.

b. Invert elevations or flow lines of culverts, drop structure inlets, and outlets.

c. Stationing, location and type of inlets, outlets, structures, and begin and end construction.

5. Water Main Plans:

a. Locations of all pipes, fittings, and fire hydrants.

b. Lengths, type, and sizes of all pipes.

c. Stationing and location and type of all water service stubs.

d. Fire hydrant number system to be labeled for each hydrant.

6. Utilities: The Project Engineer is not required to locate utilities that are not part of or affected by the construction project or private utility lines that were installed by the utility company

Proprietary Products

A. Proprietary Products

The SUDAS Standard Specifications do not use references to proprietary products. A list of products that may be considered for approval by the engineer to meet the specifications is contained on the SUDAS website (www.iowasudas.org). This list is not intended to be a comprehensive listing of all acceptable products, nor is it intended to exclude any product that meets the specifications requirements. This list of commonly used products is provided simply as a convenience to the designer. No assurance is inferred or made regarding the use of these products. No verification is made for the listed products as to the continued compliance with the specifications. It remains the responsibility of the Project Engineer to verify that the products used meet the needs of the project and the appropriate specifications. Changes to this list will be made at the request of the specifications users and will be approved by the SUDAS Board of Directors on a yearly basis.

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CHAPTER 2
Stormwater

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General Information

A. Concept

This section sets forth concepts for stormwater management objectives. Development can significantly alter the hydrology within the urbanized portion of a watershed as residential and commercial construction leads to an increase in impervious surfaces in the drainage area. As a result, the response of an urbanized watershed to precipitation is significantly different from the response of a natural watershed. Post-developed peak runoff is expected to exceed pre-developed runoff from a similar storm event. The most common effects are reduced infiltration and decreased travel time, which significantly increases peak discharge rates and runoff volumes. Factors influencing the amount (volume) of runoff include precipitation depth, the infiltrative capacity of soils, soil moisture, antecedent rainfall, cover type, the amount of impervious surfaces, and surface retention. Travel time is determined primarily by slope, length of flow path, depth of flow, and roughness of flow surfaces. To accommodate the higher rates and volumes of stormwater runoff in suburban and higher-density urban development, storm sewer conveyance systems are installed to provide efficient drainage of the landscape. Additional protection is provided through detention and storage structures to control release rates to downstream systems. Traditional design considerations have been the prevention of damage to the development site, streams, drainageways, streets, public and private property from flooding, and to the reduction of soil erosion. With the implementation of the stormwater NPDES Phase I and II regulations, stormwater runoff quality is now an additional management goal for some communities.

B. Informing the Public

Engineers typically use the storm reoccurrence interval (i.e. 100 year storm) in their discussions and presentations on stormwater projects. The reoccurrence interval concept is somewhat difficult for the general public to understand. As a result, many questions have resulted from the significant rainfall and flooding events that have occurred over the past few years. These questions often focus on the 100 year storm event. A common perception is that once this level of storm has been received, it will not occur for another 100 years.

The recurrence interval concept is somewhat difficult to understand for those not trained in hydrology. To provide a greater level of understanding, public presentations should include rainfall information in terms of percentage or probability. Thus, a 100 year reoccurrence interval storm should be expressed as a storm that has a 1% chance of occurring in any one year or a 10% chance of occurring in a 10 year period (see Table 2B-2.01). Describing the storms in terms of percentages may help break down the perception that once a 100 year storm has occurred, it will not occur for another 100 years.

The public should also be informed that the storm frequency used for design is based on past storm occurrences. Inaccuracies result from the extrapolation of that data, especially if the number of data points is limited. In addition, storm events very rarely replicate themselves in terms of rainfall intensity, duration, and location within a drainage basin. As a result, calculating runoff is not an exact science. To further complicate matters, indications from researchers show that rainfall events are becoming more intense and runoff faster in rural areas as well as in urban environments. This compounds the inaccuracies associated with predicting rainfall events and their related runoff.

The public should also be made aware of the difference between a rainfall event and a flood event. This may help them to understand how a small interval rainfall event can actually trigger a large flood event. If streams and rivers are already full and the soil is saturated, the rain cannot be absorbed. The runoff increases and even though the rainfall event may have been a 25 year event, the runoff can exceed a 100 year flood. This can also occur if the storm moves down the drainage basin at the same speed that the runoff is occurring. Conversely, during a dry period a 50 year rainfall event may result in only a 10 year flood event as a result of soil absorbing more moisture and rivers and streams flowing at low levels.

Despite the shortcomings noted above, the information presented here is the best information available and is appropriate for use to design stormwater facilities.

This chapter includes the traditional hydrologic analysis and design of stormwater runoff conveyance for larger storm events to prevent flooding. The traditional management goal for detention and storage has been to manage runoff from larger rainfall events, typically greater than the 5 year recurrence interval (RI). While traditional detention practices can reduce the peak runoff flows from urban development, the increase in runoff volume and frequency of peak flows is not reduced and little improvement in stormwater quality is accomplished.

NPDES Phase I and II communities and those desiring to implement post construction water quality practices are encouraged to reference the Iowa Stormwater Management Manual (http://www.iowadnr.gov/Environmental-Protection/Water-Quality/NPDES-Storm-Water/Storm-Water-Manual), which expands on stormwater management best management practices (BMP's).

The Engineer is encouraged to use cost-effective designs that are hydrologically and hydraulically appropriate through the use of good engineering judgment.

C. Conditions

1. Design data provided by the Project Engineer should demonstrate that investigations include:

    a. The function of the streets as part of the stormwater system, including level of anticipated flooding of street surfaces and encroachment into driving lanes.

    b. Gutters and intakes are adequate to prevent excessive flooding of streets and right-of-ways.

    c. Culverts and storm pipes are designed to sufficient size.

    d. Adequate overland relief with proper easements for storms larger than the design storm.

    e. Street grades are coordinated with lot drainage; lot drainage slopes will not be less than 1 1/2% to minimize ponding, and not excessive to cause uncontrollable erosion.

    f. Spot elevations should be listed at each rear lot corner, at the mid-point of the side yard line, and along the proposed drainage ways and easements.

2. The Project Engineer should evaluate drainage alternatives to handle the runoff and select the optimum design that will strike a balance between initial capital costs, maintenance costs, and public protection. Consideration should also be given to safety, environmental protection, and maintenance of the drainage system. Care should be exercised in developing drainage systems that depend solely on a specified protection level. Designers need to keep in mind that rainfall and runoff events seldom, if ever, occur at a specified frequency or duration. Therefore, at critical locations, additional protection should be considered, depending upon the drainage basin

characteristics and the degree of protection necessary downstream.

The following are examples of locations where damage can occur at the specified design frequency and duration when emergency spillways or outlets are not made available.

- Drainage ways between buildings such as housing and in backyards.
- Enclosed storm sewers adjacent to private property, where a single inlet could be plugged, resulting in significant damage to adjacent property.
- Single-lot or multiple-lot stormwater detention.

3. In addition to the potential damage in these particular areas, maintenance of the stormwater conveyance needs to be considered. Private-owner or homeowner association maintenance has the advantage of simplified responsibilities, without direct cost to the general taxpayer. The disadvantage is when the homeowner or association is not capable of maintaining a stormwater system on a continuous basis. Other options to be considered are delayed transfer of ownership from builder to homeowner's association, to ensure proper stormwater conveyance system operation; or the issuance of a performance or maintenance bond by the builder, valid for a specified period of time. When the stormwater conveyance system is significant enough that the normal individual or group of individuals does not have the means for continuous maintenance, other maintenance alternatives need to be developed that involve Jurisdiction-owned facilities. This would involve construction and maintenance by the Jurisdiction, funded through:
   - A one-time charge to the developer that is placed into a stormwater escrow account for immediate or future stormwater improvements.
   - A stormwater utility assessment (either a one time lump sum or monthly charge).
   - Construction of the stormwater facility by the developer that would be owned and maintained by the Jurisdiction.

4. Runoff analysis should be based upon proposed land use, and should take into consideration all contributing runoff from areas outside of the study areas.

5. All undeveloped land lying outside of the study area should be considered as fully developed based upon the Jurisdiction's comprehensive plan. The project designer should check with the Jurisdiction regarding upstream conditions.

6. If future land use of a specific undeveloped area is unknown, the runoff coefficient should be established on a conservative basis. The probable future flow pattern in undeveloped areas should be based on existing natural topographic features (existing slopes, drainage ways, etc.). Average land slopes in both developed and undeveloped areas may be used in computing runoff. However, for areas in which drainage patterns and slopes are established, these should be utilized.

7. Flows and velocities that may occur at a design point when the upstream area is fully developed should be considered. Drainage facilities should be designed such that increased flows and velocities will not cause erosion damage.

8. The primary use of streets should be for the conveyance of traffic. The computed amount of runoff in streets should not exceed the requirements set forth herein.

9. The use of detention and natural drainage ways is recommended and encouraged whenever possible. The changing of natural drainage way locations may not be approved unless such change is shown to be without unreasonable hazard and liability, substantiated by thorough analysis and investigation.

10. Restrictive covenants, surface flowage easements, and impoundment easements may be required to be executed and recorded to provide for the protection and maintenance of grassed drainage

swales and grassed drainage detention areas within build-up areas.

If the Jurisdictional Engineer's approval is given to the use of natural ditches, the Project Engineer should show that the project will have minimum disruption of the existing environment and covenants may be required to be executed and recorded to provide protection. The Jurisdictional Engineer may allow changes in the ditch, provided state and federal guidelines and regulations will be followed.

11. In the design of storm drainage systems, consideration should be given to both surface and subsurface sources. Subsurface drainage systems should be designed where required. The discharge from such underdrain systems should not flow over sidewalks or onto streets after completion of the project.

12. Land grading of the project site should be performed to take advantage of existing contours and minimize soil disturbance. Steep slopes should be avoided. If steep slopes are necessary, an attempt should be made to save natural grasses, shrubs, and trees on these slopes and re-establish ground cover and permanent erosion control measures as soon as possible.

13. The planning and design of drainage systems should be such that problems are not transferred from one location to another. Outfall points and velocities should be designed in such a manner that will not create flooding hazards downstream.

14. Where a master drainage plan for a Jurisdiction is available, the flow routing for both the minor storm and major storm runoff should conform to said plan. Drainage easements conforming to the master plan will be required and should be designated on all drainage drawings and subdivision plats.

15. Any proposed building or construction of any type of structure including retaining walls, fences, etc., or the placement of any type of fill material that will encroach on any utility or drainage easement, requires written approval of the Jurisdiction. Such structure will not impair surface or subsurface drainage from surrounding areas.

16. The design for stormwater management facilities should comply with the following:

    a. Local Jurisdiction's design standards

    b. Requirements and standards of the Iowa DNR (for large detention or retention structures)

    c. Plumbing code

    d. Iowa Code regarding drainage law

    e. In case of a conflict between the above design standards, the most restrictive requirement should apply

17. Construction should comply with the most recent edition of the SUDAS Specifications. All details, materials, and storm sewer appurtenances should comply with these specifications.

18. The Environmental Protection Agency (EPA) approved the Final Stormwater Rule under the National Pollutant Discharge Elimination System (NPDES). Under this rule, qualified projects are required to have stormwater discharge permits. An erosion and sediment control plan should be developed according to the guidelines presented in Chapter 7 - Erosion and Sediment Control.

D. Unified Sizing Criteria

1. **General Information:** This section provides a brief description of the unified sizing criteria utilized as part of overall stormwater management. The unified sizing criteria are intended to be used collectively, to address overall stormwater impacts, including both stormwater quality and quantity, of site development. When used as a set, the unified criteria control the entire range of hydrologic events, from the smallest runoff producing rainfalls (≥ 0.1 inches) to the 100 year storm.

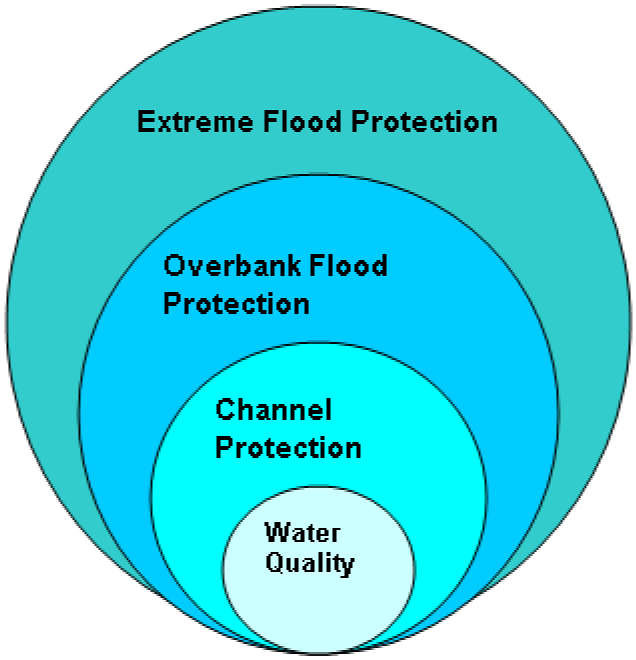
   While this manual does not address stormwater quality requirements (refer to the Iowa Stormwater Management Manual for stormwater quality design), the overall unified sizing criteria is summarized in Table 2A-2.01 and Figure 2A-2.01 below to give the designer an understanding of how each criterion fit together in the overall stormwater management approach.

Table 2A-2.01: Summary of the Recommended Unified Stormwater Sizing Criteria for Management of Stormwater Quality and Quantity

Sizing Criteria	Recommended Method
Water Quality Volume, WQv	Treat the runoff from 90% of the storms that occur in an average year. For Iowa, this equates to providing water quality treatment for the runoff resulting from a rainfall depth of 1.25 inches or less. Goal is to reduce average annual post-development total suspended solids loadings by 80%.
Recharge Volume, Rev	Fraction of WQv, depending on pre development soil hydrologic group.
Channel Protection Storage Volume, Cpv	Provide 24 hours of extended detention of the runoff from the 1 year 24 hour duration storm event to reduce bank-full flows and protect downstream channels from erosive velocities and unstable conditions.
Overbank Flood Protection, Qp	Provide peak discharge control of the 5 year storm event such that the post-development peak rate does not exceed the downstream conveyance capacity and/or cause overbank flooding in local urban watersheds. Some jurisdictions may require peak discharge control for the 2 year storm event.
Extreme Flood Protection, Qf (Major Storm)	Evaluate the effects of the 100 year storm on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts of the extreme storm event through detention controls and/or floodplain management.

Figure 2A-2.01 illustrates the relative volume requirements of each of the unified stormwater sizing criteria, as well as demonstrates that the criteria are "nested" within one another, i.e., the extreme flood protection volume requirement also contains the overbank flood protection volume, the channel protection volume, and the water quality treatment volume.

Figure 2A-2.01: Relationship of the Unified Stormwater Sizing Criteria



Source: Adapted from Georgia Stormwater Manual, Vol. 2, 2001

As previously mentioned, this manual does not address the stormwater quality aspects of the unified sizing criteria. Additional information for the stormwater quality criteria, including overbank and extreme flood protection, is provided below.

2. **Overbank Flood Protection Volume Requirements (Qp):** The primary purpose of the overbank flood protection volume sizing criteria is to prevent an increase in the frequency and magnitude of out-of-bank flooding generated by development (e.g., flow events that exceed the bank-full capacity of the channel and therefore must spill over into the floodplain). Overbank flood protection for the 10 year storm is only required if local approval authorities have no control of floodplain development, no control over infrastructure and conveyance system capacity design, or determine that downstream flooding will occur as a result of the proposed development.

   For most regions of the state, the overbank flood control criteria equates to preventing the post-development 5 year (or 10 year), 24 hour storm peak discharge rate (Qp_5) from exceeding the pre-development peak discharge rate. In some local jurisdiction drainage systems, piped conveyance constraints may dictate the use of a 2 year pre-development peak discharge for post-development flows. In many jurisdictions, the storm sewer intake and piping capacity is sized for conveyance of the 5 year frequency runoff. For control of local flooding for areas connected to these conveyance systems, the upstream release rate must be restricted to meet the existing conveyance capacity to prevent local flooding of streets and properties. For drainage areas connected directly to open channel conveyances (swales and natural stream channels), the 10 year frequency runoff discharge is used.

3. **Extreme Flood Volume (Qf):** The intent of the extreme flood criteria is to prevent flood damage from large storm events and maintain the boundaries of the pre-development 100 year Federal Emergency Management Agency (FEMA) and/or locally designated floodplain.

This is typically done in two ways:

a. 100 Year Control: Requires storage to attenuate the post development 100 year, 24 hour peak discharge (Qf) to pre-development 100 year rates. The Qf is the most stringent and expensive level of flood control, and is generally not needed if the downstream development is located out of the 100 year floodplain. In many cases, the conveyance system leading to a stormwater structure is designed based on the discharge rate for the 10 year storm (Qp_{10}). In these situations, the conveyance systems may be the limiting hydrologic control.

b. Reserve Ultimate 100 Year Floodplain: 100 year storm control may be required by an appropriate review authority in the following cases.
- Buildings or developments are located within the ultimate 100 year floodplain
- The reviewing authority does not completely control the 100 year floodplain

Hydraulic/hydrologic investigations may be required to demonstrate that downstream roads, bridges, and public utilities are adequately protected from the Qf storm. These investigations typically extend to the first downstream tributary of equal or greater drainage area or to any downstream dam, highway, or natural point of restricted stream flow. Specific requirements for floodplain management and construction of infrastructure and/or excavation within the floodway can be found in Iowa Code Chapter 335.

E. Floodplain Management

Although not a direct element of the municipal stormwater conveyance design, floodplain management should be considered along with the overall stormwater management plan to manage the floodplain as it relates to the various stormwater conveyance means, pipes, culverts, streams, and open channels.

Floodplain management, when integrated with the overall stormwater management program, provides a regulatory means to improve the surface water system throughout the municipality.

F. References

Georgia Stormwater Manual. Vol. 2. 2001.

Stormwater Regulations and Permitting

A. Iowa Drainage Law and Resources

Chapter 468 of the Iowa Code covers a majority of Iowa's drainage law with respect to landowner rights and responsibilities. This chapter covers the establishment and operation of drainage districts as well as laws governing modifying, diverting, or blocking existing drainage ways.

The Iowa Drainage Law Manual (http://www.ctre.iastate.edu/pubs/drainage_law/), developed by the Center for Transportation Research and Education (now the Institute for Transportation) at Iowa State University, summarizes drainage laws as described in the Iowa Administrative Code and provides practical solutions to common drainage problems.

B. Regulated Activities

In Iowa, two agencies administer permit programs for protecting the state's water resources and ensuring their wise use. Some local government agencies have also established permit programs related to land subdivision and land disturbing activities. The primary agencies are:

1. **The Iowa DNR:** Iowa DNR administers permit programs for conserving and protecting Iowa′s water, recreational, and environmental resources, and for the prevention of damage resulting from unwise floodplain development. In addition, Iowa DNR has jurisdiction over sovereign lands and waters, and certain fee title lands of the state, and land below the ordinary high water mark on meandered streams and lakes.

    a. **General Permit No. 2:** For "stormwater associated with industrial activity for construction activities" (land disturbing 1 acre or more). Construction activities that result in the disturbance of 1 acre or more of ground cover are required to obtain an NPDES general permit normally associated with earthwork, grading, or any other non-agricultural land-disturbing activity. The goal of the permit is to reduce the amount of sediment being transported from construction site by stormwater runoff.

    b. **Other Iowa DNR Permits:** (relating to protection of water and recreational sources or adjacent lands):
        1) **Floodplain Construction Permits:** Iowa DNR has authority to regulate construction on all floodplains and floodways in the state. http://www.iowadnr.gov/water/floodplain/index.html. Local governments may have obtained transfer of this jurisdiction from Iowa DNR.
        2) **Construction Permits:** Pursuant to the Iowa Code, no person, association, or corporation can build or erect a pier, wharf, sluice, piling, wall, fence, obstruction, building, or erection of any kind, upon or over any state-owned land or water under the jurisdiction of Iowa DNR, without first obtaining a permit from Iowa DNR. http://www.iowadnr.gov/InsideDNR/RegulatoryAir/ConstructionPermits.aspx.
        3) **Special Permits:** Projects involving a standard recreational boat dock require authorization by Iowa DNR. Permits are also required by commercial operations removing sand or aggregate from meandered streams. http://www.iowadnr.gov/

2. **The US Army Corps of Engineers (USACE):** The USACE has authority over public waterways. This includes intrastate lakes, rivers, streams, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, all impoundments of waters and tributaries of waters identified above.

   a. **Clean Water Act Section 404 Permit Program:** Prior to conducting work on or in a regulated water of the U.S., a Section 404 permit must first be obtained from the USACE. Additional information on the 404 program may be found in the Iowa DOT Local Systems I.M. No. 3.130.

   b. **Wetlands:** Wetlands are defined as "those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions." Wetlands have three essential characteristics, all of which must be present for an area to be identified as a wetland. This includes hydrophytic (water-loving) vegetation, hydric soils, and wetland hydrology

      1) **Wetland Delineation:** Identification of Section 404-regulated wetlands requires wetland delineation by the USACE, the EPA, or by submission of a wetland delineation report to the USACE by a qualified wetland specialist. Wetland delineation is often requested or contracted by a property owner who needs to know restrictions on the development or use of the land. In particular, a property owner may need wetland delineation when seeking an individual or general permit.
      2) **Wetland Mitigation:** Every effort should be made at the beginning of a project to avoid or minimize impacts. Any project that does not meet the conditions of any one of the Nationwide Permits must be sent to the USACE and probably will require satisfactory mitigation for the loss of wetlands. Mitigation is defined as wetland restoration, creation, enhancement, or preservation for the purpose of compensating for unavoidable wetland losses in advance of development actions, when such compensation cannot be achieved at the development site or would not be as environmentally beneficial.

3. **Joint Application:** Given the regulatory relationship between the Iowa DNR and the USACE, certain projects require authorization from both agencies before work can commence. Construction, excavation, or filling in streams, lakes, wetlands, or floodplains may require permits from both agencies. Specifically, State Section 401 water quality certification is mandatory for all projects requiring a Federal Section 404 permit. In order to simplify this process, a joint application form has been developed for the permit process for any of the following activities:
   - Cutting the bank of a river or stream
   - Any excavation or dredging in a stream or channel
   - Channel changes or relocations (including stream straightening)
   - Construction of any permanent dock, pier, wharf, seawall, boat ramp, beach, intake, or outfall structure on a stream, river, or lake
   - Placement of any fill, rip rap, or similar material in a stream, river channel, or lake
   - Construction of a dam across any waterway
   - Placement of fill, construction of levees, roadways, and bridges; and similar activities on a floodplain
   - Construction of buildings on a floodplain

   The joint application form and instructions are available on the Iowa DNR website (www.iowadnr.gov); search for "Sovereign Lands Construction Permit."

Stormwater Management Criteria

A. Minor and Major Design Storms

The concept of minor and major design storms is related primarily to the conveyance capacity design for storm sewer and surface drainage systems. Part 2C provides a discussion of rainfall/runoff analysis and the selection of the appropriate design storm for a particular component of the stormwater management system. The concept of the unified sizing criteria is covered in Part 2A. This discussion of minor and major design storms is related to the selection of the overbank flood protection (Qp), which is one of the five components of the unified sizing criteria.

Every urban area has two separate and distinct drainage systems, whether or not they are actually planned for and designed. One is the minor system corresponding to the minor (or ordinary) storm recurring at regular intervals, generally 2 to 10 years. The other is the major system corresponding to the major or extraordinary storm, generally the 100 year storm event. A 100 year storm event was selected as the design interval for the major storm because this is typically the largest event that can be reasonably estimated from the historical rainfall data available. In addition, designing to a level above the 100 year event becomes impractical considering the relative infrequency of the event and the substantial infrastructure required to control the runoff.

Since the effects and routing of stormwater for the major storm may not be the same for the minor storm, all storm drainage plans submitted for approval should show the routing path and effects of the major storm.

1. **Minor Storm Provisions:** The minor storm drainage system should be designed to provide protection against regularly recurring damage, to reduce street and stormwater conveyance maintenance costs, to provide an orderly urban drainage system, and to provide convenience and protection to the urban residents. Storm sewer systems consisting of underground piping, natural drainage ways, and other required appurtenances should be considered as part of the minor storm drainage system.

2. **Major Storm Provisions:** The major storm drainage system should be designed to reduce the risk of substantial damage to the primary structure from storm runoff expected from the major storm. The effects of the major storm on the minor drainage system should be noted.

3. **Extreme Storm Provisions:** It is recognized that extreme storms, greater than a 100 year event, will occur; however, fully controlling storms of this magnitude is deemed economically unfeasible and impractical. While some level of damage from these extreme storm events is both likely and acceptable, their effect must be considered and provisions made to prevent widespread devastation and loss of life. This is especially true for detention basins, ponds, and other retention structures that have the potential for overtopping or catastrophic failure leading to downstream flash flooding.

B. Design Frequencies for Conveyance Facilities

Design storms for drainage facilities are described below. A minimum cleaning velocity of 2 ft/s should be used for the 2 year storm and 3 ft/s for the design storm. When detention or overland flow provisions for storms greater than 10 years are not available, regardless of the street system, the 100 year or greater storm is required for the design to minimize impact to private properties.

1. **Intakes:** Intakes should have a minimum capacity to convey the 5 year storm under developed conditions for local streets and minor collectors during the peak flow rate. The Engineer may require 10 year frequency for intakes for major collectors, arterials, expressways, and freeways.

2. **Storm Sewers:** Storm sewers should have capacity to convey a 5 year storm under developed conditions within the pipe for local streets and minor collectors. The Engineer may require 10 year frequency for storm sewers for major collectors, arterials, expressways, and freeways. Provisions should be made for the 100 year storm, greater in critical areas, when overland flow is not allowed or available to prevent damaging private property. Storm and/or surface water conveyance easements should be provided to the Jurisdiction.

3. **Footing Drains:** For those storm sewers that will handle footing drains, the following discharge (Q) values should be used.

   a. For less than 50 houses, Q = 5.0 gpm per house.

   b. For greater than 50 houses, Q = 250 gpm plus 2.5 gpm per house for each additional house over 50.

4. **Culverts:** Culverts should have capacity to convey the following.

   a. 10 year storm without the headwater depth exceeding the diameter of the culvert.

   b. 50 year storm without the headwater depth exceeding 1 foot over the top of the culvert.

   c. 100 year storms should be conveyed through the culvert without the headwater depth exceeding 1 foot below the low point of the roadway/embankment, unless there are other, more restrictive elevations.

   d. For culverts that drain areas over 2 square miles, the Iowa DNR rules and regulations will apply.

5. **Ditches:** Ditches should have capacity to convey a 50 year storm within the ditch banks. Provisions should be made for the 100 year storm to flow overland within the flowage easement. Surface water flowage easements should be provided to the Jurisdiction for all designed drainageways. For ditches that drain areas over 2 square miles, the Iowa DNR rules and regulations will apply.

6. **Detention Basins:** Detention basins should have the capacity to retain a 100 year storm at critical duration or safely pass the 100 year discharge over an auxiliary spillway. The top of any detention embankments should be a minimum of 1 foot above the 100 year ponding elevation. Iowa DNR approval may be required when the detention basin embankment and ponding volumes meet certain thresholds for embankment height with permanent and/or temporary storage. See the Iowa Administrative Code 567, Chapter 71, 71.3 (Dams) for specific approval criteria.

C. Street Flow Criteria

1. Street Capacity for Minor Storms:

a. Pavement encroachment for minor design storm should not exceed the limitations set forth in Table 2A-3.01.

Table 2A-3.01: Allowable Pavement Encroachment and Depth of Flow for Minor Storm Runoff

Street Classification	Maximum Encroachment[1]
Local	No curb overtopping. Flow may spread to crown of street.
Collector/Minor Arterial	No curb overtopping. Flow spread must not encroach to within 8 feet of the centerline of a two-lane street. The flow spread for more than two-lane streets must leave the equivalent of two 12 foot driving lanes clear of water; one lane in each direction. For one-way streets, a single 12 foot lane is allowed.
Major Arterials (4 lanes or greater)	No curb overtopping. Flow spread must not exceed 10 feet from the face of the curb of the outside lane. The flow spread for streets with more than two-lanes must leave the equivalent of two 12 foot driving lanes clear of water; one lane in each direction. For one-way streets, two 12 foot lanes are required. For special conditions, when an intake is necessary in a raised median, the flow spread should not exceed 4 feet from the face of the median curb for an inside lane.

[1] Where no curbing exists, encroachment should not extend past property lines.

b. The storm sewer system will commence upstream from the point where the maximum allowable encroachment occurs. When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity for a particular encroachment will be computed using the modified Manning's formula for flow in a small triangular channel as shown in Section 2B-3, Figure 2B-3.01. An "n" value of 0.016 will be used unless special considerations exist.

2. **Street Capacity for Major Storms:** The allowable depth of flow and inundated area for the major design storm should not exceed the limitations set forth in Table 2A-3.02.

Table 2A-3.02: Allowable Pavement Encroachment and Depth of Flow for Major (100 Year) Storm Runoff

Street Classification	Allowable Depth and Ponded Area
Local and Collector	The ponded area should not exceed the street right-of-way and the depth of water above the street crown should not exceed 6 inches. There may be situations where other restrictions are necessary.
Major and Minor Arterial	A 12 foot lane is the minimum travel lane to be passable in the center of the street.

3. **Cross-street Flow:** Cross-street flow (called cross-pan) can occur by two separate means. One is runoff that has been flowing in a gutter and then flows across the street to the opposite gutter or inlet. The second case is flow across the crown of the street when the conduit capacity beneath the street is exceeded. If the inundated area exceeds the street right-of-way, flow easements must be obtained. The maximum allowable cross-street flow depth based on the worst condition should not exceed the limitation stipulated in Table 2A-3.03.

Table 2A-3.03: Allowable Cross-street Flow

Street Classification	Initial Design Storm Runoff	100 Year Design Storm Runoff
Local	6 inch depth at crown or in cross-pan	9 inch depth at crown or in cross-pan
Collector	Where cross-pans are allowed, depth of flow or in cross-pan should not exceed 3 inches	6 inch depth at crown
Arterial	None	3 inch or less over crown

D. References

Flood Plain Development. Title V, Iowa Administrative Code 567. Chapter 71.3.

Project Drainage Report

A. Purpose

The purpose of the project drainage report is to identify and propose specific solutions to stormwater runoff and water quality problems resulting from existing and proposed development. The report must include adequate topographic information (pre- and post-development) to verify all conclusions regarding offsite drainage. Unless known, the capacity of downstream drainage structures must be thoroughly analyzed to determine their ability to convey the developed discharge.

The drainage report and plan will be reviewed and approved by the Jurisdictional Engineer prior to preparation of final construction drawings. Approval of these preliminary submittals constitutes only a conceptual approval and should not be construed as approval of specific design details. The Project Engineer may be required by law to submit the drainage report and plan to the Iowa DNR and/or USACE. An application for a permit to construct will follow the Iowa DNR and NPDES applicable permit requirements and USACE rules and regulations, and the application will be the responsibility of the Project Engineer.

B. Instructions for Preparing Report

1. Include a cover sheet with project name and location, name of firm or agency preparing the report, Professional Engineer's signed and sealed certification, and table of contents. Number each page of the report.

2. Perform all analyses according to the intent of professionally recognized methods. Support any modifications to these methods with well documented and industry accepted research.

3. It is the designer's responsibility to provide all data requested. If the method of analysis (for example, a computer program) does not provide the required information, then the designer will select alternative or supplemental methods to ensure the drainage report is complete and accurate.

4. Acceptance of a drainage report implies the Jurisdiction concurs with the project's overall stormwater management concept. This does not constitute full acceptance of the improvement plans, alignments, and grades, since constructability issues may arise in plan review.

5. Use all headings listed in the contents (Section 2A-4, C). A complete report will include all the information requested in this format. If a heading listed does not apply, include the heading and briefly explain why it does not apply. Include additional information and headings as required to develop the report.

6. This manual does not preclude the utilization of methods other than those referenced, nor does it relieve the designer of responsibility for analysis of issues not specifically mentioned.

C. Contents

The following information contains summaries for hydrology and detention (see Tables 2A-2.01, 2A-2.02, and 2A-2.03), as well as design considerations for the preparation of project drainage reports. They are provided as a minimum guide and are not to be construed as the specific information to be supplied on every project drainage report, and other information may be required. Existing and proposed conditions for each development will require analysis unique to that area.

1. Site Characteristics:

a. Pre-development Conditions: Describe pre-developed land use, topography, drainage patterns (including overland conveyance of the 100 year storm event), storm sewer, ditches, and natural and man-made features. Describe ground coverage, soil type, and physical properties, such as hydrologic soil group and infiltration. If a geotechnical study of the site is available, provide boring logs and locations in the appendix of the report. If a soil survey was used, cite it in the references.

For the pre-development analysis where the area is rural and undeveloped, a land use description reflecting current use is typical; however, the jurisdiction may apply more stringent requirements due to downstream drainage conditions. In addition, some jurisdictions require use of pre-settlement (meadow) conditions for all development. The jurisdiction should be contacted to determine what pre-development conditions are required.

b. Post-development Conditions: Describe post-developed land use and proposed grading, change in percent of impervious area, and change in drainage patterns. If an existing drainage way is filled, the runoff otherwise stored by the drainage way will be mitigated with stormwater detention, in addition to the post-development runoff.

c. Contributing Off-site Drainage: Describe contributing off-site drainage patterns, land use, and stormwater conveyance. Identify undeveloped contributing areas with development potential and list assumptions about future development runoff contributed to the site.

d. Floodways, Floodplains, and Wetlands: Identify areas of the site located within the floodway or floodplain boundaries as delineated on Flood Insurance Rate Maps, or as determined by other engineering analysis. Identify wetland areas on the site, as delineated by the National Wetlands Inventory, or as determined by a specific wetland study.

e. Pre-development Runoff Analysis:

1) **Watershed Area:** Describe overall watershed area and relationship between other watersheds or sub-areas. Include a pre-development watershed map in the report appendix.
2) **Time of Concentration:** Describe method used to calculate the time of concentration. Describe runoff paths and travel times through sub-areas. Show and label the runoff paths on the pre-development watershed map.
3) **Precipitation Model:** Describe the precipitation model and rainfall duration used for the design storm. Typical models may include one or more of the following:
   a) NRCS Type-II Distribution.
   b) Huff Rainfall Distribution. Select the appropriate distribution based on rainfall duration.
   c) Frequency-Based Hypothetical Storm.
   d) Rainfall Intensity Duration Frequency (IDF) Curve.
   e) User-defined model based on collected precipitation data, subject to the Jurisdictional Engineer’s approval. Total rainfall amounts for given frequency and duration should

be obtained from Bulletin 71, “Rainfall Frequency Atlas of the Midwest” (see Section 2B-2). Bulletin 71 supersedes Technical Paper Number 40, “Rainfall Frequency Atlas of the United States.”

4) **Rainfall Loss Method:** List runoff coefficients or curve numbers applied to the drainage area. The Green-Ampt infiltration model may also be used to estimate rainfall loss by soil infiltration.
5) **Runoff Model:** Describe method used to project runoff and peak discharge. Typical models are as follows:
   a) Use the Rational Method for drainage areas up to 40 acres, and where flow routing is not required. Often used in storm sewer design. See Section 2B-4 for explanation of limitations.
   b) As an alternative to the Rational Method, the SCS (NRCS) Peak Flow Method may be used.
   c) For drainage areas where flow routing is required, use one of the following methods:
      - TR-55 Tabular Hydrograph Method (WIN-TR-55)
      - TR-20 Model (Computer Program for Project Formulation Hydrology).
      - Routines contained in HEC-1 or HEC-HMS computer models
      - Regression Equations and other hydrologic models approved by the Jurisdiction
   d) TR-20 Methods are not recommended for small drainage areas less than 20 acres.
6) **Summary of Pre-development Runoff:** Provide table(s) including drainage area, time of concentration, frequency, duration, peak discharge, routing, and accumulative flows at critical points where appropriate.

2. Post-development Runoff Analysis:

a. Watershed Area: Describe overall watershed area and sub-areas. Discuss if the post-development drainage area differs from the pre-development drainage area. Include a post-development watershed map.

b. Time of Concentration: The method used will be the same as used in the pre-development analysis. Describe change in times of concentration due to development (i.e. change in drainage patterns). Show and label the runoff paths on the post-development watershed map.

c. Precipitation Model: Storm event, total rainfall, and total storm duration will be the same as used for the pre-development model.

d. Rainfall Loss Method: Method will be the same as pre-development analysis. Describe the change in rainfall loss due to development.

e. Runoff Model: The runoff method will be the same as used in the pre-development analysis, except for variables changed to account for the developed conditions.

f. Summary of Post-development Runoff:

1) Provide table(s) including drainage area, time of concentration, frequency, duration, and peak discharge. Summarize in narrative form the change in hydrologic conditions due to the development. Provide a runoff summary using Tables 2A-2.01 and 2A-2.02.
2) Post-developed discharge should take into account any upstream offsite detention basins and undeveloped offsite areas assumed to be developed in the future with stormwater detention.

3) Calculate the allowable release rate from the site, based on two conditions:
   a) After development, the release rate of runoff for rainfall events having an expected return frequency of 2 years and 5 years should not exceed the existing, pre-developed peak runoff rate from those same storms.
   b) For rainfall events having an expected return frequency of 10 years to 100 years, inclusive, the rate of runoff from the developed site should not exceed the existing, pre-developed peak runoff from a 5 year frequency storm of the same duration. The allowable discharge rate may be restricted due to downstream capacity. Include this calculation in the Executive Summary.
4) Describe assumptions made for portions of the drainage area that are not included in the current development area.

3. Stormwater Conveyance Design:

a. Design Information References: At a minimum, all stormwater conveyances will be designed according to this manual. The following references may be used for supplemental design information:

1) Federal Highway Administration (2009) *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22, Washington D.C.
2) Federal Highway Administration (2005) *Design of Roadside Channels with Flexible Linings*. Hydraulic Engineering Circular No. 15, Washington D.C.
3) Federal Highway Administration (2005) *Hydraulic Design of Highway Culverts*. Hydrologic Design Series Number 5, Washington D.C.
4) US Geological Survey (1968) *Measurement of Peak Discharge at Culverts by Indirect Methods*. Book 3, Applications of Hydraulics, Washington D.C.
5) American Society of Civil Engineers (1993) *Design and Construction of Urban Stormwater Management Systems* Manual of Practice No. 77, New York, N.Y.

b. Storm Sewer:

1) List design criteria, including storm event and runoff model. Describe the hydraulic grade line and whether pressure flow or surcharging is possible. Provide a graphic of the hydraulic grade line.
2) List design criteria for intake size and spacing. Describe the anticipated gutter flow and spread at intakes.
3) List any special considerations for subdrain design, such as high water tables.
4) Provide tables of storm sewer (inlet and pipe) and intake design data.
5) Water spread on the street for intake design year and 100 year elevation in all streets in which the curb is overtopped.

c. Culverts:

1) Describe culvert capacity, inlet or outlet control conditions, and estimated tailwater and headwater. Determine if 100 year or lesser storm event will flood roadway over culvert.
2) Sketch a contour of the 100 year headwater elevation on a topographic map and/or grading plan. This delineated 100 year flood elevation is used to determine drainage easement and site grading requirements.

d. Open Channel Flow - Swales and Ditches:

1) Describe swale and ditch design. State the assumed Manning's roughness coefficients. State the anticipated flow velocity and whether it exceeds the permissible velocity based on soil types and/or ground coverage. If the permissible velocity is exceeded, describe channel lining or energy dissipation.

2) Discuss design calculations. Depending on the complexity of the design, these may range from a single steady-state equation (i.e. Manning's) to a step calculation including several channel cross-sections, culverts, and bridges.
3) Discuss the overall grading plan in terms of controlling runoff along lot lines and preventing runoff from adversely flowing onto adjacent lots.
4) The limits of swale and ditch easements will be established based upon the required design frequency. This includes 100 year overflow easements from stormwater controlled structures.

e. Storm Drainage Outlets and Downstream Analysis:

1) Discuss soil types, permissible and calculated velocity at outlets, energy dissipater design, and drainage impacts on downstream lands. Provide calculations for the energy dissipater dimensions, size, and thickness of rip rap revetment (or other material) and filter layer.
2) Include a plan and cross-sections of the drainage way downstream of the outlet, indicating the flow line slope and bank side slopes. Identify soil types on the plan.
3) Perform downstream analysis. The downstream analysis will show what impacts, if any, a project will have on the drainage systems downstream of the project site. The analysis consists of three elements: review of resources, inspection of the affected area, and analysis of downstream effects.
   a) During the review of resources, review any existing data concerning drainage of the project area. This data will commonly include area maps, floodplain maps, wetland inventories, stream surveys, habitat surveys, engineering reports concerning the entire drainage basin, known drainage problems, and previously completed downstream analyses.
   b) Physically inspect the drainage system at the project site and downstream of it. During the inspection, investigate any problems or areas of concern that were noted during the review of resources. Identify any existing or potential capacity problems in the drainage system, flood-prone areas, areas of channel destruction, erosion and sediment problems, or areas of significant destruction of natural habitat.
   c) Analyze the information gathered during the review of resources and field inspection, to determine if the project will create any drainage problems downstream or will make any existing problems worse. Note there are situations that even when minimum design standards are met the project will still have negative downstream impacts. Whenever this situation occurs, mitigation measures must be included in the project to correct for the impacts.

f. Hydraulic Model: If the design warrants hydraulic modeling, state the method used. Typical modeling programs include:

1) HEC-RAS - River Analysis Systems
2) HEC-2 - Water Surface Profiles
3) SWMM - Storm Water Management Model
4) WSPRO - Water Surface Profiles
5) HY-8 - Hydraulic Design of Highway Culverts
6) Other commercial or public domain programs approved by the Jurisdiction.

4. Stormwater Facilities Design:

a. Design Standards: All stormwater management facilities will be designed according to these design standards at a minimum. The following references may provide helpful design information for stormwater detention and water quality issues.

1) *Urban Drainage Design Manual* (Hydraulic Engineering Circular No. 22).

2) Final report of the Task Committee on Stormwater Detention Outlet Control Structures
3) *Design and Construction of Urban Stormwater Management Systems*. Manual of Practice No.77
4) *Urban Runoff Quality Management*. Manual of Practice No. 87
5) *Stormwater Detention for Drainage, Water Quality, and CSO Management*
6) Urban Runoff: *Water Quality Solutions.* Special Report No. 61

b. Detention Basin Location: Describe basin site. Discuss existing topography and relationship to basin grading. Determine if construction will be affected by rock deposits. Also determine if a high water table precludes basin storage. Floodplain locations should be avoided.

c. Detention Basin Performance: The following summarize the recommended detention requirements. The Jurisdiction may adopt different standards or modify these requirements on a case by case basis depending on existing drainage conditions, flooding problems, or future development. The designer should verify the detention requirements with the Jurisdiction for each proposed project.

1) After development, the release rate of runoff for rainfall events having an expected return frequency of 2 years should not exceed the existing, pre-developed peak runoff rate from that same storm.
2) For rainfall events having an expected return frequency of 5, 10, 25, 50, and 100 years, the rate of runoff from the developed site should not exceed the existing, pre-developed peak runoff rate from a 5 year frequency storm of the same duration unless limited by downstream conveyance. Provide a table summarizing these release rates. Also provide a stage-storage-discharge table. These tables are also to be shown in Table 2A-4.03. State the minimum freeboard provided and at what recurrence interval the basin overtops.
3) Discuss the effects on the overall stormwater system by detention basins in contributing offsite areas. If contributing offsite areas are presently undeveloped, discuss assumptions about future development and stormwater detention.
4) Calculate the basin overflow release rate. This equals the onsite 100 year post-developed peak discharge plus the contributing offsite 100 year post developed peak discharge. Include this calculation with Table 2A-4.03.

d. Detention Basin Outlet:

1) The single-stage outlet (i.e. one culvert pipe) is not recommended because of its inability to detain post-developed runoff from storms less than the 5 year interval. In many cases, runoff from storm events less than the 5 year recurrence interval has created erosion and sedimentation problems downstream of the detention basin.
2) A more desirable outlet has two or more stages. An orifice structure serves to detain runoff for water quality purposes and release runoff for low-flow events of a 2 year storm. Greater storm events are usually discharged by a separate outlet.
3) Discuss the basin outlet design in terms of performance during low- and high-flows, and downstream impact.

e. Spillway and Embankment Protection:

1) Design the spillway for high flows using weir and/or spillway design methods. The steady-state open channel flow equation is not intended for use in spillway design.
2) Describe methods to protect the basin during overtopping flow.

f. TR-55 Design Limitations: TR-55 includes a method for estimating required storage volume based upon peak inflow, peak outflow, and total runoff volume. This method may result in storage errors of 25% and should not be used in final design. The detention basin size in final design should be based upon actual hydrograph routing utilizing methods such as WINTR-55 or TR-20.

5. **Permits:** Indicate what permits have been applied for and received. Submit Iowa DNR approval letter and report for sites affecting unnumbered A-zones, as delineated on Flood Insurance Rate Maps.

6. **References:** Provide a list of all references cited, in bibliographical format.

7. **Appendix:** Drawings and calculations in the Appendix should include, but are not limited to, the following items.

   a. **Drawings:**
      1) A preliminary plat (pre-and post-topography) may be used to show the proposed development. Minimum scale of 1 inch = 500 feet or larger to ensure legibility should be used for all drainage areas. (Drawings no larger than 24 inches by 36 inches should be inserted in 8 1/2 inch by 11 inch sleeves in the back of the bound report). The plat is to show street layout and/or building location on a contour interval not to exceed 2 feet. The map must show on- and off-site conditions. Label flow patterns used to determine times of concentration.
      2) Drainage plans (preliminary plat or topography map) must extend a minimum of 250 feet from the edge of the proposed preliminary plat boundary, or a distance specified by Jurisdiction. The limits of swale and ditch easements should be established based upon the required design frequency. This includes 100 year overflow easements from stormwater controlled structures.
      3) Overall drainage basin (or sub-basin) and location of proposed site within the basin.
      4) Soil map or geotechnical information.
      5) Location and elevations of jurisdictional benchmarks. All elevations should be on jurisdictional datum.
      6) Proposed property lines (if known).
      7) If the preliminary plat does not include proposed grades, submit a grading and erosion control plan showing existing and proposed streets, names, and approximate grades.
      8) Existing drainage facilities and structures, including existing roadside ditches, drainageways, gutter flow directions, culverts, etc. All pertinent information such as size, shape, slope location, 100 year flood elevation, and floodway fringe line (where applicable) should also be included to facilitate review and approval of drainage plans.
      9) Proposed storm sewers and open drainageways, right-of-way and easement width requirements, 100 year overland flow easement, proposed inlets, manholes, culverts, erosion and sediment control, water quality (pollution) control and energy dissipation devices, and other appurtenances.
      10) Proposed outfall point for runoff from the study area.
      11) The 100 year flood elevation and major storm floodway fringe (where applicable) are to be shown on the plans, report drawings, and plats (preliminary and final). In addition, the report should demonstrate that the stormwater system has adequate capacity to handle a 100 year storm event, or provisions are made for overland flow.
      12) Show the critical minimum lowest opening elevation of a building for protection from major and minor storm runoff. This elevation is to be reviewed with the Jurisdiction to confirm if previous changes were made to the minimum lowest opening elevation for major storm event.

   b. **Calculations:**
      1) Determine runoff coefficients and curve numbers
      2) Determine times of concentration

3) Calculations for intake capacity, sewer design, and culvert design
4) Peak discharge calculations - show results in tabular format and pre- and post-developed hydrographs
5) Detention basin design - show tabular stage-storage-discharge results and inflow/outflow hydrographs
6) Detention basin outlet design
7) Open channel flow calculations
8) Erosion protection design

Table 2A-4.01: Hydrology Summary

	Area 1				**Area 2**			
	Onsite		***Offsite***		***Onsite***		***Offsite***	
	Pre	***Post***	***Pre***	***Post***	***Pre***	***Post***	***Pre***	***Post***
Size (Acres)								
Predominant Land Use								
Watershed Length								
Time of Concentration								
Runoff Coefficient								
Runoff (Q)								
2 yr								
5 yr								
10 yr								
25 yr								
50 yr								
100 yr								

Table 2A-4.02: Hydrology Summary (Critical Points)

Design Flows	**Critical Point 1**	**Critical Point 2**	**Critical Point 3**	**Critical Point 4**
2 yr				
5 yr				
10 yr				
25 yr				
50 yr				
100 yr				

Table 2A-4.03: Detention Summary

Detention Basin
A. Inlet Design Storm Frequency: _____________
B. Outlet Design Storm Frequency: _____________

Standard Release Rate
A. Allowable release rate: _____ cfs
B. Offsite (developed) rate: _____ cfs
Total Release: _____ cfs

Overflow Release Rate
A. Onsite pre-developed (100 yr) ______ cfs
B. Offsite developed (100 yr)* ______ cfs
Total Release: ______ cfs

Structures
A. Inflow Structure: _____________
B. Outflow Structure: _____________

	Stage**	Storage (ac-ft)	Inflow (cfs)	Outflow (cfs)	Comments
1					
2					
3					
4					
5					
6					
7					
8					
9					
10					

* Routed through basin ** Max. 1 foot interval

D. Computer Analysis

Hydraulic and hydrologic calculations can be iterative and tedious. Due to the time consuming and repetitive nature of these calculations, a high probability of error exists when performing the calculations by hand. For these reasons, the use of computer programs for analysis is both allowed and encouraged.

A variety of both proprietary and publicly available software programs are available. While this manual sets no standards as to the brand or version of analysis software allowed, the following tables list programs utilized in Iowa. Table 2A-4.04 provides a partial list of hydrologic models meeting the minimum requirements of the National Flood Insurance Program. Table 2A-4.05 lists additional programs that are used in Iowa.

Before using computer software, the user should thoroughly understand the theory behind the analysis method being used, understand the impact that various inputs have on the results, and verify that the program yields expected results for given inputs.

Table 2A-4.04: Hydrologic Models Meeting the Minimum Requirements of NFIP

Name	Version	Developer (available from)	Public Domain or Proprietary
One Dimensional Steady Flow Models			
Culvert Master	2.0 (Sept. 2000) & up	Bentley Systems	Proprietary
HEC-HMS	v. 1.1 and up	USACE	Public Domain
HEC-RAS	3.1.1 and up	USACE	Public Domain
HY-8	4.1 (Nov. 1992) & up	FHWA	Public Domain
PondPak	v. 8 (May 2002) & up	Bentley Systems	Proprietary
QUICK-2	1.0 & up	FEMA	Public Domain
SWMM 5	v. 5.0.005 (May 2005) & up	US EPA	Public Domain
StormCAD	4 (June 2002) & up	Bentley Systems	Proprietary
TR-20	Win 1.00	USDA - NRCS	Public Domain
WinTR-55	1.0.08 (Jan. 2005)	USDA - NRCS	Public Domain
WSPGW	12.96 (Oct. 2000) & up	LA Flood Control Dist.	Proprietary
WSPRO	June 1988 & up	USGS / FHWA	Public Domain
XP-STORM	10.0 (May 2006)	XP Software	Proprietary
XP-SWMM	8.52 & up	XP Software	Proprietary
One Dimensional Unsteady Flow Models			
FLDWAV	Nov. 1998	Nat. Weather Svc., NOAA	Public Domain
HEC-RAS	3.1.1 and up	USACE	Public Domain
SWMM 5	v. 5.0.005 (May 2005) & up	US EPA	Public Domain
XP-STORM	10.0 (May 2006)	XP Software	Proprietary
XP-SWMM	8.52 & up	XP Software	Proprietary

Source: FEMA website

Table 2A-4.05: Other Hydraulic Software Utilized in Iowa

Name	Version	Developer (available from)	Public Domain or Proprietary
Iowa DOT Bridge Backwater	v. 2	Iowa DOT	Public Domain
Iowa DOT Culvert	v. 1	Iowa DOT	Public Domain
SITES (dam hydraulics)	v. 2005	Kansas USDA	Public Domain

E. References

Federal Emergency Management Agency (FEMA). *Hydrologic Models: Determination of Flood Hydrographs.*
Available at: http://www.fema.gov/national-flood-insurance-program-flood-hazard-mapping/hydrologic-models-meeting-minimum-requirement. Accessed: October 2012.

Federal Emergency Management Agency (FEMA). *Hydraulic Models: Determination of Water-Surface Elevations for Riverine Analysis.*
Available at: http://www.fema.gov/numerical-models-meeting-minimum-requirements-national-flood-insurance-program. Accessed: January 2016.

General Information for Urban Hydrology and Runoff

A. Introduction

Urban stormwater hydrology includes the information and procedures for estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems for conveyance of surface runoff and structural stormwater controls for quality and quantity. In the hydrologic analysis of a site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that must be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

The typical hydrologic processes of interest in urban hydrology are related to:

- Precipitation and losses (rainfall abstractions)
- Determination of peak flow rate
- Determination of total runoff volume
- Runoff hydrograph (flow vs. time)
- Stream channel hydrograph routing and combining of flows
- Reservoir (storage) routing

The practice of urban stormwater hydrology is not an exact science. While the hydrologic processes are well-understood, the necessary equations and boundary conditions required to solve them are often quite complex. In addition, the required data is often not available. There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for a site or drainage subbasin; the methods presented in this section have been selected to support hydrologic site analysis for the design methods and procedures included in this manual:

- Rational method
- NRCS Peak Flow method (SCS Curve Number)
- NRCS Urban Hydrology for Small Watersheds (TR-55, 1986; WinTR-55, 2003)
- U.S. Geological Survey (USGS) regression equations

These methods have been included since the applications are well-documented in urban stormwater hydrology design practice, and have been verified for accuracy in duplicating local hydrologic estimates for a range of design storms. The applicable design equations, nomographs, and computer programs are readily available to support the methods.

Table 2B-1.01 lists the hydrologic methods and circumstances for their use in various analysis and design applications. Table 2B-1.02 includes some limitations on the use of several of the methods.

1. **Rational Method:** The Rational method is recommended for small, highly-impervious drainage areas, such as parking lots, roadways, and developed areas draining into inlets and gutters.

   The Rational method (see Section 2B-4) may be used in both the minor and major storm runoff computations for relatively uniform basins in land use and topography, which generally have less than 40 acres. The averaging of runoff coefficients for significantly different land uses should be minimized where possible. For basins that have multiple changes in land use and topography, or are larger than 40 acres, or both; the design storm runoff should be analyzed by other methods. These basins should be broken down into subbasins of like uniformity and routing methods applied to determine peak runoff at specified points.

   If the Rational method is not used, TR-55, Urban Hydrology for Small Watersheds (NRCS) (see Section 2B-5), may be used for drainage areas up to 2,000 acres. For areas larger than 2,000 acres, TR-20 or an approved alternative may be used. When computer programs are used for design calculation, it is important to understand the assumptions and limits for the maximum and minimum drainage area or other limits before it is selected.

2. **NRCS Peak Flow Method:** The NRCS Peak Flow method (also known as the SCS Curve Number method) may be utilized as an alternative to the Rational method. The NRCS Peak Flow method (Section 2B-5) can be utilized for larger drainage areas (up to 2,000 acres). Like the Rational method, use of this method should be limited to basins with relatively homogeneous curve numbers and an overall curve number greater than 40.

   The NRCS Peak Flow method does not contain an expression for time; therefore, the equation does not account for storm intensity or duration. This prohibits the use of this method for calculating runoff from a specific storm event (e.g. 5 year, 1 hour storm).

3. **Modified Rational Method:** The Modified Rational method is one of the simplest methods for developing a hydrograph and routing a storm. Due to its simplicity, the Modified Rational method is also one of the least accurate routing methods. However, this method can be sufficient for routing storms from small drainage areas (up to 5 acres) with significantly varied runoff coefficients.

4. **NRCS Tabular Hydrograph Method (TR-55):** The Tabular Hydrograph method described in the NRCS' Urban Hydrology for Small Watersheds (TR-55) is applicable to non-homogeneous areas beyond the limitations of the Rational method. This method has wide application for existing and developing urban watersheds and can be utilized for estimating the effects of land use change as well as the effects of proposed structures. The method is limited to drainage areas less than 2,000 acres with a time of concentration less than or equal to 2 hours.

5. **Other Methods:** For drainage areas larger than 2,000 acres, or for situations where the methods described above are not appropriate, TR-20, HEC-1, HEC-HMS, or other approved alternatives may be used.

Table 2B-1.01: Applications of Hydrologic Methods

Method	Rational Method	NRCS Peak Flow	Modified Rational	NRCS TR-55
Channel protection volume (CPv)				✓
Overbank flood protection (Qp_5)				✓
Extreme flood protection (Qf)				✓
Storage facilities			✓	✓
Outlet structures				✓
Gutter flow and inlets	✓	✓		
Storm sewer piping	✓	✓		✓
Culverts	✓	✓		✓
Small ditches	✓	✓		✓
Open channels	✓	✓		✓
Energy dissipation				✓

Small storm hydrology and low impact development (LID) methods (utilized for water quality based design) as well as water balance calculations (utilized for permanent pond / wet detention design) are discussed in the Iowa Stormwater Management Manual (ISMM).

Table 2B-1.02: Limitations of Hydrologic Methods

Method	Size Limitations	Comments
Rational	40 acres	Method can be used for drainage areas with similar land uses for estimating peak flows and for the design of small site or subdivision storm sewer systems. *Should not be used for storage design.*
NRCS Peak Flow	0 to 2,000 acres	Method can be used for estimating peak flows for storm sewer or channel design. *Should not be used for storage design.*
Modified Rational	0 to 5 acres	Method can be used for estimating peak flows and developing simple hydrographs from small drainage areas with significantly different runoff coefficients.
NRCS TR-55	0 to 2,000 acres	Method can be used for estimating peak flows and developing hydrographs for all design applications. Can be used for low-impact development hydrologic analysis.

B. Definitions

Depression Storage: Depression storage is the natural depressions within the ground surface and landscape that collect and store rainfall runoff, either temporarily or permanently.

Hydrograph: A hydrograph is a graph of the time distribution of runoff from a watershed.

Hyetograph: A hyetograph is a graph of the time distribution of rainfall over a watershed [rainfall intensity (in/hr) or volume vs. time].

Infiltration: Infiltration is the process through which precipitation enters the soil surface and moves through the upper soil profile.

Interception: Interception is the storage of rainfall on foliage and other intercepting surfaces, such as vegetated pervious areas, during a rainfall event.

Peak Discharge: The peak discharge (peak flow) is the maximum rate of flow of water passing a given point during or after a rainfall event (or snowmelt).

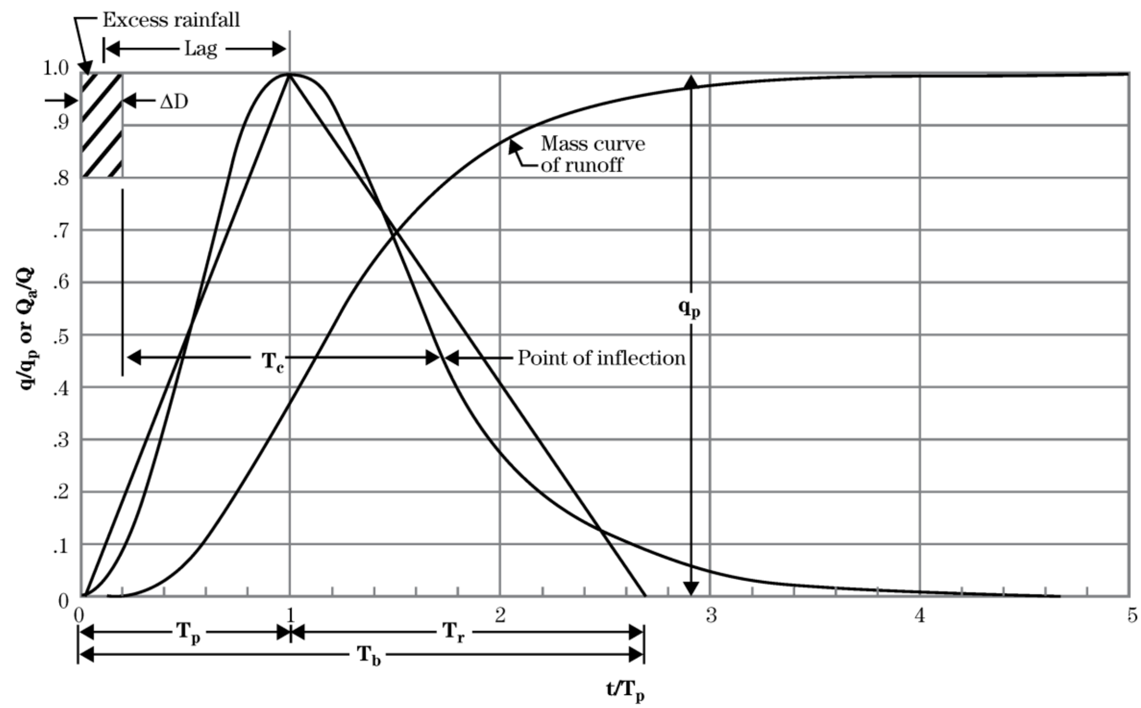
Rainfall Excess: After interception, depression storage, and infiltration have been satisfied, rainfall excess is the remaining water available to produce runoff.

Runoff Volume: The runoff volume represents the volume of rainfall excess generated from the watershed area. The runoff volume is often expressed in watershed-inches or acre-feet. The runoff volume for a rainfall event can also be represented by the area under the runoff portion of the hydrograph

Travel Time (T_t) and Time of Concentration (T_c): Travel time is the time it takes for water to travel from one location to another in a watershed. T_t is a component of the time of concentration, T_c, which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system

Unit Hydrograph: The hydrograph resulting from 1 inch of rainfall excess generated uniformly over the watershed, at a uniform rate, for a specified period of time. There are several types of unit hydrographs. The use of unit hydrographs to create direct runoff hydrographs is discussed in more detail in Section 2B-5.

Figure 2B-1.01: NRCS Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph



Source: NRCS NEH Part 630, Chapter 16A

C. References

USDA Natural Resource Conservation Service. *National Engineering Handbook - Part 630*. Chapter 16: Hydrographs. 2007.

Rainfall and Runoff Periods

A. Introduction

1. The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

    a. Duration (hours): Length of time over which rainfall (storm event) occurs.

    b. Depth (inches): Total amount of rainfall occurring during the storm duration.

    c. Intensity (inches per hour): Depth divided by the duration.

2. A design event is used as a basis for determining the requirements of new stormwater improvements or evaluating an existing project. It is presumed that the project will function properly if it can accommodate the design event at full capacity. For economic reasons, some risk of failure is allowed in selection of the design event. This risk is usually related to return period.

3. The frequency of a rainfall event is the average recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of exceedence probability or return period.

    a. Exceedence Probability: Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1 year.

    b. Return Period: Average length of time between events that have the same duration and volume.

    Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an probability of exceeding 0.01, and a return period of 100 years.

Table 2B-2.01: Chance of a Storm Equaling or Exceeding a Given Frequency During a Given Time Period

Return Period (years)	Time Period in Years					
	1	***5***	***10***	***25***	***50***	***100***
2	50%	97%	99.9%	99.9%	99.9%	99.9%
5	20%	67%	89%	99.6%	99.9%	99.9%
10	10%	41%	65%	93%	99%	99.9%
25	4%	18%	34%	64%	87%	98%
50	2%	10%	18%	40%	64%	87%
100	1%	5%	10%	22%	40%	63%

B. Rainfall Frequency Analysis

In April 2013, the National Oceanic and Atmospheric Administration (NOAA) released "Atlas 14: Precipitation-Frequency Atlas of the United States, Volume 8." Volume 8 of this publication covers the Midwestern States, including Iowa, and supersedes "Bulletin 71: Rainfall Frequency Atlas of the Midwest" (1992) as the most current precipitation data available.

The Atlas 14 results are provided through NOAA's Precipitation Frequency Data Server (http://hdsc.nws.noaa.gov/hdsc/pfds). Based upon user input, the online database generates a precipitation-frequency estimate (PFE) for an individual location from the historical records of approximately 280 precipitation recording stations across the State of Iowa.

The location-specific PFE attribute of Atlas 14 means that precipitation-frequency estimates could be generated for each community or even each individual project, resulting in hundreds or even thousands of PFE's across Iowa. This situation would be both inefficient for designers and impractical for reviewers.

To avoid this dilemma, regional intensity-duration-frequency (IDF) tables corresponding to the nine Iowa climatic sections in Bulletin 71 were developed. Utilizing Atlas 14, PFE's were obtained at each county seat. The county values within each climatic section were then averaged to represent the section as a whole. The resulting IDF values for each climatic section are provided in Tables 2B-2.02 through 2B-2.10 below.

Figure 2B-2.01: Climatic Sectional Codes for Iowa

1 - Northwest	4 - West Central	7 - Southwest
2 - North Central	5 - Central	8 - South Central
3 - Northeast	6 - East Central	9 - Southeast

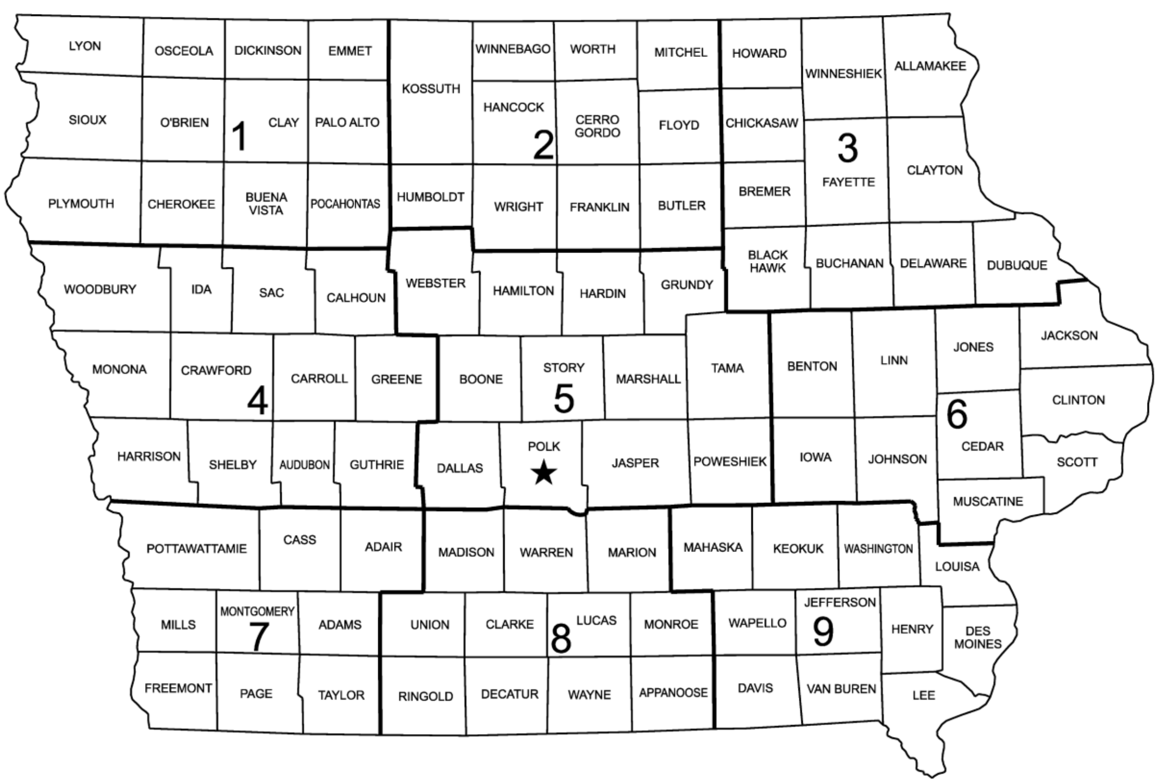
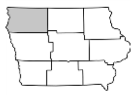
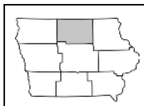


Table 2B-2.02: Section 1 - Northwest Iowa
Rainfall Depth and Intensity for Various Return Periods

	Return Period															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.39	4.69	0.46	5.53	0.57	6.92	0.67	8.11	0.80	9.69	0.91	10.9	1.01	12.1	1.25	15.0
10 min	0.57	3.43	0.67	4.06	0.84	5.07	0.98	5.92	1.18	7.09	1.33	8	1.48	8.91	1.84	11.0
15 min	0.69	2.78	0.82	3.29	1.03	4.12	1.20	4.82	1.44	5.77	1.62	6.50	1.81	7.24	2.24	8.98
30 min	0.97	1.94	1.15	2.30	1.44	2.89	1.69	3.38	2.02	4.05	2.28	4.56	2.54	5.08	3.15	6.30
1 hr	1.25	1.25	1.48	1.48	1.86	1.86	2.18	2.18	2.64	2.64	3.01	3.01	3.38	3.38	4.30	4.30
2 hr	1.53	0.76	1.80	0.90	2.27	1.13	2.68	1.34	3.26	1.63	3.74	1.87	4.23	2.11	5.45	2.72
3 hr	1.69	0.56	1.99	0.66	2.51	0.83	2.97	0.99	3.66	1.22	4.22	1.40	4.81	1.60	6.33	2.11
6 hr	1.95	0.32	2.3	0.38	2.91	0.48	3.47	0.57	4.32	0.72	5.04	0.84	5.81	0.96	7.84	1.30
12 hr	2.21	0.18	2.59	0.21	3.30	0.27	3.95	0.32	4.95	0.41	5.81	0.48	6.74	0.56	9.21	0.76
24 hr	2.51	0.10	2.92	0.12	3.67	0.15	4.39	0.18	5.50	0.22	6.46	0.26	7.50	0.31	10.3	0.43
48 hr	2.89	0.06	3.30	0.06	4.08	0.08	4.82	0.10	5.98	0.12	6.99	0.14	8.10	0.16	11.1	0.23
3 day	3.16	0.04	3.60	0.05	4.41	0.06	5.17	0.07	6.36	0.08	7.38	0.10	8.50	0.11	11.5	0.15
4 day	3.38	0.03	3.85	0.04	4.70	0.04	5.49	0.05	6.71	0.06	7.74	0.08	8.85	0.09	11.8	0.12
7 day	3.93	0.02	4.49	0.02	5.46	0.03	6.32	0.03	7.6	0.04	8.64	0.05	9.74	0.05	12.5	0.07
10 day	4.46	0.01	5.08	0.02	6.12	0.02	7.02	0.02	8.32	0.03	9.36	0.03	10.4	0.04	13.1	0.05

D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.03: Section 2 - North Central Iowa
Rainfall Depth and Intensity for Various Return Periods

	Return Period															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.39	4.69	0.46	5.53	0.57	6.93	0.68	8.18	0.83	9.96	0.95	11.4	1.07	12.9	1.39	16.6
10 min	0.57	3.44	0.67	4.04	0.84	5.07	0.99	5.98	1.21	7.29	1.39	8.35	1.57	9.45	2.03	12.2
15 min	0.69	2.79	0.82	3.28	1.03	4.12	1.21	4.87	1.48	5.92	1.69	6.79	1.92	7.68	2.48	9.93
30 min	0.99	1.98	1.16	2.33	1.47	2.94	1.73	3.47	2.11	4.23	2.42	4.85	2.75	5.50	3.56	7.13
1 hr	1.28	1.28	1.52	1.52	1.92	1.92	2.27	2.27	2.80	2.80	3.23	3.23	3.69	3.69	4.85	4.85
2 hr	1.58	0.79	1.87	0.93	2.37	1.18	2.82	1.41	3.49	1.74	4.04	2.02	4.63	2.31	6.14	3.07
3 hr	1.76	0.58	2.08	0.69	2.64	0.88	3.15	1.05	3.91	1.30	4.56	1.52	5.24	1.74	7.04	2.34
6 hr	2.06	0.34	2.42	0.40	3.07	0.51	3.67	0.61	4.6	0.76	5.38	0.89	6.22	1.03	8.45	1.40
12 hr	2.34	0.19	2.74	0.22	3.46	0.28	4.14	0.34	5.18	0.43	6.07	0.50	7.03	0.58	9.59	0.79
24 hr	2.65	0.11	3.06	0.12	3.83	0.15	4.55	0.18	5.67	0.23	6.63	0.27	7.68	0.32	10.4	0.43
48 hr	3.04	0.06	3.46	0.07	4.26	0.08	5.01	0.10	6.18	0.12	7.19	0.14	8.29	0.17	11.2	0.23
3 day	3.31	0.04	3.78	0.05	4.63	0.06	5.42	0.07	6.64	0.09	7.68	0.10	8.80	0.12	11.8	0.16
4 day	3.55	0.03	4.06	0.04	4.97	0.05	5.80	0.06	7.06	0.07	8.12	0.08	9.26	0.09	12.2	0.12
7 day	4.19	0.02	4.79	0.02	5.83	0.03	6.76	0.04	8.12	0.04	9.24	0.05	10.4	0.06	13.4	0.07
10 day	4.78	0.01	5.45	0.02	6.58	0.02	7.56	0.03	8.99	0.03	10.1	0.04	11.3	0.04	14.3	0.05

D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.04: Section 3 - Northeast Iowa

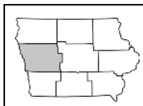
Rainfall Depth and Intensity for Various Return Periods

	Return Period															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.38	4.66	0.45	5.47	0.56	6.76	0.65	7.86	0.78	9.42	0.88	10.5	0.98	11.8	1.22	14.7
10 min	0.56	3.40	0.66	4.00	0.82	4.94	0.96	5.76	1.14	6.89	1.29	7.75	1.44	8.64	1.79	10.7
15 min	0.69	2.77	0.81	3.24	1.00	4.02	1.17	4.68	1.40	5.60	1.57	6.31	1.75	7.03	2.19	8.77
30 min	0.96	1.93	1.14	2.28	1.41	2.83	1.65	3.31	1.98	3.96	2.23	4.47	2.49	4.98	3.10	6.20
1 hr	1.25	1.25	1.47	1.47	1.85	1.85	2.17	2.17	2.64	2.64	3.01	3.01	3.39	3.39	4.34	4.34
2 hr	1.53	0.76	1.81	0.90	2.28	1.14	2.70	1.35	3.30	1.65	3.79	1.89	4.30	2.15	5.58	2.79
3 hr	1.71	0.57	2.01	0.67	2.55	0.85	3.03	1.01	3.74	1.24	4.32	1.44	4.94	1.64	6.55	2.18
6 hr	2.01	0.33	2.36	0.39	2.98	0.49	3.56	0.59	4.43	0.73	5.17	0.86	5.97	0.99	8.07	1.34
12 hr	2.32	0.19	2.69	0.22	3.38	0.28	4.02	0.33	5.02	0.41	5.86	0.48	6.79	0.56	9.25	0.77
24 hr	2.63	0.10	3.04	0.12	3.78	0.15	4.48	0.18	5.56	0.23	6.48	0.27	7.48	0.31	10.1	0.42
48 hr	3.00	0.06	3.44	0.07	4.23	0.08	4.98	0.10	6.12	0.12	7.10	0.14	8.15	0.16	10.9	0.22
3 day	3.28	0.04	3.73	0.05	4.56	0.06	5.32	0.07	6.49	0.09	7.48	0.10	8.56	0.11	11.4	0.15
4 day	3.53	0.03	4.00	0.04	4.85	0.05	5.64	0.05	6.84	0.07	7.86	0.08	8.95	0.09	11.8	0.12
7 day	4.17	0.02	4.72	0.02	5.70	0.03	6.58	0.03	7.87	0.04	8.95	0.05	10.1	0.06	13.0	0.07
10 day	4.76	0.01	5.38	0.02	6.45	0.02	7.39	0.03	8.77	0.03	9.90	0.04	11.0	0.04	14.0	0.05

D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.05: Section 4 - West Central Iowa

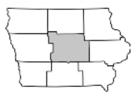
Rainfall Depth and Intensity for Various Return Periods



	Return Period															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.37	4.47	0.44	5.3	0.55	6.67	0.65	7.88	0.80	9.63	0.92	11.0	1.04	12.5	1.35	16.2
10 min	0.54	3.29	0.64	3.86	0.81	4.88	0.96	5.76	1.17	7.05	1.34	8.09	1.53	9.18	1.98	11.9
15 min	0.66	2.66	0.78	3.14	0.99	3.96	1.17	4.69	1.43	5.74	1.64	6.58	1.86	7.46	2.42	9.68
30 min	0.95	1.91	1.13	2.26	1.43	2.87	1.69	3.39	2.08	4.16	2.39	4.78	2.71	5.42	3.53	7.06
1 hr	1.24	1.24	1.48	1.48	1.89	1.89	2.26	2.26	2.81	2.81	3.28	3.28	3.77	3.77	5.05	5.05
2 hr	1.53	0.76	1.82	0.91	2.35	1.17	2.83	1.41	3.55	1.77	4.17	2.08	4.83	2.41	6.57	3.28
3 hr	1.71	0.57	2.03	0.67	2.61	0.87	3.16	1.05	4.02	1.34	4.75	1.58	5.55	1.85	7.69	2.56
6 hr	2.01	0.33	2.36	0.39	3.03	0.50	3.67	0.61	4.69	0.78	5.58	0.93	6.57	1.09	9.24	1.54
12 hr	2.30	0.19	2.68	0.22	3.39	0.28	4.08	0.34	5.17	0.43	6.12	0.51	7.17	0.59	10.0	0.83
24 hr	2.63	0.10	3.01	0.12	3.74	0.15	4.45	0.18	5.59	0.23	6.58	0.27	7.67	0.31	10.6	0.44
48 hr	2.99	0.06	3.41	0.07	4.21	0.08	4.96	0.10	6.16	0.12	7.19	0.14	8.33	0.17	11.4	0.23
3 day	3.26	0.04	3.71	0.05	4.56	0.06	5.35	0.07	6.58	0.09	7.63	0.10	8.78	0.12	11.8	0.16
4 day	3.50	0.03	3.98	0.04	4.86	0.05	5.68	0.05	6.93	0.07	8.00	0.08	9.15	0.09	12.2	0.12
7 day	4.11	0.02	4.67	0.02	5.66	0.03	6.55	0.03	7.86	0.04	8.94	0.05	10.0	0.06	13.0	0.07
10 day	4.67	0.01	5.30	0.02	6.38	0.02	7.32	0.03	8.69	0.03	9.80	0.04	10.9	0.04	13.8	0.05

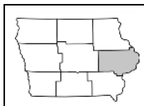
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.06: Section 5 - Central Iowa
Rainfall Depth and Intensity for Various Return Periods

	**Return Period**															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.39	4.78	0.46	5.59	0.57	6.91	0.67	8.1	0.81	9.76	0.92	11.1	1.04	12.4	1.33	15.9
10 min	0.58	3.51	0.68	4.08	0.84	5.08	0.98	5.92	1.19	7.16	1.35	8.13	1.52	9.15	1.94	11.6
15 min	0.71	2.84	0.83	3.32	1.03	4.12	1.20	4.82	1.45	5.81	1.65	6.61	1.86	7.44	2.37	9.50
30 min	0.99	1.99	1.16	2.33	1.45	2.91	1.70	3.40	2.05	4.11	2.34	4.68	2.63	5.27	3.36	6.73
1 hr	1.29	1.29	1.51	1.51	1.89	1.89	2.23	2.23	2.72	2.72	3.13	3.13	3.55	3.55	4.62	4.62
2 hr	1.58	0.79	1.85	0.92	2.33	1.16	2.76	1.38	3.39	1.69	3.91	1.95	4.46	2.23	5.88	2.94
3 hr	1.75	0.58	2.06	0.68	2.60	0.86	3.09	1.03	3.82	1.27	4.42	1.47	5.07	1.69	6.76	2.25
6 hr	2.05	0.34	2.40	0.40	3.03	0.50	3.61	0.60	4.47	0.74	5.20	0.86	5.98	0.99	8.02	1.33
12 hr	2.34	0.19	2.74	0.22	3.44	0.28	4.07	0.33	5.01	0.41	5.79	0.48	6.62	0.55	8.79	0.73
24 hr	2.67	0.11	3.08	0.12	3.81	0.15	4.46	0.18	5.44	0.22	6.26	0.26	7.12	0.29	9.37	0.39
48 hr	3.06	0.06	3.49	0.07	4.25	0.08	4.94	0.10	5.96	0.12	6.81	0.14	7.71	0.16	10.0	0.20
3 day	3.34	0.04	3.81	0.05	4.63	0.06	5.36	0.07	6.43	0.08	7.31	0.10	8.25	0.11	10.6	0.14
4 day	3.59	0.03	4.09	0.04	4.96	0.05	5.74	0.05	6.86	0.07	7.78	0.08	8.74	0.09	11.1	0.11
7 day	4.25	0.02	4.83	0.02	5.82	0.03	6.69	0.03	7.93	0.04	8.93	0.05	9.98	0.05	12.5	0.07
10 day	4.87	0.02	5.50	0.02	6.58	0.02	7.52	0.03	8.86	0.03	9.94	0.04	11.0	0.04	13.8	0.05

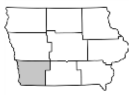
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.07: Section 6 - East Central Iowa
Rainfall Depth and Intensity for Various Return Periods

	**Return Period**															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.38	4.56	0.44	5.30	0.54	6.56	0.63	7.65	0.76	9.18	0.86	10.3	0.97	11.6	1.23	14.8
10 min	0.55	3.33	0.64	3.87	0.8	4.8	0.93	5.58	1.11	6.70	1.26	7.60	1.42	8.54	1.80	10.8
15 min	0.67	2.70	0.78	3.14	0.97	3.88	1.13	4.53	1.36	5.45	1.54	6.18	1.73	6.94	2.20	8.81
30 min	0.95	1.90	1.11	2.22	1.38	2.76	1.61	3.22	1.94	3.88	2.20	4.40	2.47	4.95	3.14	6.29
1 hr	1.23	1.23	1.44	1.44	1.80	1.80	2.11	2.11	2.58	2.58	2.96	2.96	3.36	3.36	4.37	4.37
2 hr	1.51	0.75	1.77	0.88	2.22	1.11	2.62	1.31	3.22	1.61	3.71	1.85	4.24	2.12	5.60	2.80
3 hr	1.68	0.56	1.96	0.65	2.47	0.82	2.93	0.97	3.63	1.21	4.22	1.40	4.85	1.61	6.50	2.16
6 hr	1.97	0.32	2.30	0.38	2.89	0.48	3.45	0.57	4.3	0.71	5.02	0.83	5.8	0.96	7.87	1.31
12 hr	2.28	0.19	2.65	0.22	3.31	0.27	3.93	0.32	4.88	0.40	5.68	0.47	6.56	0.54	8.87	0.73
24 hr	2.60	0.10	3.01	0.12	3.75	0.15	4.42	0.18	5.44	0.22	6.29	0.26	7.22	0.30	9.64	0.40
48 hr	2.98	0.06	3.43	0.07	4.22	0.08	4.93	0.10	6.01	0.12	6.90	0.14	7.86	0.16	10.3	0.21
3 day	3.28	0.04	3.72	0.05	4.51	0.06	5.24	0.07	6.32	0.08	7.22	0.10	8.19	0.11	10.7	0.14
4 day	3.53	0.03	3.98	0.04	4.78	0.04	5.50	0.05	6.58	0.06	7.49	0.07	8.46	0.08	10.9	0.11
7 day	4.17	0.02	4.67	0.02	5.53	0.03	6.29	0.03	7.39	0.04	8.30	0.04	9.25	0.05	11.6	0.06
10 day	4.75	0.01	5.30	0.02	6.24	0.02	7.04	0.02	8.20	0.03	9.12	0.03	10.0	0.04	12.4	0.05

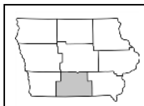
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.08: Section 7 - Southwest Iowa
Rainfall Depth and Intensity for Various Return Periods

	**Return Period**															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.38	4.58	0.45	5.42	0.57	6.88	0.67	8.09	0.82	9.85	0.93	11.2	1.05	12.6	1.36	16.3
10 min	0.55	3.33	0.66	3.98	0.83	5.01	0.98	5.92	1.20	7.23	1.37	8.26	1.55	9.31	1.99	11.9
15 min	0.68	2.72	0.80	3.22	1.02	4.08	1.20	4.82	1.46	5.87	1.67	6.70	1.89	7.57	2.43	9.72
30 min	0.97	1.94	1.16	2.32	1.47	2.95	1.75	3.5	2.13	4.27	2.44	4.88	2.76	5.52	3.53	7.07
1 hr	1.27	1.27	1.52	1.52	1.95	1.95	2.33	2.33	2.90	2.90	3.36	3.36	3.85	3.85	5.11	5.11
2 hr	1.58	0.79	1.88	0.94	2.43	1.21	2.92	1.46	3.66	1.83	4.29	2.14	4.95	2.47	6.68	3.34
3 hr	1.76	0.58	2.10	0.70	2.71	0.90	3.28	1.09	4.16	1.38	4.90	1.63	5.71	1.90	7.86	2.62
6 hr	2.09	0.34	2.46	0.41	3.15	0.52	3.82	0.63	4.87	0.81	5.78	0.96	6.78	1.13	9.49	1.58
12 hr	2.42	0.20	2.81	0.23	3.56	0.29	4.27	0.35	5.38	0.44	6.36	0.53	7.42	0.61	10.3	0.86
24 hr	2.76	0.11	3.18	0.13	3.95	0.16	4.7	0.19	5.86	0.24	6.88	0.28	7.99	0.33	11.0	0.45
48 hr	3.13	0.06	3.60	0.07	4.47	0.09	5.29	0.11	6.55	0.13	7.62	0.15	8.79	0.18	11.9	0.24
3 day	3.41	0.04	3.93	0.05	4.87	0.06	5.73	0.07	7.05	0.09	8.16	0.11	9.36	0.13	12.5	0.17
4 day	3.67	0.03	4.21	0.04	5.19	0.05	6.08	0.06	7.43	0.07	8.57	0.08	9.79	0.10	12.9	0.13
7 day	4.35	0.02	4.94	0.02	5.98	0.03	6.93	0.04	8.35	0.04	9.54	0.05	10.8	0.06	14.0	0.08
10 day	4.95	0.02	5.60	0.02	6.74	0.02	7.75	0.03	9.26	0.03	10.5	0.04	11.8	0.04	15.2	0.06

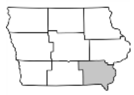
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.09: Section 8 - South Central Iowa
Rainfall Depth and Intensity for Various Return Periods

	**Return Period**															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.38	4.64	0.45	5.45	0.56	6.81	0.67	8.05	0.81	9.81	0.94	11.3	1.07	12.8	1.39	16.7
10 min	0.56	3.39	0.66	3.98	0.83	4.98	0.98	5.89	1.19	7.19	1.38	8.28	1.56	9.39	2.04	12.2
15 min	0.69	2.76	0.80	3.23	1.01	4.05	1.19	4.78	1.46	5.85	1.68	6.72	1.91	7.64	2.49	9.98
30 min	0.98	1.96	1.15	2.30	1.45	2.90	1.71	3.43	2.10	4.20	2.41	4.83	2.75	5.50	3.59	7.19
1 hr	1.29	1.29	1.51	1.51	1.88	1.88	2.24	2.24	2.77	2.77	3.23	3.23	3.72	3.72	5.02	5.02
2 hr	1.62	0.81	1.86	0.93	2.32	1.16	2.76	1.38	3.45	1.72	4.04	2.02	4.69	2.34	6.45	3.22
3 hr	1.82	0.60	2.08	0.69	2.59	0.86	3.08	1.02	3.88	1.29	4.58	1.52	5.35	1.78	7.49	2.49
6 hr	2.15	0.35	2.45	0.40	3.05	0.50	3.64	0.60	4.60	0.76	5.45	0.90	6.40	1.06	9.04	1.50
12 hr	2.44	0.20	2.81	0.23	3.53	0.29	4.21	0.35	5.29	0.44	6.24	0.52	7.28	0.60	10.1	0.84
24 hr	2.77	0.11	3.20	0.13	3.99	0.16	4.74	0.19	5.90	0.24	6.90	0.28	7.98	0.33	10.8	0.45
48 hr	3.18	0.06	3.64	0.07	4.49	0.09	5.28	0.11	6.50	0.13	7.54	0.15	8.66	0.18	11.6	0.24
3 day	3.47	0.04	3.99	0.05	4.91	0.06	5.75	0.07	7.01	0.09	8.07	0.11	9.21	0.12	12.1	0.16
4 day	3.73	0.03	4.29	0.04	5.26	0.05	6.13	0.06	7.43	0.07	8.51	0.08	9.65	0.10	12.6	0.13
7 day	4.43	0.02	5.04	0.03	6.09	0.03	7.01	0.04	8.38	0.04	9.49	0.05	10.6	0.06	13.6	0.08
10 day	5.07	0.02	5.73	0.02	6.85	0.02	7.84	0.03	9.27	0.03	10.4	0.04	11.6	0.04	14.7	0.06

D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

Table 2B-2.10: Section 9 - Southeast Iowa
Rainfall Depth and Intensity for Various Return Periods

	**Return Period**															
	1 year		***2 year***		***5 year***		***10 year***		***25 year***		***50 year***		***100 year***		***500 year***	
Duration	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***	***D***	***I***
5 min	0.38	4.57	0.44	5.33	0.54	6.58	0.64	7.68	0.76	9.22	0.87	10.4	0.97	11.7	1.24	14.8
10 min	0.55	3.34	0.65	3.9	0.80	4.82	0.93	5.62	1.12	6.76	1.27	7.66	1.43	8.60	1.81	10.8
15 min	0.68	2.72	0.79	3.17	0.98	3.93	1.14	4.57	1.37	5.49	1.55	6.23	1.74	6.98	2.21	8.85
30 min	0.95	1.9	1.11	2.22	1.38	2.76	1.61	3.22	1.94	3.88	2.20	4.40	2.46	4.93	3.12	6.25
1 hr	1.23	1.23	1.43	1.43	1.78	1.78	2.09	2.09	2.54	2.54	2.90	2.90	3.28	3.28	4.24	4.24
2 hr	1.51	0.75	1.76	0.88	2.19	1.09	2.58	1.29	3.14	1.57	3.61	1.80	4.10	2.05	5.35	2.67
3 hr	1.68	0.56	1.96	0.65	2.45	0.81	2.89	0.96	3.54	1.18	4.08	1.36	4.66	1.55	6.15	2.05
6 hr	1.99	0.33	2.32	0.38	2.91	0.48	3.44	0.57	4.25	0.70	4.92	0.82	5.63	0.93	7.50	1.25
12 hr	2.31	0.19	2.71	0.22	3.41	0.28	4.03	0.33	4.96	0.41	5.74	0.47	6.56	0.54	8.68	0.72
24 hr	2.68	0.11	3.12	0.13	3.90	0.16	4.59	0.19	5.62	0.23	6.46	0.26	7.35	0.30	9.64	0.40
48 hr	3.12	0.06	3.58	0.07	4.39	0.09	5.11	0.10	6.18	0.12	7.06	0.14	7.98	0.16	10.3	0.21
3 day	3.41	0.04	3.9	0.05	4.73	0.06	5.47	0.07	6.56	0.09	7.45	0.10	8.39	0.11	10.7	0.14
4 day	3.66	0.03	4.16	0.04	5.02	0.05	5.78	0.06	6.88	0.07	7.78	0.08	8.72	0.09	11.0	0.11
7 day	4.33	0.02	4.87	0.02	5.79	0.03	6.59	0.03	7.72	0.04	8.63	0.05	9.57	0.05	11.8	0.07
10 day	4.95	0.02	5.54	0.02	6.54	0.02	7.38	0.03	8.57	0.03	9.51	0.03	10.4	0.04	12.8	0.05

D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

C. References

Perica, et. al. *NOAA Atlas 14: Precipitation-Frequency Atlas of the United States, Volume 8 Version 2.0: Midwestern States*. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, & National Weather Service. 2013. http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume8.pdf

Huff & Angel. *Bulletin 71: Rainfall Frequency Atlas of the Midwest.* Midwestern Climate Center, Illinois State Water Survey. 1992.

Time of Concentration

A. Introduction

Time of concentration (T_c) is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet.

Time of concentration is a critical component in some analysis methods for calculating peak discharge from an area. The peak discharge occurs when all segments of the drainage area are contributing to the runoff from the site.

There are many methods available to estimate the time of concentration including the Kirpich formula, Kerby formula, NRCS Velocity Method, and NRCS Lag Method. The NRCS Velocity and Lag methods are two of the most commonly used methods for determining time of concentration and are described below.

B. Factors Affecting Time of Concentration

1. **Surface Roughness:** One of the most significant effects of urban development on overland flow is the lowering of retardance to flow causing higher velocities. Undeveloped areas with very slow and shallow overland flow (sheet flow and shallow concentrated flow) through vegetation become modified by urban development. Flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

2. **Channel Shape:** In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

3. **Slope:** Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions

   Urbanization usually decreases time of concentration, thereby increasing the peak discharge. However, time of concentration can be increased as a result of ponding behind small or inadequate drainage systems (including inlets and road culverts) or by reduction of land slope through grading.

C. NRCS Velocity Method

The NRCS Velocity method is described in full detail in NRCS TR-55.

Travel time (T_t) is the time it takes water to travel from one location to another. The travel time between two points is determined using the following relationship:

$$T_t = \frac{\ell}{3{,}600V}$$ **Equation 2B-3.01**

where:

T_t = travel time, hours
ℓ = flow length, ft
V = average velocity, ft/s
3,600 = conversion factor, seconds to hours

Surface water flow through the watershed occurs as three different flow types: sheet flow, shallow concentrated flow, and open channel flow. The NRCS Velocity Method assumes that time of concentration (T_c) is the sum of travel times for each of these flow segments along the hydraulically most distant flow path.

$$T_c = T_s + T_c + T_o$$ **Equation 2B-3.02**

where:

T_c = time of concentration, hours
T_s = travel time for sheet flow, hours
T_c = travel time of shallow concentrated flow, hours
T_o = travel time for open channel flow, hours

1. **Sheet Flow:** Sheet flow is defined as flow over plane surfaces. Sheet flow usually occurs in the headwaters of a stream near the ridgeline that defines the watershed boundary. Typically, sheet flow occurs for no more than 100 feet before transitioning to shallow concentrated flow. A simplified version of the Manning’s kinematic solution may be used to compute travel time for sheet flow.

   $$T_{\boldsymbol{t}} = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$ **Equation 2B-3.03**

   where:

   T_t = travel time, h
   n = Manning’s roughness coefficient (Table 2B-3.01)
   ℓ = sheet flow length, ft
   P_2 = 2 year, 24 hour rainfall, in
   S = slope of land surface, ft/ft

Table 2B-3.01: Manning's Roughness Coefficient for Sheet Flow

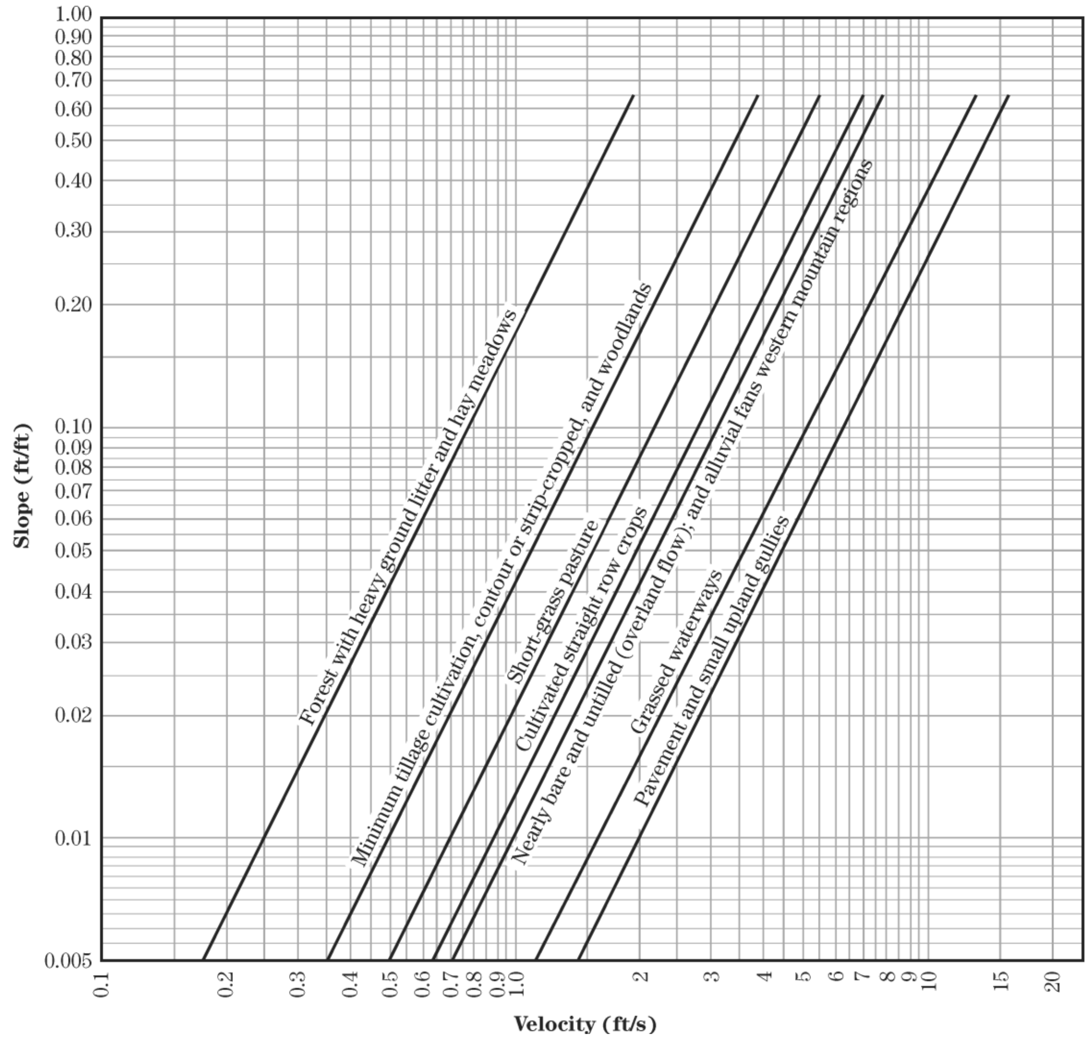
Surface Description	***n***
Smooth Surface (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[1]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[2]	
Light underbrush	0.40
Dense underbrush	0.80

[1] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[2] When selecting n, consider cover to a height of about 0.1 foot. This is the only part of the plant cover that will obstruct sheet flow.

2. **Shallow Concentrated Flow:** After approximately 100 feet, sheet flow usually becomes shallow concentrated flow collecting in swales, small rills, and gullies. Shallow concentrated flow is assumed not to have a well-defined channel and has flow depth of 0.1 to 0.5 feet. It is assumed that shallow concentrated flow can be represented by one of seven flow types. These flow types are shown in Figure 2B-3.01 and Table 2B-3.02.

   After estimating average velocity using Figure 2B-3.01 or the equations from Table 2B-3.02, use Equation 2B-3.01 to estimate travel time for the shallow concentrated flow segment.

Figure 2B-3.01: Velocity Versus Slope for Shallow Concentrated Flow



Source: NRCS National Engineerining Handbook, Part 630, Chapter 15

Table 2B-3.02: Equations and Assumptions Developed from Figure 2B-3.01

Flow Type	Depth (feet)	Manning's n	Velocity Equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V = 20.238(s)^{0.5}$
Grassed waterways (and unpaved urban areas)	0.4	0.050	$V = 16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans	0.2	0.051	$V = 9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V = 8.762(s)^{0.5}$
Short-grass prairie	0.2	0.073	$V = 6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V = 5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V = 2.516(s)^{0.5}$

3. **Open Channel Flow:** Open channels (swales, ditches, storm sewers, and tiles not flowing full) are assumed to begin where surveyed cross-sectional information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on U.S. Geological Survey (USGS) quadrangle sheets.

   Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for the bankfull elevation. Manning's equation is:

   $$V = \frac{1.49\left(r^{\frac{2}{3}}\right)\left(s^{\frac{1}{2}}\right)}{n}$$ **Equation 2B-3.04**

   where:
   V = average velocity, ft/s

   R = hydraulic radius, ft
   = a/P
   a = cross-sectional areas of flow, ft^2
   P = wetted perimeter, ft
   s = slope of the hydraulic grade line (channel slope), ft/ft
   n = Manning's value for open channel flow

   Refer to Parts 2D (Storm Sewer Design), 2E (Culvert Design), or 2F (Open Channel Flow) for additional details on evaluating flow velocity for open channel flow.

Table 2B-3.03: Manning's Roughness Coefficients (n) for Open Channel Flow

Type of Channel and Description	n
A. Closed Conduits Flowing Partly Full	
1. Steel - Riveted and Spiral	0.016
2. Cast Iron - Coated	0.013
3. Cast Iron - Uncoated	0.014
4. Corrugated Metal - Subdrain	0.019
5. Corrugated Metal - Storm Drain	0.024
6. Concrete Culvert, straight and fee of debris	0.011
7. Concrete Culvert, with bends, connections, and some debris	0.013
8. Concrete Sewer with manholes, inlet, etc., straight	0.015
9. Concrete, Unfinished, steel form	0.013
10. Concrete, Unfinished, smooth wood form	0.014
11. Wood - Stave	0.012
12. Clay - Vitrified sewer	0.014
13. Clay - Vitrified sewer with manholes, inlet, etc.	0.015
14. Clay - Vitrified subdrain with open joints	0.016
15. Brick - Glazed	0.013
16. Brick - Lined with cement mortar	0.015
B. Lined or Built-Up Channels	
1. Corrugated Metal	0.025
2. Wood - Planed	0.012
3. Wood - Unplaned	0.013
5. Concrete - Trowel finish	0.013
6. Concrete - Float finish	0.015
7. Concrete - Finished, with gravel on bottom	0.017
8. Concrete - Unfinished	0.017
9. Concrete Bottom Float Finished with sides of:	
a. Random stone in mortar	0.020
b. Cement rubble masonry	0.025
c. Dry ruble or rip rap	0.030
10. Gravel Bottom with sides of:	
a. Formed concrete	0.020
b. Dry rubble or rip rap	0.033
11. Brick - Glazed	0.013
12. Brick - In cement mortar	0.015
13. Masonry Cemented Rubble	0.025
14. Dry Rubble	0.032
15. Smooth Asphalt	0.013
16. Rough Asphalt	0.016
C. Excavated or Dredged Channel	
1. Earth, straight and uniform	
a. Clean, after weather	0.022
b. Gravel, uniform section, clean	0.025
c. With short grass, few weeds	0.027
2. Earth, winding and sluggish	
a. No vegetation	0.025
b. Grass, some weeds	0.030
c. Dense weeds or aquatic plants in deep channels	0.035
d. Earth bottom and rubble sides	0.030
e. Stony bottom and weedy banks	0.040
3. Channels not maintained, weeds and brush uncut	
a. Dense weeds, high as flow depth	0.080
b. Clean bottom, brush on sides	0.050
D. Natural Streams	
1. Clean, straight bank, full stage, no rifts or deep pools	0.030
2. As D.1 above, but some weeds and stones	0.035
3. Winding, some pools and shoals, clean	0.040
4. As D.3 above, but lower stages, more ineffective slope and sections	0.045
5. As D.3 above, but some weeds and stones	0.048
6. As D.4 above, but with stony sections	0.050
7. Sluggish river reaches, rather weedy or with very deep pools	0.070
8. Very weedy reaches	0.100

Source: Chow, V.T. 1959

D. NRCS Lag Method

In drainage basins where a large segment of the area is rural in character and has long hydraulic length, the potential for retention of rainfall on the watershed increases along with travel time. Under these conditions, the NRCS lag method may be used since it includes most of the factors to estimate travel time and thus time of concentration.

The NRCS lag method was developed from observations of agricultural watersheds where overland flow paths are poorly defined and channel flow is absent. However, it has been adapted to small urban watersheds less than 2,000 acres. For situations where the lag method is used in urban areas, an adjustment factor needs to be applied to the results to account for the effects of urbanization. This adjustment is described in number 5 below. The method performs reasonably well for completely paved areas, but performs poorly when channel flow (including storm sewers) is a significant part of the time of concentration.

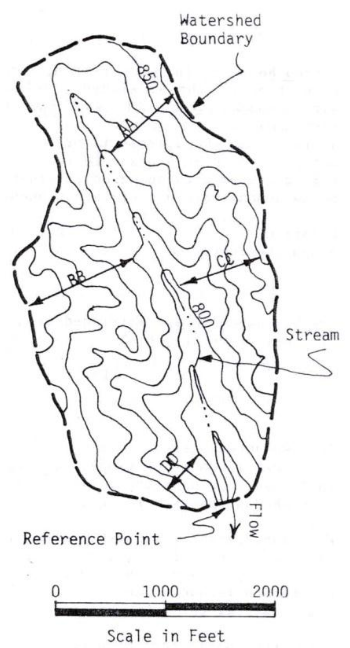
Lag is the delay between the time runoff from a rainfall event over a watershed begins until runoff reaches its maximum peak. Lag is a function of the flow length of the watershed, average land slope of the watershed, and the potential maximum retention of rainfall on the watershed.

1. **Flow Length of Watershed:** The flow length of the watershed, ℓ, is the length from the point of design along the main channel to the ridgeline at the upper end of the watershed. Moving upstream, the main channel may appear to divide into two channels at several points along its length. The main channel is then defined as the channel that drains the greater tributary drainage area. This same definition is used for all further upstream channel divisions until the watershed ridgeline is reached.

   Since many channels meander through their floodplains and since most designs are based on floods that exceed channel capacity, the proper channel length to use is actually the length along the valley; i.e., the channel meanders should be ignored.

2. **Average Watershed Slope:** The average watershed land slope, Y, is estimated using one of the two methods described below. Average watershed slope is a variable, which is usually not readily apparent. Therefore, a systematic procedure for finding slope is desirable. Several observations or map measurements are commonly needed. Care should be taken in determining this parameter as the time of concentration (and subsequently the peak discharge and hydrograph shape) is sensitive to the value used for watershed slope. Best hydrologic results are obtained when the slope value represents a weighted average for the area. Two methods for computing slope are demonstrated in example exercises below.

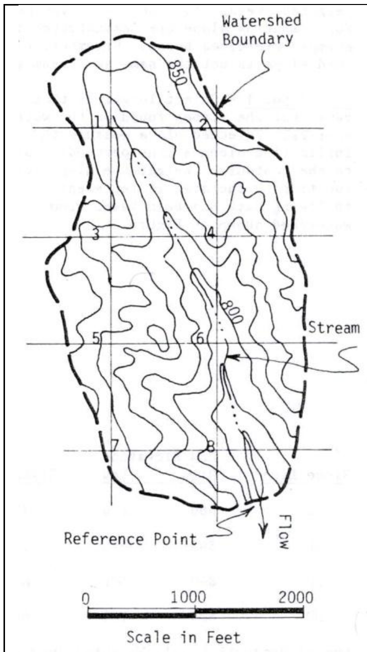
3. **Maximum Potential Retention:** The parameter S represents the potential maximum moisture retention of the soil and is related to soil and cover conditions of the watershed. It is empirically-determined using the SCS curve number (CN), which is provided in Tables 2B-4.03 through 2B-4.05 in Section 2B-4.



Method One

Select locations that represent the slopes found in the watershed. Near each selected place, measure the inclination along a line perpendicular to the contours. Weight the slope for each location by the area it represents. The following data has been taken from the watershed shown below.

Slope Line	End Elevation		Distance	Slope (Pct)	Prop. Of Watershed (Pct)	Product (Pct x Pct)
	High	Low				
AA	860	820	780	5	25	1.25
BB	845	810	1070	3	35	1.05
CC	840	800	800	5	25	1.25
DD	820	790	460	7	15	1.05
Sum of Products = Weighted Average Watershed Slope						4.60 Use 5%



Method Two

In this method, each sample location represents the same proportion of the watershed. Select the locations by overlaying the map with a grid system. The watershed slope perpendicular to contours through each intersection of grid lines is determined as in Method One and the average for all intersections is considered to be watershed slope. The watershed used as an example for this method is the same watershed as above. A grid system with numbered intersections is shown in the figure. Tabulations below demonstrate use of this procedure.

Location	1	2	3	4	5	6	7	8	Sum
Slope (Percent)	6	8	6	7	5	10	3	6	51

The Weighted Average Watershed Slop is the arithmetic average, 6.4%. Use 6%.

The two answers are not identical. Due to the greater number of sample locations used in Method Two, perhaps the answer of 6% watershed slope is more accurate.

When subareas of a watershed have widely varying slopes, this may justify separate analyses by subareas and use of the hydrograph method for hydrologic data at the watershed outlet. With other parameters held constant, a slope variation of 10% affects peak discharge approximately 3% to 4%. A 20% change in slope is reflected by a 6% to 8% change in the peak rate.

4. **Lag Equation:** The equations for calculating the time of concentration by the Lag method are as follows.

$$T_c = \frac{L}{0.6}$$ **Equation 2B-3.05**

and

$$L = \frac{\ell^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}$$ **Equation 2B-3.06**

where:

L = lag, hr
T_c = Time of concentration, hr
ℓ = flow length, ft.
Y = average watershed land slope, %
S = maximum potential retention, in
= $\frac{1000}{CN} - 10$
CN = NRCS Curve Number (Section 5B-4, Tables 2B-4.03 through 2B-4.05)

Note: Curve numbers less than 50 or greater than 95 should not be used with the Lag method.

5. **Adjustments for Urbanization:** Because the lag equation was developed for rural areas, it can overestimate lag and T_c in urban areas for two reasons. First, the increased amount of impervious area allows water from overland flow sources and side channels to reach the main channel at a much faster rate than under natural conditions. Second is the extent to which a stream (usually the major watercourse in the watershed) has been changed over natural conditions to allow higher flow velocities. The lag time can be corrected for the effects of urbanization utilizing the adjustment factors from Figures 2B-3.02 and 2B-3.03. The amount of modification to the hydraulic flow length must be determined from topographic maps or aerial photographs following a field inspection of the area. The modification to the hydraulic flow length not only includes pipes or channels, but also the length of flow in streets.

   For situations where the lag equation is utilized in urban areas, the following equation should be used to adjust the T_c calculated by the NRCS lag method:

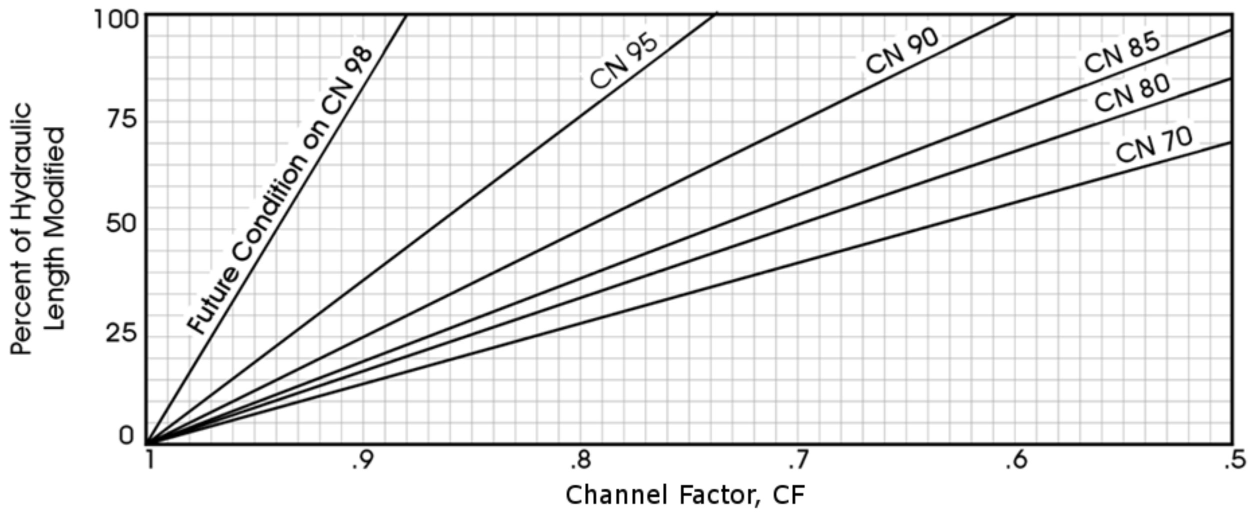
$$T'_c = T_c \times CF \times IF$$ **Equation 2B-3.07**

where:

T'_c = Adjusted time of concentration, hr
T_c = Time of concentration, hr (from Equation 2B-3.05)
CF = Channel Improvement Factor
IF = Impervious area factor

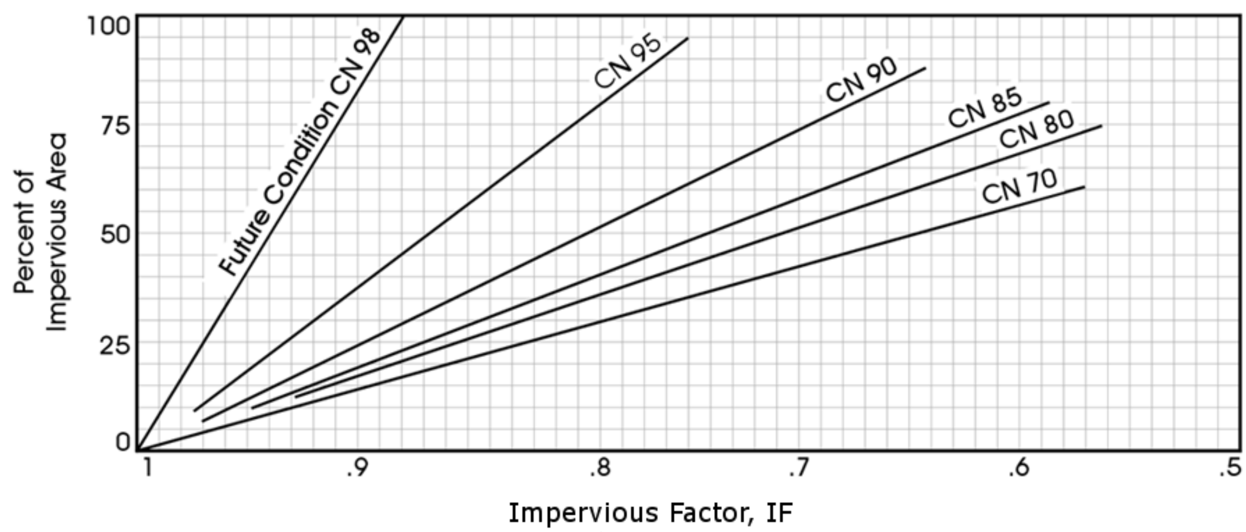
Source: FHWA Hydraulic Engineering Circular No. 19

Figure 2B-3.02: Factors for Adjusting Lag When the Main Channel Has Been Hydraulically Improved



Source: FHWA, HEC-19

Figure 2B-3.03: Factors for Adjusting Lag When Impervious Areas Occur in the Watershed



Source: FHWA, HEC-19

Worksheet 2B-3.01: Time of Concentration (T_c) or Travel Time (T_t)

Project ______________________________________ By _____________ Date _______

Location ______________________________________ Checked _________ Date _______

Circle one: Present Developed

Circle one: T_c T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

		Unit	Segment 1		Segment 2		Total
	Sheet flow (Applicable to T_c only)	Segment ID					
1.	Surface description (Table 2B-3.01)……………………						
2.	Manning's roughness coeff., n (Table 2B-3.01)…………						
3.	Flow Length, L (Total L less than or equal to 300')…	ft					
4.	Two year, 24 hour rainfall, P_2…………………………….	in					
5.	Land slope, s……………………………………………….	ft / ft					
6.	$T_t = \frac{0.007(nL)^{0.8}}{\left(\sqrt{P_2}\right)\left(s^{0.4}\right)}$ Compute T_t……………..	hr		+		=	
	Shallow concentrated flow	Segment ID					
7.	Surface description (paved or unpaved)………………..						
8.	Flow length, L………………………………………………	ft					
9.	Watercourse slope, s…………………………………...	ft / ft					
10.	Average velocity, V (Figure 2B-3.01)……………………	ft / s					
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t……………..	hr		+		=	
	Open channel / pipe flow	Segment ID					
12.	Cross sectional flow area, a……………………………...	ft^2					
13.	Wetted perimeter, P_w……………………………………..	ft					
14.	Hydraulic radius, $r = \frac{a}{P_w}$ Compute r………………	ft					
15.	Channel slope, s…………………………………………..	ft / ft					
16.	Manning's roughness coeff., n…………………………...						
17.	$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$ Compute V……………...	ft / s					
18.	Flow length, L………………………………………………	ft					
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t……………..	hr		+		=	
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11 and 19)………………………………………	hr					

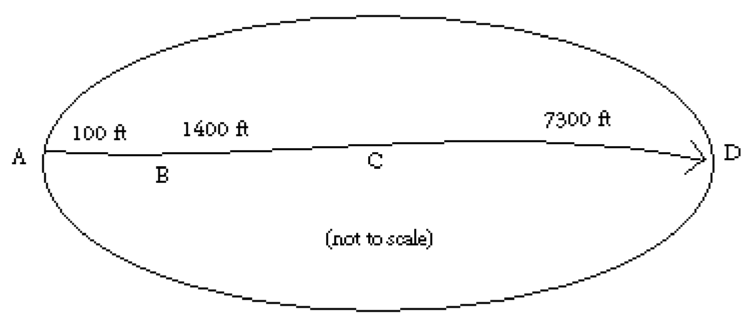
Example 2B-3.01: Time of Concentration

Example: The sketch below shows a watershed. The problem is to compute T_C at the outlet of the watershed (point D). The 2 year 24 hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_C, first determine T_t for each segment from the following information:

Segment AB: Sheet flow
Dense grass
Slope (s) = 0.01 ft/ft
Length (L) = 100 ft

Segment BC: Shallow concentrated flow
Unpaved
s = 0.01 ft/ft
L = 1400 ft

Segment CD: Channel flow
Manning's n = .05
Flow area (a) = 27 ft^2
Wetted perimeter (p_w) = 28.2 ft
s = 0.005 ft/ft
L = 7300 ft



Worksheet 2B-3.02: Time of Concentration (T_c) or Travel Time (T_t)

Project *Example* By ____________ Date _______

Location ________________________________ Checked _________ Date _______

Circle one: Present Developed

Circle one: T_c T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

	Sheet flow (Applicable to T_c only)	Segment ID	AB			
1.	Surface description (Table 2B-3.01)……………………		Dense Grass			
2.	Manning's roughness coeff., n (Table 2B-3.01)……………		0.24			
3.	Flow Length, L (Total L less than or equal to 300')…	ft	100			
4.	Two year, 24 hour rainfall, P_2…………………………….	in	3.6			
5.	Land slope, s……………………………………………….	ft / ft	0.01			
6.	$T_t = \frac{0.007(nL)^{0.8}}{(\sqrt{P_2})(s^{0.4})}$ Compute T_t……………..	hr	0.30	+	=	0.30
	Shallow concentrated flow	Segment ID	BC			
7.	Surface description (paved or unpaved)………………..		Unpaved			
8.	Flow length, L………………………………………………	ft	1400			
9.	Watercourse slope, s…………………………………...	ft / ft	0.01			
10.	Average velocity, V (Figure 2B-3.01)……………………	ft / s	1.6			
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t……………..	hr	0.24	+	=	0.24
	Open channel/pipe flow	Segment ID	CD			
12.	Cross sectional flow area, a……………………………...	ft^2	27			
13.	Wetted perimeter, P_w……………………………………..	ft	28.2			
14.	Hydraulic radius, $r = \frac{a}{P_w}$ Compute r………………	ft	0.957			
15.	Channel slope, s…………………………………………..	ft / ft	0.005			
16.	Manning's roughness coeff., n…………………………...		0.05			
17.	$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$ Compute V……………...	ft / s	2.05			
18.	Flow length, L………………………………………………	ft	7300			
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t……………..	hr	0.99	+	=	0.99
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11 and 19)…………………………………………				hr	1.53

E. References

Chow, V.T. *Open Channel Hydraulics*. 1959.

U.S. Department of Transportation. *Hydraulic Engineering Circular No. 19: Hydrology*. 1984.

USDA Natural Resource Conservation Service. *National Engineering Handbook - Part 630. Chapter 15: Time of Concentration.* 2010.

Runoff and Peak Flow

A. Introduction

Determining the volume and peak rate of runoff from a site is critical in designing and signing stormwater infrastructure including storm sewer, ditches, culverts, and detention basins. The common methods used to evaluate stormwater runoff include the Rational method for determination of peak flow and SCS methods for determination of both peak flow and runoff volume.

B. Rational Method

The Rational equation is commonly used for design in developed urban areas. The Rational equation is given as:

$$Q_T = Ci_TA \qquad \textbf{Equation 2B-4.01}$$

where:

Q_T = estimate of the peak rate of runoff (cfs) for some recurrence interval, T
C = runoff coefficient; fraction of runoff, expressed as a dimensionless decimal fraction, that appears as surface runoff from the contributing drainage area.
i_T = average rainfall intensity (in/hr) for some recurrence interval, T, during that period of time equal to the T_c.
A = the contributing drainage area (acres) to the point of design that produces the maximum peak rate of runoff.
T_c = Time of concentration, minutes.

1. Rational Method Characteristics:

a. When using the Rational formula, an assumption is made that the maximum rate of flow is produced by a constant rainfall, which is maintained for a time equal to the time of concentration, which is the time required for the surface runoff from the most remote part of the drainage basin to reach the point being considered. There are other assumptions used in the Rational method, and thus the designer or engineer should consider how exceptions or other unusual circumstances might affect those results.
   1) The rainfall is uniform in space over the drainage area being considered.
   2) The rainfall intensity remains constant during the time period equal to the time of concentration.
   3) The runoff frequency curve is parallel to the rainfall frequency curve. This implies that the same value of the runoff coefficient is used for all recurrence intervals. In practice, the runoff coefficient is adjusted with a frequency coefficient (Cf) for the 25 year through 100 year recurrence intervals.
   4) The drainage area is the total area tributary to the point of design.

b. The following are additional factors that might not normally be considered, yet could prove important:
   1) The storm duration gives the length of time over which the average rainfall intensity (i_T) persists. Neither the storm duration, nor i_T, says anything about how the intensity varies during the storm, nor do they consider how much rain fell before the period in question.
   2) A 20% increase or decrease in the value of C has a similar effect as changing a 5 year recurrence interval to a 15 year or a 2 year interval, respectively.
   3) The chance of all design assumptions being satisfied simultaneously is less than the chance that the rainfall rate used in the design will actually occur. This, in effect, creates a built-in factor of safety.
   4) In an irregularly-shaped drainage area, a part of the area that has a short time of concentration (T_c) may cause a greater runoff rate (Q) at the intake or other design point) than the runoff rate calculated for the entire area. This is because parts of the area with long concentration times are far less susceptible to high-intensity rainfall. Thus, they skew the calculation.
   5) A portion of a drainage area that has a value of C much higher than the rest of the area may produce a greater amount of runoff at a design point than that calculated for the entire area. This effect is similar to that described above. In the design of storm sewers for small subbasin areas such as a cul-de-sac in a subdivision, the designer should be aware that an extremely short time of concentration will result in a high estimate of the rainfall intensity and the peak rate of runoff. The time of concentration estimates should be checked to make sure they are reasonable. For most applications, a minimum T_c of 15 minutes may be assumed.
   6) In some cases, runoff from a portion of the drainage area that is highly-impervious may result in a greater peak discharge than would occur if the entire area was considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow that is critical for a particular application.
   7) When designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 100 feet in urban areas and 300 feet in rural areas should be done only after careful consideration.

2. Rational Method Limitations: The use of the rational formula is subject to several limitations and procedural issues in its use.

a. The most important limitation is that the only output from the method is a peak discharge (the method provides only an estimate of a single point on the runoff hydrograph).

b. The average rainfall intensities used in the formula have no time sequence relation to the actual rainfall pattern during the storm.

c. The computation of T_c should include the overland flow time, plus the time of flow in open and/or closed channels to the point of design.

d. The runoff coefficient, C, is usually estimated from a table of values (see Table 2B-4.01). The user must use good judgment when evaluating the land use in the drainage area under consideration. Note in Table 2B-4.01, that the value of C will vary with the return frequency.

e. Many users assume the entire drainage area is the value to be entered in the Rational method equation. In some cases, the runoff from the only interconnected impervious area yields the larger peak flow rate.

f. Studies and experience have shown that the Rational method tends to underestimate runoff rates for large drainage areas. This is due, in part, to the fact that a difference can exist between intense point rainfall (rainfall over a small area) and mean catchment area rainfall (average rainfall). For these reasons, use of the Rational method should be limited to drainage areas 40 acres or less.

3. Use of the Rational Method:

a. Runoff Coefficient: The runoff coefficient (C) represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception; all of which affect the time distribution and peak rate of runoff. The runoff coefficient is the variable of the Rational method that requires the most judgment and understanding on the part of the designer. While engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters. The Engineer should realize the C values shown in Table 2B-4.01 are typical values, and may have to be adjusted if the site deviates from typical conditions such as an increase or decrease in percent impervious.

The values are presented for different surface characteristics, as well as for different aggregate land uses. The coefficient for various surface areas can be used to develop a composite value for a different land use. The runoff values for business, residential, industrial, schools, and railroad yard areas are an average of all surfaces typically found in the particular land use.

The hydrologic soil groups used in Table 2B-4.01 are discussed in detail later in this section.

Table 2B-4.01: Runoff Coefficients for the Rational Method

Cover Type and Hydrologic Condition		Runoff Coefficients for Hydrologic Soil Group											
		A			***B***			***C***			***D***		
	Recurrence Interval	***5***	***10***	***100***	***5***	***10***	***100***	***5***	***10***	***100***	***5***	***10***	***100***
Open Space (lawns, parks, golf courses, cemeteries, etc.)													
Poor condition (grass cover < 50%)		.25	.30	.50	.45	.55	.65	.65	.70	.80	.70	.75	.85
Fair condition (grass cover 50% to 75%)		.10	.10	.15	.25	.30	.50	.45	.55	.65	.60	.65	.75
Good condition (grass cover >75%)		.05	.05	.10	.15	.20	.35	.35	.40	.55	.50	.55	.65
Impervious Areas													
Parking lots, roofs, driveways, etc. (excluding ROW)		.95	.95	.98	.95	.95	.98	.95	.95	.98	.95	.95	.98
Streets and roads:													
Paved; curbs & storm sewers (excluding ROW)		.95	.95	.98	.95	.95	.98	.95	.95	.98	.95	.95	.98
Paved; open ditches (including ROW)		---	---	---	.70	.75	.85	.80	.85	.90	.80	.85	.90
Gravel (including ROW)		---	---	---	.60	.65	.75	.70	.75	.85	.75	.80	.85
Dirt (including ROW)		---	---	---	.55	.60	.70	.65	.70	.80	.70	.75	.85
Urban Districts (excluding ROW)													
Commercial and business (85% impervious)		---	---	---	---	---	---	.85	.85	.90	.90	.90	.95
Industrial (72% impervious)		---	---	---	---	---	---	.80	.80	.85	.80	.85	.90
Residential Districts by Average Lot Size (excluding ROW)[1]													
1/8 acre (36% impervious)		---	---	---	---	---	---	.55	.60	.70	.65	.70	.75
1/4 acre (36% impervious)		---	---	---	---	---	---	.55	.60	.70	.65	.70	.75
1/3 acre (33% impervious)		---	---	---	---	---	---	.55	.60	.70	.65	.70	.75
1/2 acre (20% impervious)		---	---	---	---	---	---	.45	.50	.65	.60	.65	.70
1 acre (11% impervious)		---	---	---	---	---	---	.40	.45	.60	.55	.60	.65
2 acres (11% impervious)		---	---	---	---	---	---	.40	.45	.60	.55	.60	.65
Newly Graded Areas (pervious areas only, no vegetation)													
Agricultural and Undeveloped													
Meadow - protected from grazing (pre-settlement)		.10	.10	.25	.10	.15	.30	.30	.35	.55	.45	.50	.65
Straight Row Crops													
Straight Row (SR)	*Poor Condition*	.33	.39	.55	.52	.58	.71	.70	.74	.84	.78	.81	.89
	Good Condition	.24	.30	.46	.45	.51	.66	.62	.67	.78	.73	.76	.86
SR + Crop Residue (CR)	*Poor Condition*	.31	.37	.54	.50	.56	.70	.67	.72	.82	.75	.79	.87
	Good Condition	.19	.25	.41	.38	.45	.61	.55	.60	.73	.62	.67	.78
Contoured (C).............................	*Poor Condition*	.29	.35	.52	.47	.53	.70	.60	.65	.77	.70	.74	.84
	Good Condition	.21	.26	.43	.38	.45	.61	.55	.60	.73	.65	.69	.80
C+CR...	*Poor Condition*	.27	.33	.50	.45	.51	.66	.57	.63	.75	.67	.72	.82
	Good Condition	.19	.25	.41	.36	.43	.59	.52	.58	.71	.62	.67	.78
Contoured & Terraced (C&T)	*Poor Condition*	.22	.28	.45	.36	.43	.59	.50	.56	.70	.55	.60	.73
	Good Condition	.16	.22	.38	.31	.37	.54	.45	.51	.66	.52	.58	.71
C&T + CR	*Poor Condition*	.13	.19	.35	.31	.37	.54	.45	.51	.66	.52	.58	.71
	Good Condition	.10	.16	.32	.27	.33	.50	.43	.49	.65	.50	.56	.70

[1] The average percent impervious area shown was used to develop composite coefficients.

Note: Rational coefficients were derived from SCS CN method

b. Composite Runoff Analysis: Care should be taken not to average runoff coefficients for large segments that have multiple land uses of a wide variety (i.e., business to agriculture). However, within similar land uses, it is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. The composite procedure can be applied to an entire drainage area, or to typical sample blocks as a guide to selection of reasonable values of the coefficient for an entire area.

c. Rainfall Intensity: The intensity (i_T) is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency, with duration equal to the time of concentration. The method(s) for determining time of concentration are presented in Section 2B-3.

From a practical standpoint, using a T_c of less than 15 minutes may yield unreasonably high flow rates. For most applications, a minimum T_c of 15 minutes may be used.

After the T_c has been determined, the rainfall intensity should be obtained. For the Rational method, the design rainfall intensity is that which occurs for the design year storm whose duration equals the time of concentration. Tables 2B-2.02 through 2B-2.10 in Section 2B-2 provide the Iowa rainfall data from Bulletin 71 to allow determination of rainfall intensity based on duration equaling the time of concentration.

d. Area: The area (A) of the basin in acres. A map showing the limits of the drainage basin used in design should be provided with design data and will be superimposed on the grading plan showing subbasins. As mentioned earlier, the configuration of the contributing area with respect to pervious and impervious sub-areas and the flow path should be considered when deciding whether to use all or a portion of the total area.

C. SCS Methods

Several methods of determining total runoff and peak runoff have been developed by the SCS (now known as the NRCS). The two methods described below include the SCS Runoff Curve Number method for determining the total runoff depth and the SCS Peak flow method, which utilizes the runoff depth and site conditions to determine the peak rate of runoff from a drainage area.

These methods are described in full detail in the NRCS Technical Release 55: Urban Hydrology for Small Watersheds. This document is also the basis for the publicly available computer program WIN-TR55. This section also includes information from the NRCS National Engineering Handbook, Part 630.

1. SCS Curve Number: The SCS methods classify the land use and soil type by a single parameter called the Curve Number (CN). The CN can be used to represent the drainage properties for any sized homogeneous watershed with a known percentage of imperviousness.

The major factors that determine CN are the hydrologic soil group, cover type, treatment, hydrologic condition, and antecedent runoff condition. Tables 2B-4.03 through 2B-4.05 include typical CN values for urban and agricultural areas respectively.

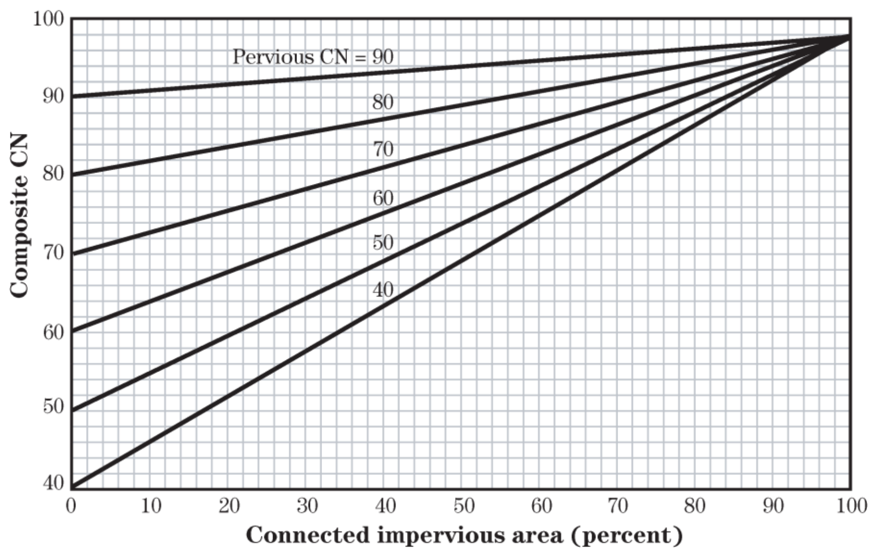
Several factors, such as the percentage of impervious area and the means of conveying runoff from the impervious areas to the drainage system, should be considered in computing the CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The urban CN values (Table 2B-4.03) were developed for typical land use relationships based upon specific assumed percentages of impervious area. These CN values were developed on the assumptions that (a) the pervious urban areas are equivalent to pasture in good hydrologic condition, (b) impervious areas have a CN of 98 and are directly connected to the drainage system, and (c) the CN values for urban and residential districts assume an average percent impervious as shown in Table 2B-4.03.

a. Connected Impervious Areas: An impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages in Table 2B-4.02, or the pervious land use assumptions are not applicable, use Figure 2B-4.01 or Equation 2B-4.02 to compute a composite CN.

Figure 2B-4.01: Composite CN with Connected Impervious Area



Source: NRCS National Engineering Handbook, Part 630, Chapter 9

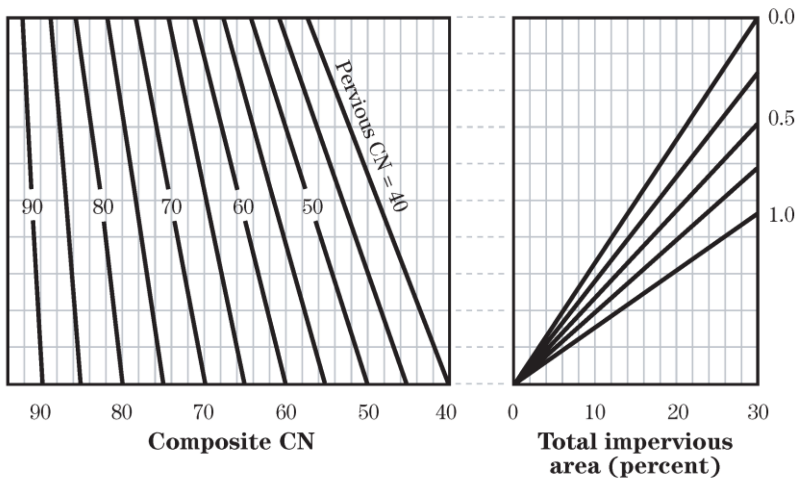
$$CN_c = CN_p + \left(\frac{P_{imp}}{100}\right)\left(98 - CN_p\right)$$ **Equation 2B-4.02**

where:

CN_c = composite runoff curve number
CN_p = pervious runoff curve number
P_{imp} = percent imperviousness

b. Unconnected Impervious Areas: If runoff from impervious areas occurs over a pervious area as sheet flow prior to entering the drainage system, the impervious area is unconnected. To determine the CN when all or part of the impervious area is not directly connected to the drainage system use Figure 2B-4.02 or Equation 2B-4.03 if the total impervious area is less than 30% of the total area. If the total impervious area is equal to or greater than 30% of the total area, utilize Figure 2B-4.02 or Equation 2B-4.02 because the absorptive capacity of the remaining pervious area will not significantly affect runoff.

Figure 2B-4.02: Composite CN with Unconnected Impervious Areas and Total Impervious Areas Less Than 30%



When the impervious area is less than 30%, obtain the composite CN by entering the right half of the figure with the percentage of total unconnected impervious area to total impervious area. Then move left to the appropriate CN and read down to find the composite CN.

Source: NRCS National Engineering Handbook, Part 630, Chapter 9

$$CN_c = CN_p + \left(\frac{P_{imp}}{100}\right)(98 - CN_p)(1 - 0.5R)$$ **Equation 2B-4.03**

where:

CN_c = composite runoff curve number
CN_p = pervious runoff curve number
P_{imp} = percent imperviousness
R = ratio of unconnected impervious area to total impervious area.

c. Hydrologic Soil Groups: Most urban areas are only partially covered by impervious surfaces and the soil remains an important factor in runoff estimates. Urbanization has a greater effect on runoff in watersheds with soils having high infiltration rates (sands and gravels) than in watersheds with low infiltration rates (silts and clays) since undeveloped runoff volumes are already elevated.

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into hydrologic soil groups (HSG's) to indicate the minimum rate of infiltration obtained for bare soil after prolonged wetting. The HSG's, which are A, B, C, and D, are one element used in determining runoff curve numbers. The soil classification may be obtained from NRCS soil survey publications and can be obtained from the local NRCS offices for use in estimating soil types. Exhibit A of TR-55 includes a list of soils of the United States and the hydrologic soils group associated with each soil type.

The infiltration rate is the rate at which water enters the soil at the soil surface. It is controlled by surface conditions. HSG also indicates the transmission rate - the rate at which the water moves within the soil. This rate is controlled by the soil profile. The four groups are defined by SCS soil scientists as follows:

1) **Group A:** Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (greater than 0.30 in/hr).
2) **Group B:** Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15 to 0.30 in/hr).
3) **Group C:** Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05 to 0.15 in/hr).
4) **Group D:** Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0 to 0.05 in/hr).
5) **Disturbed Soil Profiles:** Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction, or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. As a result of urbanization, the soil profile may be considerably altered and the listed group classification may no longer apply. In these circumstances, use the following to determine the hydrologic soil group according to the texture of the new surface soil (provided that significant compaction has not occurred).

Table 2B-4.02: Hydrologic Soil Group for Disturbed Soils

HSG	Soil Texture
A	Sand, loamy sand, or sandy loam
B	Silt loam or loam
C	Sandy clay loam
D	Clay loam, silty clay loam, sandy clay, silty clay, or clay

Source: NRCS TR-55

Table 2B-4.03: Runoff Curve Numbers for Urban Areas[1]

Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	CN's for Hydrologic Soil Group			
		A	***B***	***C***	***D***
Fully Developed Urban Areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.):[3]					
Poor condition (grass cover < 50%)	-------------	68	79	86	89
Fair condition (grass cover 50% to 75%)	-------------	49	69	79	84
Good condition (grass cover >75%)	-------------	39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	-------------	98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)	-------------	98	98	98	98
Paved; open ditches (including right-of-way)	-------------	83	89	92	93
Gravel (including right-of-way)	-------------	76	85	89	91
Dirt (including right-of-way)	-------------	72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town homes)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing Urban Areas					
Newly graded areas (pervious areas only, no vegetation)[4]	-------------	77	86	91	94
Idle lands (CN's are determined using cover types similar to those in Table 2B-4.01)					

[1] Average runoff condition and I_a=0.2S
[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using Figures 2B-4.01 or 2B-4.02.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figures 2B-4.01 or 2B-4.02 based upon the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Source: NRCS National Engineering Handbook, Part 630, Chapter 9

Table 2B-4.04: Runoff Curve Numbers for Cultivated Agricultural Lands[1]

Cover Description			CN's for Hydrologic Soil Group			
Cover Type	***Treatment[2]***	***Hydrologic Condition[3]***	***A***	***B***	***C***	***D***
Fallow	Bare Soil	---	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight Row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small Grain	Straight Row (SR)	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured (C)	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured & terraced (C&T)	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close Seeded or Broadcast Legumes or Rotation Meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition and I_a=0.2S
[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[3] Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Source: NRCS National Engineering Handbook, Part 630, Chapter 9

Table 2B-4.05: Runoff Curve Numbers for Other Agricultural Lands[1]

Cover Description		CN's for Hydrologic Soil Group			
Cover Type	***Hydrologic Condition[3]***	***A***	***B***	***C***	***D***
Pasture, grassland, or range - continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow - continuous grass, protected from grazing and generally mowed for hay	---	30	58	71	78
Brush - brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods - grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads - buildings, lanes, driveways, and surrounding lots	---	59	74	82	86

[1] Average runoff condition and I_a=0.2S.
[2] *Poor*: <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.
[3] *Poor*: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
[4] Actual curve number is less than 30; use CN = 30 for runoff computations
[5] CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
[6] *Poor*: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed, but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing and litter and brush adequately cover the soil

2. **SCS Depth of Runoff:** Depth of runoff may be calculated through the SCS Curve Number Method. This method separates total rainfall into direct runoff, retention, and initial abstraction to yield the following equation for rainfall runoff.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$ **Equation 2B-4.04**

where:

Q = Depth of direct runoff, in
P = Depth of 24 hour precipitation, in. for design year storm (e.g. 10 year, 24 hour)
S = Potential maximum retention after runoff begins, in
I_a = Initial abstraction, in

The initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration during the early part of the storm. Interception and surface depression storage may be estimated from cover and surface conditions, but infiltration during the early part of the storm is highly variable and dependent on such factors as rainfall intensity, soil crusting, and soil moisture. Establishing a relationship for I_a

is not easy. Therefore, I_a is assumed to be a function of the maximum potential retention, S. An empirical relationship between I_a and S is expressed as:

$$I_a = 0.2S$$ **Equation 2B-4.05**

Removing I_a and substituting Equation 2B-4.05 into Equation 2B-4.04 gives:

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)}$$ **Equation 2B-4.06**

The potential maximum (S) is related to the soil cover and conditions of the watershed through the CN as follows:

$$S = \frac{1000}{CN} - 10$$ **Equation 2B-4.07**

After determining the CN and calculating the value for S, the total amount of rainfall, P, for the 24 hour storm with the selected return interval must be determined. Values for total rainfall depth by storm duration and return interval are listed in Section 2B-2. These values are inserted into Equation 2B-4.06 to calculate the total depth of runoff from the watershed.

3. **SCS Peak Runoff:** After the total runoff is determined, the SCS Peak Discharge Method may be utilized to determine the peak rate of discharge from the watershed. The equation for the peak discharge is given as:

$$q_p = q_u A_m Q F_p$$ **Equation 2B-4.08**

where:

q_p = peak discharge, cfs
q_u = unit peak discharge, $ft^3/s/mi^2/in$ (csm)
A_m = drainage area, mi^2
Q = runoff, in (from Equation 2B-4.04 above)
F_p = pond and swamp adjustment factor (Table 2B-4.05)

The unit peak flow is calculated with the following equation (graphical depictions are presented in TR-55):

$$q_u = 10^{[C_0+(C_1)(\log t_c)+(C_2)(\log t_c)^2]}$$ **Equation 2B-4.09**

where:

C_0, C_1, C_2 = Coefficients, listed in Table 2B-4.06. These are a function of the 24 hour rainfall distribution type and Ia/P.
t_c = time of concentration (refer to Section 2B-3)
I_a = Initial abstraction (refer to Equation 2B-4.05), in

Source: HEC-22, FHWA

Table 2B-4.06: Coefficients for SCS Peak Discharge Method

I_a/P	C_0	C_1	C_2
0.10	2.55323	-0.61512	-0.16403
0.30	2.46532	-0.62257	-0.11657
0.35	2.41896	-0.61594	-0.08820
0.40	2.36409	-0.59857	-0.05621
0.45	2.29238	-0.57005	-0.02281
0.50	2.20282	-0.51599	-0.01259

Note: Values are for Type II rain distribution, which applies to all of Iowa.

Source: TR-55, USDA

Table 2B-4.07: Adjustment Factor (F_p) for Pond and Swamp Areas that are Spread Throughout the Watershed

Percentage of pond and swamp area	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Source: HEC-22, FHWA

4. SCS Limitations: The SCS methods presented herein are subject to the following limitations.

a. These methods provide a determination of total runoff or peak flow only. If a hydrograph is needed or watershed subdivision is required the Tabular Hydrograph method (Section 2B-5) should be utilized.

b. The watershed must be hydrologically homogenous, that is, describable by one of the CN. Land use, soils, and cover are distributed uniformly throughout the watershed.

c. The watershed may have only one main stream or, if more than one, the branches must have nearly equal time of concentrations.

d. The method cannot perform valley or reservoir routing.
e. The F_p factor can be applied only for ponds or swamps that are not in the t_c flow path.

f. I_a/P values should be between 0.1 and 0.5.

g. This method should only be used if the composite CN is greater than 40.

h. The SCS methods are typically applicable for drainage areas between 0 and 2,000 acres.

D. References

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular, No. 22. Third Ed. 2009.

USDA Natural Resource Conservation Service. *National Engineering Handbook - Part 630. Chapter 9: Hydrologic Soil Cover Complexes*. 2004.

Watershed Routing (Hydrograph Determination)

A. Introduction

Watershed routing is utilized when the watershed contains multiple subbasins and it is desired to add the flows from each subbasin together to determine the combined flow rate at critical locations along the conveyance system. This method follows the flow through the basin and results in the development of an inflow hydrograph. The resulting hydrograph plots the flow rate against the time of the storm event. The most common location where an inflow hydrograph is required is at a stormwater detention basin. (See Section 2G-1 for detention basin design). Two methods for watershed routing are provided in this chapter: Modified Rational Method for Basin Routing and the Tabular Hydrograph TR-55 Method.

B. Modified Rational Method for Basin Routing

1. **Method Description:** The Modified Rational Method can estimate peak flows at critical points in basins with numerous subbasins. The Modified Rational Method can give a triangular and trapezoidal hydrograph for determining storage volumes. To assist the engineer in the calculations, there are numerous computer programs available, such as MODRAT, which is a Modified Rational Method program developed by the Los Angeles County Department of Public Works.

   The basis of the Modified Rational Method (and any hydrograph) is that the area under the hydrograph equals the volume of runoff. For the Modified Rational Method hydrograph, it is assumed that runoff begins at the start of the storm and increases linearly to the peak value (equal to the T_c). The peak runoff is sustained until the event duration has elapsed, and then decreases linearly to zero. For real-world conditions, this is highly unlikely.

   When using the Modified Rational Method, it is recommended that a coefficient be used in order to account for the antecedent moisture conditions of storms with a 25 year, or greater, recurrence interval. This attempts to predict a more realistic runoff volume for major storms. The equation to account for this increased volume is:

   $$Q = (C_a)(C)(i)(A)$$ **Equation 2B-5.01**

Table 2B-5.01: Recommended Antecedent Precipitation Factors for the Rational Method

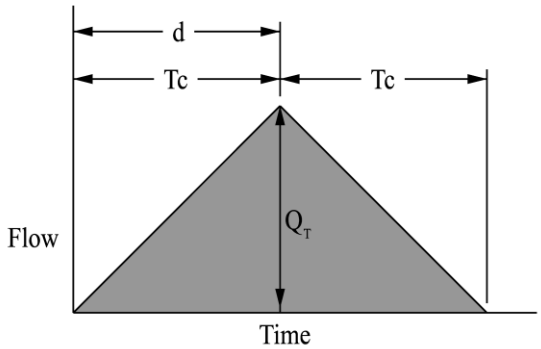
Recurrence Interval (years)	C_a
2 to 10	1.0
25	1.1
50	1.2
100	1.25

Note: The product of C x C_a cannot exceed 1.0.

The time of concentration (T_c), which is the time of travel from the most remote point (in time of flow), determines the largest peak discharge. Therefore, there are two possible approximate hydrographs that can be used for runoff and storage requirements.

If the rainfall duration is equal to the T_c, the approximate hydrograph is a triangle.

Figure 2B-5.01: Modified Rational Method Hydrograph

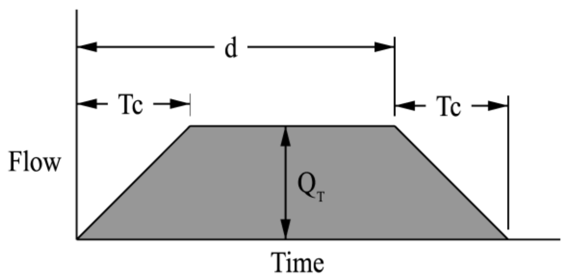


d = Duration of Storm
Q_T = Peak flow rate (=CiA)
T_c = Time of concentration

In this example, the storm duration equals the T_c resulting in a triangular shaped hydrograph.

If the rainfall duration is greater than the T_c, the approximate hydrograph is a trapezoid.

Figure 2B-5.02: Modified Rational Method Hydrograph



d = Duration of Storm
Q_T = Peak flow rate (=CiA)
T_c = Time of concentration

In this example, the storm duration exceeds the T_c resulting in a trapezoidal shaped hydrograph.

For storage volume determination using the Modified Rational Method, see Section 2G-1.

2. **Limitations:** It should be noted that the Modified Rational Method does have limitations. Because this method assumes a constant intensity storm event, and does not recognize soil conditions, the method does not produce a true hydrograph, only an approximation.

   Because of this limitation, the Modified Rational method should be limited to drainage basins of 5 acres or less with no off-site pass-through.

C. Tabular Hydrograph Method

The TR-55 Tabular Hydrograph Method is used for computing discharges from rural and urban areas, using the time of concentration (T_c) and travel time (T_t) from a subarea as inputs. The SCS TR-55 methodology can determine peak flows from areas of up to 2,000 acres, provide a hydrograph for times of concentration between 0.1 to 2 hours, and estimate the required storage for a specified outflow.

This method can develop composite flood hydrographs at any point in a watershed by dividing the watershed into homogeneous subareas. In this manner, the method can estimate runoff from non-homogeneous watersheds; a common occurrence in developed urban areas. The method is especially applicable for estimating the effects of land use change in a portion of a watershed.

1. **Method Description:** The Tabular Hydrograph method is based on a series of unit discharge hydrographs developed by the SCS. The tabular data was developed by computing hydrographs for one-square mile of drainage area for selected T_c's and routing them through stream reaches with a range of T_t's. The resulting values, expressed in cubic feet per second per square mile of watershed per inch of runoff, are summarized in ten tables provided in the SCS TR-55 manual.

   Chapter 5 of TR-55 provides a detailed description for manual calculation with the tabular hydrograph method, in addition to the tables necessary to complete the calculation. The input data required to develop a flood hydrograph by the SCS TR-55 method includes:
   - 24 hour rainfall, in
   - Appropriate rainfall distribution, (I, IA, II, or III) (Iowa is type II)
   - Curve Number (Refer to Section 2B-4)
   - Time of Concentration, T_c, hr.
   - Travel Time, T_t, hr.
   - Drainage Area, sq. mi.

   The 24 hour rainfall amount, rainfall distribution, and the runoff curve number are used in Equations 2B-4.06 and 2B-4.07 to determine the runoff depth in each subarea. The product of the runoff depth times drainage is multiplied times each tabular hydrograph value to determine the final hydrograph ordinate for a particular subarea. Subarea hydrographs are then added to determine the final hydrograph at a particular point in the watershed.

   Calculating runoff hydrographs manually utilizing the tabular method is time consuming, tedious, and rarely done. This calculation is typically completed utilizing user-created spreadsheets, WinTR-55, or other software that utilizes the TR-55 methodology.

2. **Limitation:** The tabular method is used to determine peak flows and hydrographs within a watershed. However, the accuracy of the Tabular Method decreases as the complexity of the watershed increases. The Tabular Method should not be used if any of the following conditions exist:
   - The drainage area of the watershed is greater than 2,000 acres.
   - T_t is greater than 3 hours (largest T_t in tabular hydrograph data)
   - T_c is greater than 2 hours (largest T_c in tabular hydrograph data)
   - Drainage areas of individual subareas differ by a factor of 5 or more

   If any of the above situations exist, NRCS TR-20, or another applicable methodology should be utilized.

D. References

U.S. Department of Agriculture. *Urban Hydrology for Small Watersheds.* Technical Release No. 55. 1975.

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular, No. 22. Third Ed. 2009.

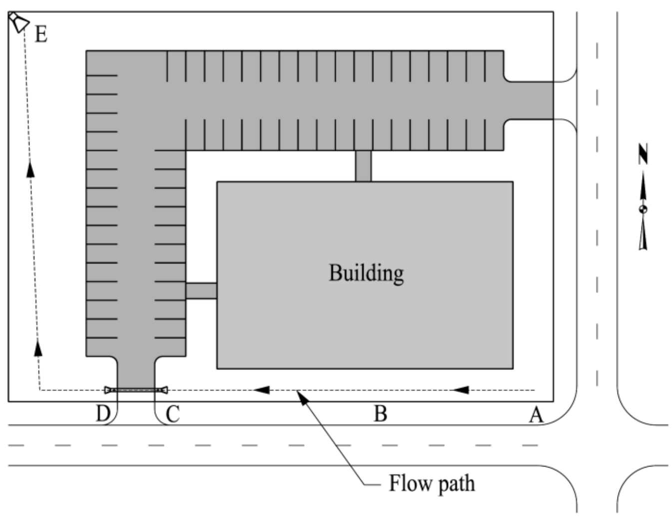
Runoff Examples

A. Rational Method Example

1. **Problem Statement:** A 2 acre commercial site (350 feet by 250 feet) is being developed with a new building and parking lot. The site drains to a culvert located at the northwest corner of the property. The hydraulically most distant point is located at the southeast corner of the property. Runoff from the SE corner of the property flows west, through a driveway culvert under the south drive, and then north to the main culvert. The average slope along this route is 3%. All runoff drains to the northwest corner and the site does not have any off-site drainage.

   Assuming this site is located in Iowa Climactic Section 4, with Group C soils; use the Rational Method to determine the peak runoff from the property.

Figure 2B-6.01: Example Commercial Development



2. **Time of Concentration:** The first step in calculating the peak runoff rate is determining the Time of Concentration. For the Rational Method, the Velocity Method, as described in Section 2B-3, is typically used to calculate T_c.

   The velocity method consists of three components, sheet flow, shallow concentrated flow, and open channel flow.

Table 2B-6.01: Site Conditions for Rational Method Example

Segment	Flow Type	Segment Properties
A-B	Sheet	Dense Grass, Slope = 2.0%, Length = 100’
B-C	Shallow Con. Flow	Grassed Waterway, Slope = 2.0%, Length = 140’
C-D	Pipe Flow	12” RCP, Assume 1/2 pipe flow, Slope = 1.0%, Length = 140’
D-E	Open Channel	Earth channel with short grass, Slope = 2.0%, Length = 275’ Assume a rectangular channel with 6’ bottom and flow depth of 4”

Worksheet 2B-6.01: Time of Concentration (T_c) or Travel Time (T_t)

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☑ Developed

Check one: ☑ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

		Unit	Segment 1	Segment 2	Total
	Segment ID		AB		
1.	Surface description (Table 2B-3.01)		Dense Grass		
2.	Manning's roughness coeff., n (Table 2B-3.01)		0.24		
3.	Flow Length, L (Total 100' max.)	ft	100		
4.	2 year 24 hour rainfall, P_2 (Section 2B-2)	in	3.01		
5.	Land slope, s	ft / ft	0.02		
6.	Travel Time, $T_t = \frac{0.007(nL)^{0.8}}{(\sqrt{P_2})(S)^{0.4}}$, (Eq. 2B-3.03)	hr	0.25	+	= 0.25

Shallow concentrated flow

		Unit	Segment 1	Segment 2	Total
	Segment ID		BC		
7.	Surface description (Figure 2B-3.01)		Grassed waterway		
8.	Flow length, L	ft	140		
9.	Watercourse slope, s	ft / ft	0.02		
10.	Average velocity, V (Fig. 2B-3.01 or Table 2B-3.02)	ft / s	2.3		
11.	Travel Time, $T_t = \frac{l}{3600V}$, (Eq. 2B-3.01)	hr	0.02	+	= .02

Open channel / pipe flow

		Unit	Segment 1	Segment 2	Total
	Segment ID		CD	DE	
12.	Cross sectional flow area, A (Section 2F-2)	ft^2	0.39	2	
13.	Wetted perimeter, P_w (Section 2F-2)	ft	1.57	6.67	
14.	Hydraulic radius, $R = \frac{A}{P_w}$ (Section 2F-2)	ft	0.25	0.30	
15.	Channel slope, s	ft / ft	0.01	0.02	
16.	Manning's roughness coefficient, n		0.013	0.027	
17.	Velocity, $V = \frac{1.49(r^{2/3})(s^{1/2})}{n}$, (Eq. 2B-3.04)	ft / s	4.55	3.5	
18.	Flow length, L	ft	140	275	
19.	Travel Time, $T_t = \frac{l}{3600V}$, (Eq. 2B-3.01)	hr	0.01	+ 0.02	= 0.03
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11 and 19)	**hr**			**0.30**

From Worksheet 2B-6.01, the time of concentration is 0.30 hours (18 minutes).

3. **Runoff Coefficient:** The county soil survey indicates the existing soils are Group C. Because this site is being regraded and developed, it is assumed that the resulting soil profile will more closely resemble Group D soils due to compaction during construction.

   Because the drainage area contains multiple surfaces, a composite runoff coefficient must be determined. The values for the Rational coefficient are provided in Table 2B-4.01. A summary of the surface areas and associated Rational coefficients for the site is provided in Table 2B-6.02 below.

   The 5 year composite runoff coefficient (C_5) for the site is calculated by finding the overall average:

$$C_5 = \frac{(27{,}282 \times 0.95) + (22{,}800 \times 0.95) + (36{,}818 \times 0.5)}{87{,}500} = 0.75$$

   The 100 year is found in a similar manner.

Table 2B-6.02: Summary of Surface Areas for Rational Method Example

Proposed Surface	Area (sf)	Rational Coefficient	
		5 year	***100 year***
Parking Lot and Sidewalk	27,282	0.95	0.98
Building	22,800	0.95	0.98
Lawn (good condition)	36,818	0.50	0.65
Total / Composite	87,500 (2 acres)	0.75	0.83

4. **Peak Runoff:** The Rational method requires three components to calculate peak runoff: runoff coefficient, rainfall intensity, and drainage area. The runoff coefficient was determined in number 3 above and the area was given above as 2 acres. The only missing component is the rainfall intensity (i).

   The rainfall intensity is found in the rainfall depth and intensity tables in Section 2B-2. This site is located in Iowa climactic zone 4 so Table 2B-2.05 is utilized. The time of concentration was calculated as 18 minutes. For design, the T_c is typically rounded down to the next standard duration; in this case is 15 minutes. From Table 2B-2.05, the 5 year and 100 year intensities for a 15 minute T_c are 3.96 and 7.46 inches/hour respectively.

   The peak runoff rate is determined from Equation 2B-4.01 as follows:

   $Q_5 = 0.75 \times 3.96 \times 2.0 =$ 5.9 cfs

   $Q_{100} = 0.83 \times 7.46 \times 2.0 =$ 12.4 cfs

B. SCS Method Example

1. **Problem Statement:** A watershed covers 180 acres in Carroll County, Iowa. The current land use is agricultural with 60 acres in pasture and 120 acres in active corn and soybean production (row crops). The cultivated portion has been contoured and terraced and is farmed utilizing no-till farming practices (crop residue). The entire watershed is in good hydrologic condition and the county soil survey indicates that this area contains group B soils.

   A new, 60 acre development near the upstream end of this watershed is being considered for construction of single family one-acre lots. This development is being proposed in the cultivated portion of the watershed. It is estimated that the development will contain approximately 35% impervious area (streets, driveways, homes, outbuildings, etc.)

   Determine the peak runoff rates for the watershed before and after development.

2. **Curve Number:** The first step is to determine the existing and proposed curve number (CN) for the watershed. CN values are provided in Tables 2B-4.03 through 2B-4.05. The value for row crops, contoured and terraced with crop residue is 70 for a good hydrologic soil condition and soil group B (from Table 2B-4.04). For pasture, the value is 61 (Table 2B-4.05).

   The value for the proposed developed condition must also be obtained. The original land was assessed as a group B soil; however, given the compaction that occurs as a result of mass grading and construction, it is likely that the soil condition will be reduced to a Group C or D soil. A Group C soil is assumed for this example. Table 2B-4.03 includes CN values for 1 acre residential lots; however, these values assume and impervious area of 20%. The assumed impervious are of this development 35% as stated above. Therefore the impervious and pervious (lawn) areas will be assessed separately.

   A composite CN must be determined to represent the average CN of the entire watershed. This is done by determining a weighted average, based upon ground area. This is shown in Worksheets 2B-6.02 and 2B-6.03.

3. **Time of Concentration:** The time of concentration may be determined with either the Velocity or Lag methods. In this example the Lag method, as described in Section 2B-3, will be used.

   Assume the watershed has a flow length of 4,700 feet and an average land slope of 8.0 percent. The example calculation for T_c is shown in Worksheets 2B-6.02 and 2B-6.03.

   For the developed example in Worksheet 2B-6.03, an adjustment for urbanization was applied. This process is necessary when utilizing the lag method in developed areas.

4. **Runoff:** The total runoff, in inches, from the watershed and the peak rate of runoff is then determined as shown in the worksheets below.

Worksheet 2B-6.02: Runoff Curve Number and Runoff - *Existing Conditions*

Project: SCS Example – Existing Conditions	By	Date
Location: Carroll County, Iowa	Checked	Date
Check one: ☑ Present ☐ Developed		

1. Runoff Curve Number

Soil name & hydrologic group (County soil survey)	Cover Description (cover type, treatment and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN[1] Tables 2B-4.03, 4.04, & 4.05	CN[1] Figure 2B-4.01	CN[1] Figure 2B-4.02	Area ☑ac ☐mi^2 ☐%	CN x Area
Marshall, B	Row crops with contouring, terracing, and crop residue.	70			120	8,400
Marshall, B	Pasture, continuous forage	61			60	3,660
				Totals ➡	180	12,060

[1]Use only one CN source per line

CN (weighted) = $\frac{Total\ product}{total\ area} = \frac{12060}{180} = 67$ Use CN ➡ 67

Potential max. retention, S = $\frac{1000}{67} - 10 = 4.9$ (Eq. 2B-4.07) S ➡ 4.9

2. Time of Concentration

Watershed Lag, L = $\frac{4700^{0.8}(4.9+1)^{0.7}}{1900(8.0)^{0.5}} = 0.56$ (Eq. 2B-3.05)

$T_c = \frac{0.56}{0.6} = 0.93$ hr (Eq. 2B-3.05) T_c ➡ 0.93

3. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	5	100	
Rainfall, P (24-hour) ('D' from tables in Section 2B-2)......	in	3.74	7.67	
Runoff, Q = $\frac{(P-0.2S)^2}{(P+0.8S)}$ (Eq. 2B-4.06)..................................	in	1.0	3.8	

4. Peak Runoff Rate

		Storm #1	Storm #2	Storm #3
Ratio of Initial abstraction to Rainfall $\frac{I_a}{P} = \frac{0.2\times S}{P}$............................		0.26	0.13	
Coef. for Peak Discharge (Table 2B-4.06 - interpolated)	C_0	2.48290	2.54004	
	C_1	-0.62108	-0.62630	
	C_2	-0.12606	-0.15691	
Unit peak runoff, q_u (Eq. 2B-4.09)	$ft^3/s/mi^2$	318	363	
Peak Runoff $q_p = q_u \times \frac{Area(ac)}{640\ ^{ac}/_{mi^2}} \times Q \times F_p$ (Eq. 2B-4.08)	cfs	**89**	**388**	

Worksheet 2B-6.03: Runoff Curve Number and Runoff - *Proposed Conditions*

Project: SCS Example – Existing Conditions	By	Date
Location: Carroll County	Checked	Date
Check one: ☐ Present ☑ Developed		

1. Runoff Curve Number

Soil name & hydrologic group (County soil survey)	Cover Description (cover type, treatment and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN[1] Tables 2B-4.03, 4.04, & 4.05	CN[1] Figure 2B-4.01	CN[1] Figure 2B-4.02	Area ☑ac ☐mi² ☐%	CN x Area
Marshall, B	Row crops with contouring, terracing, and crop residue.	70			40	2,800
Marshall, B	Pasture, continuous forage	61			60	3,660
Marshall, C	Open space, lawn in good condition	74			52	3,848
Marshall, C	Impervious area (streets, roofs, etc).	98			28	2,744
				Totals ➡	180	13,052

[1]Use only one CN source per line

CN (weighted) = $\frac{Total\ product}{total\ area} = \frac{13052}{180} = 73$ Use CN ➡ 73

Potential max. retention, S = $\frac{1000}{73} - 10 = 3.7$ (Eq. 2B-4.07) S ➡ 3.7

2. Time of Concentration

Watershed Lag, L = $\frac{4700^{0.8}(3.7+1)^{0.7}}{1900(8.0)^{0.5}} = 0.48$ (Eq. 2B-3.05)

$T_c = \frac{0.48}{0.6} = 0.80$ hr (Eq. 2B-3.05)

For lag method, adjust for urbanization: % impervious = 28 ac / 180 ac = 16%
From Figure 2B-3.03, Imp. Factor = 0.9. No channel improvements assumed.
$T'_c = 0.80 \times 1.0 \times 0.9 = 0.72$ (Eq. 2B-3.07)

T_c ➡ 0.72

3. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	5	100	
Rainfall, P (24-hour) ('D' from tables in Section 2B-2)	in	3.74	7.67	
Runoff, $Q = \frac{(P-0.2S)^2}{(P+0.8S)}$ (Eq. 2B-4.06)	in	1.3	4.5	

4. Peak Runoff Rate

		Storm #1	Storm #2	Storm #3
Ratio of Initial abstraction to Rainfall $\frac{I_a}{P} = \frac{0.2\times S}{P}$		0.20	0.10	
Coef. for Peak Discharge (Table 2B-4.06 - interpolated)	C_0	2.50928	2.55323	
	C_1	-0.61885	-0.61512	
	C_2	-0.1403	-0.16403	
Unit peak runoff, q_u (Eq. 2B-4.09)	ft³/s/mi²	390	438	
Peak Runoff $q_p = q_u \times \frac{Area(ac)}{640\ ^{ac}/_{mi^2}} \times Q \times F_p$ (Eq. 2B-4.08)	cfs	**143**	**554**	

General Information for Pavement Drainage and Intake Capacity

A. Introduction

Effective drainage of pavements is essential to maintaining the desired level of service and to traffic safety. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter standing water.

Designing pavements to drain requires consideration of surface drainage, gutter flow, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface.

This chapter presents design guidance for the design of these elements. Most of the information presented is this section is based upon FHWA's Hydraulic Engineering Circular No. 22 (HEC-22), Urban Drainage Design Manual. Designers may refer to this document for additional information.

B. Design Criteria

Two of the more significant variables considered in the design of highway pavement drainage are the frequency of the design runoff event and the allowable spread of water on the pavement. The design criteria for these requirements are summarized in Section 2A-1.

In addition to the storm frequency and allowable spread, the slope of the pavement also directly affects the design of the pavement drainage and the intake spacing.

Together, these three criteria are the key elements in designing pavement drainage and determining intake spacing. A summary of the importance of each is provided below.

1. **Stormwater Spread:** The objective of roadway storm drainage design is to provide for safe passage of vehicles during the design storm event. The design of a drainage system for a curbed urban roadway is to collect runoff in the gutter and convey it to the stormwater intakes in a manner that provides reasonable safety for traffic at a reasonable construction cost. As spread from the curb increases, the risk of traffic accidents and delays increases.

   Due to the increased traffic volume and vehicle speed, water on traffic lanes of higher classification roadways poses more risk than for lower classification roadways. Because of the increased risk, water encroaching into the traffic lanes is less tolerable on these roadways and the additional cost of controlling the spread is justified. This is reflected in the stormwater spread criteria described in Section 2A-1.

2. **Design Frequency:** Stormwater spread should be checked for both the minor and major storm events. As described in Section 2A-1, the minor storm is generally considered a 2 to10 year recurrence event while the major storm is considered a 50 or 100 year storm. Due to the decreased frequency of the major storm, an increased spread into the traveled way is tolerated.

3. **Pavement Slopes:** Both the longitudinal slope and cross slope of the pavement directly impact the width of the stormwater spread and the resulting intake spacing.

   a. **Longitudinal Slopes:** A minimum longitudinal grade is more important for an urban roadway (with a curb) than for a rural roadway (with no curb) since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge. This can create a potential for unexpected hydroplaning and loss of vehicle control.

      As recommended in Section 5C-1, the desirable minimum gutter grade is 0.6%. The minimum gutter grade is 0.5%. Grades of 0.4% may be allowed in certain circumstances. While some publications indicates that grades as flat as 0.3% are allowable, constructing pavements this flat becomes difficult and often results in “bird baths” in the pavement.

      Special attention to drainage must be provided at vertical curves. Both crest and sag vertical curves that have a grade change from positive to negative (or vice versa) contain a level area at some point along the curve. Generally, as long as a grade of 0.30% is provided within 50 feet of the level area, no drainage problems develop. This criterion corresponds to a K value of 167. Refer to Section 5C-2 for additional information regarding vertical curves.

   b. **Cross (Transverse) Slopes:** Section 5C-1 provides the minimum cross slope requirements for urban and rural roadways. In general, for streets with three or fewer travel lanes, the cross slope should be 2%. For roadways with four or more lanes, the cross slope of the inside lanes, including left turn lanes, should be 2%. In order to reduce stormwater spread, the cross slope of the outside lanes should be 3%, if both lanes slope in the same direction.

      At intersections and other cross-slope transition areas where the longitudinal grade drains toward the direction of decreasing cross slope, care must be taken to ensure that the transition length is long enough to prevent trapping water or reducing the longitudinal slope below the recommended minimum.

$$TL = \frac{(S_L - S_{Lm}) \times P_w}{\Delta T_s}$$

Equation 2C-1.10

where:

TL = minimum transition length, ft
S_L = longitudinal slope of the mainline pavement, ft/ft
S_{Lm} = min. desirable longitudinal slope through transition (typically 0.5% or greater), ft/ft
P_w = pavement width, ft.
ΔS_x = change in cross slope through transition, ft/ft

C. References

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22. Third Ed. 2009.

Flow in Gutters

A. Introduction

A pavement gutter is defined as a section of pavement adjacent to the roadway that conveys stormwater runoff from the pavement and adjacent areas behind the back of curb. Conventional gutter sections may have a straight cross slope, a composite cross slope where the gutter slope varies from the pavement cross slope, or a parabolic section. The standard SUDAS gutter section consists of a straight cross slope and is the type discussed below.

Most of the information presented is this section is based upon FHWA's Hydraulic Engineering Circular No. 22 (HEC-22), Urban Drainage Design Manual. Designers may refer to this document for additional information, including the design of composite, parabolic, and other types of gutter sections.

B. Gutter Capacity and Spread

Gutter flow calculations are necessary to establish the spread of water on the adjacent parking lane or traveled way. A modification of the Manning's equation can be used for computing flow in triangular channels. The modification is necessary because the hydraulic radius in the equation does not adequately describe the gutter cross-section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting equation is:

$$Q = \left(\frac{0.56}{n}\right)(S_x)^{1.67}(S_L)^{0.5}T^{2.67}$$ **Equation 2C-2.01**

or in terms of T:

$$T = \left[\frac{Q \times n}{(0.56)\left({S_x}^{1.67}\right)\left({S_L}^{0.5}\right)}\right]^{0.375}$$ **Equation 2C-2.02**

where:

Q = Flow rate, cfs
T = Width of flow (spread), ft
n = Manning's coefficient (see Table 2C-2.01)
S_x = Cross slope of pavement, ft/ft
S_L = Longitudinal slope of pavement, ft/ft

Source: FHWA HEC-22

Equations 2C-2.01 and 2C-2.02 neglect the resistance of the curb face since this resistance is negligible.

Table 2C-2.01: Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Asphalt Pavement	
Smooth texture (surface course)	0.013
Rough texture (base course or open graded mix)	0.016
Concrete Gutter with Asphalt Pavement	
Smooth	0.013
Rough	0.015
Concrete Pavement	
Float Finish	0.014
Broom finish (typical for most streets value)	0.016
Concrete Gutter, Troweled Finish	0.012
For gutters with small slope, where sediment may accumulate, increase values of "n" above by	0.002

C. Flow in Sag Vertical Curves

As gutter flow approaches the low point in a sag vertical curve, the flow can exceed the allowable design spread values as a result of the continually decreasing gutter slope. The spread in these areas should be checked to ensure it remains within tolerable limits. If the computed spread exceeds design values, additional intakes should be provided to reduce the flow as it approaches the low point.

D. Gutter Flow Times

The flow time in gutters is an important component of the time of concentration for the contributing drainage area to an inlet. To find the gutter flow component of the time of concentration, a method for estimating the average velocity in a reach of gutter is needed. The velocity in a gutter varies with the flow rate and the flow rate varies with the distance along the gutter (i.e. both the velocity and flow rate in a gutter vary). The time of flow can be estimated by use of an average velocity obtained by integration of the Manning's equation for the gutter section with respect to time.

$$V = \left(\frac{1.11}{n}\right)(S_L)^{0.5}(S_x)^{0.67}{T_A}^{0.67}$$ **Equation 2C-2.03**

where:

V = Velocity in a triangular channel (gutter), ft/s
T_A = Average width of flow (spread) between intakes, ft
n = Manning's coefficient (see Table 2C-2.01)
S_x = Cross slope of pavement, ft/ft
S_L = Longitudinal slope of pavement, ft/ft

Source: FHWA HEC-22

When using Equation 2C-2.03 to determine the average flow velocity through a gutter section upstream of an intake, or between two intakes with bypass flow, the average spread (T_A) through the flow section should be used.

E. References

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22. Third Ed. 2009.

Intake Design and Spacing

A. Introduction

Storm sewer intakes are the main access points by which urban runoff enters the storm sewer system. In fact, the storm sewer intake is an important element of the design in its own right. The hydraulics of flow into an intake are based on principles of weir and orifice flow, modified by laboratory and field observation of entrance losses under controlled conditions.

Curb and gutter intakes are installed along street sections having curbs and gutters to intercept stormwater runoff and to allow its passage into a storm sewer. Intakes can be located at low points (sumps), directly upstream from street intersections, and at intermediate locations. The spacing of these intermediate curb intakes depends on several criteria but is usually controlled by rate of flow and the permissible water spread toward the street crown. The classification of road is also important since the greater the speed and volume of traffic, the greater the potential for hydroplaning. On the other hand, it is also considered acceptable practice to allow some periodic and temporary flooding of low speed, low volume streets (see Section 2A-3 for criteria).

B. Definitions

Bypass Flow: Bypass flow is defined as the flow in the gutter that is not intercepted by a given intake. Bypass flow is calculated by subtracting the allowable capacity of the given intake from the design flow assigned to that intake. Bypass flow is added to the design storm runoff for the next downstream intake. As a minimum, intakes at a low point will have design capacity equal to the assigned storm discharge plus upstream bypass flows.

Design Flow: Design flow is defined as the quantity of water at a given point calculated from the design storm runoff. For gutter applications, design flow should include bypass flow from upstream intakes.

Frontal Flow: The portion of the flow that passes over the upstream side of a grate.

Low Flow: Low flow is defined as the peak runoff rate from the one-year storm event.

Side-flow Interception: Flow that is intercepted along the side of a grate intake, as opposed to frontal interception.

Splash-over: Portion of the frontal flow at a grate that skips or splashes over the grate and is not intercepted.

C. Intake Types

A storm sewer intake is an opening into a storm sewer system for the entrance of surface storm runoff. There are four basic types of intakes:

1. **Grate Intakes:** Grate intakes, as a class, perform satisfactorily over a wide range of gutter grades. Grate intakes generally lose capacity with increase in grade, but to a lesser degree than curb opening intakes. The principal advantage of grate intakes is that they are installed along the roadway where the water is flowing. Their principal disadvantage is that they may be clogged by floating trash or debris. Additionally, where bicycle traffic occurs, grates should be bicycle safe.

2. **Curb Opening:** Curb-opening (open-throat) intakes are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening intakes decreases as the longitudinal gutter grade steepens. Consequently, the use of curb-opening intakes is recommended in sags and on grades less than 3%. Of course, they are bicycle safe as well.

3. **Combination Intakes:** Combination intakes provide the advantages of both curb opening and grate intakes. This combination results in a high capacity intake that offers the advantages of both grate and curb opening intakes.

4. **Slotted Drain Intakes:** Slotted drain intakes can be used in areas where it is desirable to intercept sheet flow before it crosses onto a section of roadway. Their principal advantage is their ability to intercept flow over a wide section. However, slotted intakes are very susceptible to clogging from sediments and debris, and are not recommended for use in environments where significant sediment or debris loads may be present. Slotted intakes on a longitudinal grade do have the same hydraulic capacity as curb openings when debris is not a factor. Slotted drain intakes are not commonly utilized within the public right-of-way; therefore, the detailed design for these intakes is not included herein. For additional information on slotted drain intakes, refer to HEC-22.

D. Intake Capacity

The capacity of an intake is decreased by such factors as debris plugging, pavement overlaying, etc. Therefore, the allowable capacity of an intake is determined by applying the applicable reduction factor from the following table to the theoretical capacity calculated from the design procedures outlined in this section. These reduction factors are based on vane grates, which are required on all curb grate intakes within the street. Other intake grates may be approved by the Jurisdictional Engineer outside of the street right-of-way. The Iowa DOT normally requires curb opening intakes on primary roads.

Table 2C-3.01: Reduction Factors to Apply to Intakes

Figure No.[1]	Location	Reduction Factor[2]	Intake Description
6010.501, 6010.502, 6010.503, and 6010.504	Continuous Grade	90% Vane Grates with Curb	Single Grate with Curb Opening
	Low Point	80% Vane Grates with Curb	
6010.505 and 6010.506	Continuous Grade	90% Vane Grates with Curb	Double Grate with Curb Opening
	Low Point	80% Vane Grates with Curb	
6010.507 and 6010.508	Continuous Grade	80% Curb Only (No Grate)	Single Open-throat
	Low Point	70% Curb Only (No Grate)	
6010.509 and 6010.510	Continuous Grade	80% Curb Only (No Grate)	Double Open-throat
	Low Point	70% Curb Only (No Grate)	
6010.501 and 6010.502 (Driveway Grate)	Continuous Grade	75% Grate Only (No Curb Opening)	Single Grate Only
	Low Point[3]	50% Grate Only (No Curb Opening)	

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.

[2] Minimum reduction factor is to be used to reduce intake capacity.

[3] Use of driveway grates at low points is discouraged due to their tendency to become plugged with debris and flood the surrounding area. Obtain permission of the Jurisdictional Engineer prior to placing a driveway grate in a low point. If allowed, the Jurisdictional Engineer may also require installation of standard curb intake(s) immediately upstream of the driveway.

E. Design of Intakes On-grade

1. **Intake Efficiency:** Intake interception capacity (Q_i) is the flow intercepted by an intake under a given set of conditions. The efficiency (E) of an intake is the percent of the total flow that the intake will intercept for those conditions. The efficiency of an intake is dependent on the cross slope, longitudinal slope, total gutter flow, and pavement roughness. Efficiency is defined by the following equation:

$$E = \frac{Q_i}{Q_t}$$ **Equation 2C-3.01**

where:

E = intake efficiency
Q_t = total gutter flow, cfs
Q_i = intercepted gutter flow, cfs

Flow that is not intercepted by an intake is termed carryover or bypass flow and is defined by:

$$Q_b = Q_t - Q_i$$ **Equation 2C-3.02**

where:

Q_b = flow that is not intercepted by the intake and must be included in the evaluation of downstream gutters, channels, and intakes.

The interception capacity of all intake configurations increases with increasing flow rates while intake efficiency generally decreases with increasing flow rates. Factors affecting gutter flow also affect intake interception capacity. The depth of water next to the curb is the major factor in the interception capacity of both grate intakes and open-throat intakes.

The interception capacity of a grate intake depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter.

Interception capacity of an open-throat intake is largely dependent on flow depth at the curb and curb opening length. Flow depth at the curb and consequently, open-throat intake interception capacity and efficiency, can be increased by the use of local gutter depression at the curb-opening.

The interception capacity of a combination intake, consisting of a grate placed alongside an open-throat section, does not differ significantly from that of a grate alone. Interception capacity and efficiency are computed by neglecting the curb opening.

Intakes on-grade should be designed to intercept a minimum of 50% of the design flow.

2. **Grate Intakes (On-grade):** In order to determine the capacity of an intake on-grade, the amount of frontal flow (flowing perpendicularly over the grate), and side flow (flowing longitudinally along the side of the grate) must be determined.

The ratio of frontal flow to total gutter flow (E_0) for straight cross slope is expressed by the following equation:

$$E_0 = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$ **Equation 2C-3.03**

where:

E_0 = ratio of frontal flow to total gutter flow
W = width of depressed gutter or grate, ft
T = total spread of water, ft

At low velocities, all of the frontal flow passes over the grate and is intercepted by the intake. As the longitudinal slope of the gutter is increased, the velocity of the flow also increases until the flow begins to skip or splash over the grate, reducing the efficiency of the grate. The velocity at which this occurs is termed the splash-over velocity, and is dependent upon the design of the grate and the length of the grate.

The splash-over velocity for the SUDAS style intake grates is indicated in Table 2C-3.01.

Table 2C-3.02: Splash-over Velocity for SUDAS Intake Grates

Figure No.[1]	**Casting Type**	**Typical Use**	**Splash-over Velocity, fps**		
			Single	***Double***	***Triple***
6010.603	Type Q	Driveway	1.5	2.4	2.9
6010.603	Type R	Combination / Median	7.4	11.4	16.2
6010.603	Type S	Combination / Median	8.3	13.0	20.7

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.

Source: Type Q - Colorado State University, 2009 (CDOT Type 13)
Type R/S - HEC-22 (Curved vane) (CDOT Type 16)

The ratio of frontal flow intercepted to total frontal flow (R_f) or frontal flow interception efficiency is expressed by:

$$R_f = 1 - 0.09(V - V_0)$$ (see note below) **Equation 2C-3.04**

where:

V = velocity of flow in the gutter, ft/s
V_0 = gutter velocity where splash over first occurs, ft/s

Note: R_f cannot exceed 1.0. if V is less than V_0, R_f=1 – meaning that all flow is intercepted. If V is greater than V_0, R_f is less than 1, meaning that only a portion of the flow is intercepted.

In addition to frontal flow, the intake also intercepts a portion of the side flow, flowing adjacent to the intake. Only a small portion of the side flow is intercepted.

The ratio of side flow intercepted to total side flow (R_s) or side flow interception efficiency is expressed by:

$$R_s = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)}$$ (see note below) **Equation 2C-3.05**

where:

V = velocity of flow in the gutter, ft/s
L = length of the grate, ft
S_x = cross slope, ft/ft

Note: R_f cannot exceed 1.0. If V is less than V_0, R_f = 1 meaning that all flow is intercepted. If V is greater than V_0, R_f is less than 1, meaning that only a portion of the flow is intercepted.

The efficiency (E) of a grate is expressed as:

$$E = R_f E_0 + R_s(1 - E_0)$$ **Equation 2C-3.06**

The interception capacity (Q_i) of a grate intake on-grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ_t = Q_t\left[R_f E_0 + R_s(1 - E_0)\right]$$ **Equation 2C-3.07**

3. **Open-throat Intakes (On-grade):** Open-throat intakes are effective in draining pavements where the flow depth at the curb is sufficient for the intake to perform efficiently. Open-throat intakes are less susceptible to clogging and offer little interference to traffic operations. They are a viable alternative to grates where grates would be in traffic lanes or would be hazardous to pedestrians or bicycles.

   The length of open-throat intakes required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = 0.6Q_t^{0.42}S_L^{0.3}\left(\frac{1}{nS_x}\right)^{0.6}$$ **Equation 2C-3.08**

where:

L_T = length of throat opening required to intercept 100% of the gutter flow, ft
S_x = cross slope in ft/ft
n = Manning's coefficient for the pavement
Q_t = total gutter flow, cfs
S_L = longitudinal slope, ft/ft

The efficiency of an open-throat intake shorter than the length required for total interception is expressed by:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$ **Equation 2C-3.09**

where:

L = length of throat opening, ft

Most open-throat intakes incorporate a locally depressed gutter section through the length of the throat opening. This depression aids in increasing the interception capacity of the intake. For depressed open-throat intakes the interception capacity can be found by use of an equivalent cross slope (S_e) in the following equation:

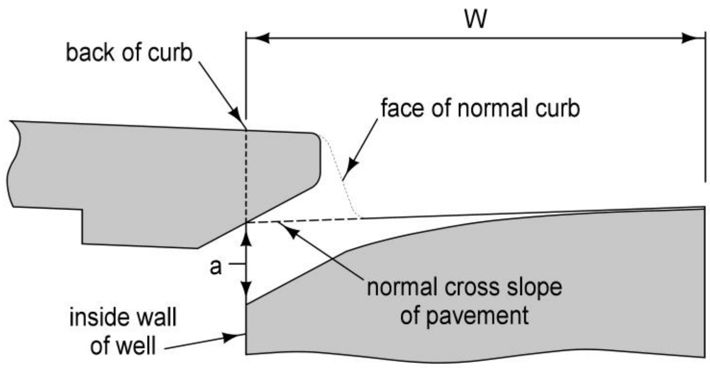
$$S_e = S_x + S'_w E_0$$ **Equation 2C-3.10**

where:

S'_w = cross slope of the gutter measured from the cross slope of the pavement = a/W, ft/ft
a = gutter depression, ft
for standard SUDAS/Iowa DOT open-throat intakes, a = 6"
W = width of depressed gutter section, ft
for standard SUDAS/Iowa DOT open-throat intakes, W= 3' typical and 1.5' min.
E_0 = Ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the intake.
S_x = roadway cross slope, ft/ft

Note: S_e can be used to calculate the length of the throat opening by substituting S_e for S_x in Equation 2C-3.08

Figure 2C-3.01: Open-throat Intake Depression - On-grade



4. **Combination Intakes (On-grade):** The interception capacity of a combination intake with the open throat segment immediately behind the grate is determined by neglecting the open-throat portion and treating the intake as a grate.

F. Design of Intakes in Sag Locations

Intakes in sag locations operate as weirs under low-head conditions and orifices at greater depths. When grate head is developed they function as an orifice. Flow may fluctuate between weir and orifice control depending on the grate size, grate configuration, or the curb-opening height. At depths between those at which weir flow definitely prevails and those at which orifice flow definitely prevails, flow is in a transition stage. At these depths, control is ill-defined and flow may fluctuate between weir and orifice control. Design procedures presented here are based on a conservative approach to estimating the capacity of intakes in sump locations.

The efficiency of intakes in passing debris is critical in sag locations because all runoff that enters the sag must be passed through the intake. Total or partial clogging of intakes in these locations can result in hazardous ponded conditions. Grate intakes alone are not recommended for use in sag locations because of the tendencies of grates to become clogged. Combination intakes or open-throat intakes are recommended for use in these locations.

1. **Grate Intakes in Sags:** A grate inlet in a sag location operates as a weir to depth dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates.

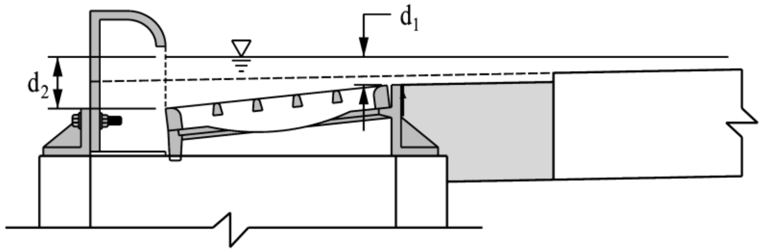
   The capacity of a grate intake operating as a weir is:

   $$Q_i = 3.0Pd^{1.5}$$ **Equation 2C-3.11**

   where:

   Q_i = Grate intake capacity, cfs
   P = Perimeter of the grate disregarding the side against the curb, ft
   d = average depth across the grate, ft (see Figure 2C-3.02 below)

Figure 2C-3.02: Average Depth for Grate Intakes



Average depth (d) is determined as follows:

$$d = \frac{d_1 + d_2}{2}$$

The capacity of a grate intake operating as an orifice is:

$$Q_i = 0.67A_g(2gd)^{0.5}$$ **Equation 2C-3.12**

where:

A_g = Clear opening of the grate, ft^2
g = gravitational constant = 32.16 ft/s^2

In order to determine if an intake is operating under weir flow or orifice flow, both equations should be solved for a given depth. The equation resulting in the lowest calculated flow determines the control type.

Figure No.[1]	Grate Type	Description	Perimeter[2,4] P (feet)	Open Area[4] A_g, (sq-ft)	Weir to Orifice[3] Transition (feet)
6010.603	Type Q	Driveway	9.91	2.62	0.4
6010.603	Type R	Curb Inlet with Vane	5.86	1.95	0.5
6010.603	Type S	Median Barrier	7.52	2.3	0.4
6010.604	Type 3A	Beehive for 18” RCP	5.2	1.2	0.4
6010.604	Type 3B	Beehive for 24” RCP	6.77	1.64	0.5
6010.604	Type 4A	Flat Round for 18” RCP	5.2	0.8	0.3
6010.604	Type 4B	Flat Round for 24” RCP	6.82	1.29	0.3
6010.604	Type 4C	Flat Round for 30” RCP	8.41	2.2	0.5
6010.604	Type 4D	Flat Round for 36” RCP	9.99	2.93	0.5
6010.604	Type 5	Stool Type for 24” to 30” RCP	6	3.06	0.9
6010.604	Type 6	30” x 42” Rectangular	10.54	2.91	0.5

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.
[2] Perimeter length disregards side against curb for curb inlets.
[3] This is the approximate depth at which the intake transitions from weir flow to orifice flow and should be verified by the designer.
[4] Average of Neenah Foundry and East Jordan Iron Works values.

2. **Open-throat Intakes in Sags:** The capacity of an open-throat intake in a sag depends on the water depth at the curb, the length of the throat opening, and the height of the throat opening. The intake operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The equation for the interception capacity of a depressed open-throat intake operating as a weir is:

$$Q_i = 2.30(L + 1.8W)d^{1.5}$$ **Equation 2C-3.13**

where:

L = Length of curb opening, ft
W = Lateral width of depression, ft
d = depth at curb measured from the normal cross slope (i.e. d= T x S_x)

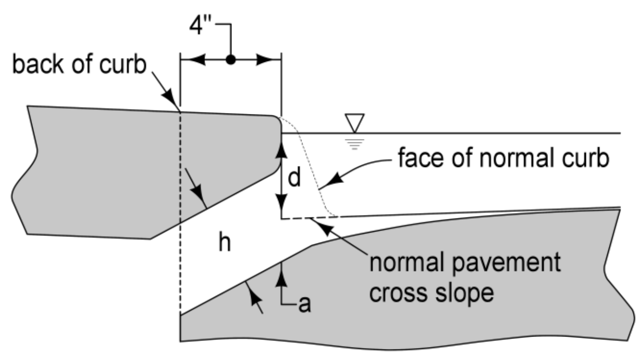
The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation 2C-3.13 for a depressed open-throat intake is:

$$d \leq h + a/12$$ **Equation 2C-3.14**

where:

h = height of curb opening, ft
a = depth of depression, in
for standard SUDAS/Iowa DOT open-throat intakes, a = 4"

Figure 2C-3.03: Open-throat Intake Depression - in Sag



The weir equation for open-throat intakes without depression is:

$$Q_i = 3.0Ld^{1.5}$$ **Equation 2C-3.15**

Open-throat intakes operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity can be computed by Equations 2C-3.16 or 2C-3.17. These equations are applicable to both depressed and undepressed open-throat intakes. The depth at the intake includes any gutter depression.

$$Q_i = 0.67hL(2gd_0)^{0.5}$$ **Equation 2C-3.16**

or

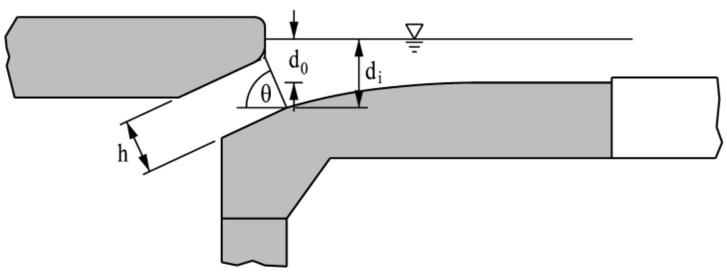
$$Q_i = 0.67A_g\left[2g\left(d_i - \left(\frac{h}{2}\right)\right)\right]^{0.5}$$ **Equation 2C-3.17**

where:

d_0 = Effective head on the center of the orifice throat, ft - (see note below)
d_i = Depth at the lip of the curb opening, ft (see Figure 2C-3.04)
L = Length of the open-throat section, ft
A_g = Clear area of opening, ft^2
h = Height of open-throat orifice, ft

Note: the configuration of the SUDAS/Iowa DOT style intakes is an "inclined throat" according to HEC-22. Based upon this configuration HEC-22 provides the following definition: **$d_0 = d_i - (h/2) \sin\theta$**

Figure 2C-3.04: Standard SUDAS/Iowa DOT Open-throat Curb Section



The following apply for SUDAS/ Iowa DOT open-throat intakes:

h = 5"
θ = 66.7°
(h/2)sin θ = 2.3" = 0.19'
A_g = 1.67 ft^2 - single intake
A_g = 3.33 ft^2 - double intake

Unless otherwise approved by the Jurisdictional Engineer, intakes at low points or on dead-end streets on downgrades should be designed to intercept 100 percent of the design flow

3. **Combination Intakes in Sags:** Combination intakes consisting of a grate and open-throat curb section are recommended for use in sags and locations where hazardous ponding can occur because of their superior hydraulic capacity and debris handling capabilities.

   The interception capacity of a combination intake, where the open-throat section is equal in length to, and immediately behind, the grate, is essentially equal to that of a grate intake alone operating under weir flow conditions. In orifice flow, the capacity of a combination intake is equal to the capacity of the grate plus the capacity of the curb opening.

   Where the depth at the curb is such that orifice flow occurs, the interception capacity of the intake is computed by adding equations 2C-3.12 and 2C-3.16.

$$Q_i = 0.67A_g(2gd)^{0.5} + 0.67hL(2gd_0)^{0.5}$$ **Equation 2C-3.18**

where:

A_g = Clear opening of the grate, ft^2
g = Gravitational constant = 32.16 ft/s^2
d = Depth at the curb, ft
H = Height of the open-throat orifice, ft
L = length of open-throat section, ft
d_0 = effective depth at the center of the open-throat orifice, ft

G. Storm Sewer Structure Requirements

1. **Manholes or Intakes:** Manholes or intakes are required under the following conditions:

   a. At the end of each sewer line.

   b. At all changes in pipe size, elevation and grade, or alignment, and at all bends.

   c. At all sewer pipe intersections, except where the size of the storm sewer conduit (54 inches diameter or greater pipe) eliminates the need for a manhole. Manholes are required for 54 inches or greater pipes when direct access is desired every 400 feet.

   d. At all sewer pipe intersections and at intervals not exceeding 400 feet. If owner has adequate cleaning equipment, the allowable spacing may be increased to 500 feet for sewers 24 inches and larger.

2. **Openings:**

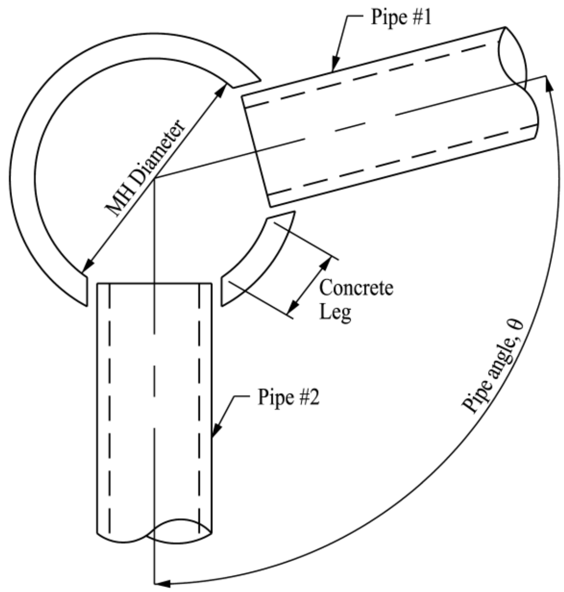
   a. Standard: The minimum size for a manhole is 48 inches in diameter. Jurisdictions require concentric manholes, without built-in steps, with the manhole opening over the centerline of the pipe or on an offset not to exceed 12 inches. Some Jurisdictions may allow for eccentric manholes.

   b. Special: For square or rectangular manholes, the manhole openings should be over the centerline of the pipes or on an offset not to exceed 12 inches. The distance from the centerline of the manhole opening to the face of the inside manhole wall should not exceed 30 inches to better facilitate video inspection and maintenance equipment. This may require more than one manhole opening.

   c. Determining Diameters: When utilizing circular precast manholes, it is necessary to determine the diameter required to maintain the structural integrity of the manhole. As a general rule, a minimum concrete leg of 6 inches should remain between the manhole blockouts for adjacent pipes. Determining the required manhole diameter to provide this minimum distance may be done as follows:

      1) Determine the diameters of, and the angle between, the two pipes in question. If more than two pipes connect at the manhole, the adjacent pipes with the critical configuration (i.e. smallest angle and largest pipes) should be selected. If the critical configuration is not apparent, calculations may be required for all adjacent pipes.

Figure 2C-3.05: Manhole Sizing Requirements



2) Determine the blockout diameter. The blockout is the opening provided in the manhole for the pipe. Blockout dimensions are based on the outside diameter of the pipe. For storm sewer, a circular or doghouse type opening is provided with additional clearance to allow for the insertion of the pipe and sufficient space to accommodate placement of concrete grout in the opening. Typical blockout dimensions for various pipe sizes and materials are given in Table 2C-3.04 below.

Table 2C-3.04: Manhole Blockout Sizes

Pipe Diameter (inches)	**Manhole Blockout (inches)**		
	RCP	***PVC***	***DIP***
12	21	16	16
14	N/A	16	18
15	24	19	N/A
16	N/A	N/A	20
18	28	22	23
20	N/A	N/A	24
21	31	25	N/A
24	35	28	29
27	38	31	N/A
30	42	35	36
33	47	N/A	N/A
36	48	42	41
42	57	N/A	N/A
48	64	N/A	N/A
54	71	N/A	N/A
60	78	N/A	N/A

3) Determine the diameter of the manhole required to provide the minimum concrete leg dimension. This diameter may be calculated with the following equation:

$$MH_d = \frac{BO_1 + BO_2 + 2CL}{\theta \times (\pi/_{180})}$$ **Equation 2C-3.19**

where:

MH_d = Manhole diameter, in
BO = Blockout diameter, in
CL = Minimum concrete leg length, in (typically 6 inches)
θ = Angle between pipe centerlines, degrees

4) Round the minimum manhole diameter calculated, up to the next standard manhole size (48 inches, 60 inches, 72 inches, 84 inches, 96 inches, 108 inches, or 120 inches).
5) Verify that the manhole diameter calculated is sufficient for the largest pipe diameter (See Table 2C-3.04).

Table 2C-3.04: Minimum Manhole Diameter Required for Pipe Size

Pipe Diameter (inches)	**Minimum Manhole Diameter (inches)**		
	RCP	***PVC***	***DIP***
8	N/A	48	48
10	N/A	48	48
12	48	48	48
14	N/A	N/A	48
15	48	48	N/A
16	N/A	N/A	48
18	48	48	48
20	N/A	N/A	48
21	48	48	N/A
24	48	48	48
27	*60	48	N/A
30	*60	*60	*60
33	*60	N/A	N/A
36	*60	*60	*60
42	*72		
48	*84		
54	*96		
60	*96		

*48 inch diameter Tee-section manhole may be used for pipes 27 inches and greater.

3. **Intake/Manhole Combination:** Intake/manhole combinations will be used when the size of the connecting pipes so indicate or when horizontal clearance is necessary behind the back of curb. The Engineer is encouraged to utilize intake/manhole combinations for storm sewers that are parallel to the street. This will prevent storm sewers from being installed under pavement; improving maintenance access without requiring pavement removal.

4. **Cleanouts:** Lamp holes or cleanout structures are required at the beginning of footing drains and subdrains in street right-of-way. Cleanouts may be allowed in place of a manhole at the end of lines that are less than 150 feet in length. Approval to use cleanouts is required.

5. **Access Spacing:** Storm sewer structures (manholes, intakes, combination intakes, or cleanouts) in street right-of-way must be located in areas that allow direct access by maintenance vehicles.

Areas outside the street right-of-way will be subject to the approval of the Jurisdictional Engineer.

a. **Manhole Spacing:** Manholes are to be spaced at intervals not exceeding 400 feet or at intervals not exceeding 500 feet when adequate cleaning equipment is available.

b. **Intake Spacing:** Locate street intakes upgrade from intersections, sidewalk ramps, and outside of intersection radii. At least one intake is to be installed at the low point of the street grade.
   1) **First Intake:** An intake should be located no further than 500 feet from the street high point.
   2) **Remaining Intakes:** To be spaced at a distance no greater than 400 feet, regardless of gutter flow capacity, in order to meet maintenance needs.

6. **Invert Drop:** When there is a change in pipe size at a structure, the invert of the smaller sewer must be raised to maintain the same energy gradient. An approximate method of doing this is to place the 0.8 depth point of both sewers at the same elevation. When there is a change in alignment between storm sewer of 45 degrees or greater, the suggested minimum manhole drop is 0.10 foot.

H. Manhole and Intake Standards

1. Manhole Standards to be Utilized:

Figure No.[1]	**Description**	**Use**	
		Main Pipe Size	***Depth Restrictions***
6010.401	Circular Storm Sewer Manhole	12” min. See table on Figure 6010.401 for max. pipe size	N/A
6010.402	Rectangular Storm Sewer Manhole	12” to 54”	8’ max.
6010.403	Deep Well Rectangular Storm Sewer Manhole	12” to 72”	12’ max.
6010.404	Rectangular Base/Circular Top Storm Sewer Manhole	12” to 96”	12’ min. to 22’ max.
6010.405	Tee-section Storm Sewer Manhole	12” or greater	N/A

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.

2. Manhole Castings to be Utilized:

Figure No.[1]	**Casting Type**	**Number of Pieces**	**Ring/ Cover**	**Bolted Frame**	**Bolted Cover (Floodable)**	**Gasket**
6010.602	E	2	Fixed[2]	Yes	No	No
6010.602	F	3	Adjustable[3]	No	No	No

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.
[2] Typically used with non-paved or flexible surfaces, including HMA, seal coat, gravel, and brick.
[3] Typically used with PCC surfaces, including castings in concrete boxouts.

3. Intake Standards to be Utilized:

Intake Type[1]	Intake Casting[1]	Standard	Conditions
Curb-Grate 6010.501	6010.603 Type Q	Single, poured 6" walls	Intake depth ≤ 7' Pipe size: 18" max. on 2' side, 30" max. on 3' side
Curb-Grate 6010.502	6010.603 Type Q	Single, precast walls	Intake depth > 7' Pipe size: 24" max. for 48" diameter
Curb-Grate (Combination) 6010.503/6010.504	6010.603 Type Q	Single, poured 6" walls	Intake depth ≤ 6' 6" Pipe size: 30" max. on 3' side, 36" max. on 6' side
Curb-Grate 6010.505	6010.603 Type Q	Double, poured 6" walls	Intake depth ≤ 7' Pipe size: 18" max. on 2' side, 66" max. on 6' 8" side
Curb-Grate (Combination) 6010.506	6010.603 Type Q	Double, poured 6" walls	Intake depth ≤ 6' 6" Pipe size: 30" max. on 3' side, 36" max. on 6' side, 48" max. on 6' 8" side
Curb Only 6010.507	N/A	Single open-throat, poured 6" walls	Intake depth ≤ 10' Pipe size: 30" max. on 3' side, 36" max. on 4' side
Curb Only 6010.508	N/A	Single open-throat, poured 6" walls	Intake depth ≤ 16' Pipe size: 36" max.
Curb Only 6010.509	N/A	Double open-throat, poured 6" reinforced walls	Intake depth ≤ 10' Pipe size: 30" max. on 3' side, 66" max. on 8' side
Curb Only 6010.510	N/A	Double open-throat, poured 6" reinforced walls	Intake depth < 10' Pipe size: 36" max. on 4' side, 66" max. on 8' side
Driveway or Alley Grate Intake 6010.511	6010.604 Type 6	Single (Surface Intake), poured 6" walls	Intake depth ≤ 7' Pipe size: 18" max. on 2' side, 30" max. on 3' side
Area Intake 6010.512	6010.604 Type 3, 4, or 5	Precast, Area Intake	Intake depth > 7' Pipe size varies on structure size
Ditch Intake 6010.513	6010.602 Type G	Area Intake (side open intake), poured 6" walls	Intake depth ≤ 7' Pipe size varies on structure size

[1] The figure numbers listed in this table (e.g. 6010.501) refer to figures from the SUDAS Specifications.

I. References

Comport, Thornton, & Cox. *Hydraulic Efficiency of Grate and Curb Inlets for Urban Storm Drainage*. Colorado State University. 2009.

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22. Third Ed. 2009.

General Information for Storm Sewer Design

A. Introduction

Storm sewer facilities collect stormwater runoff and convey it away from structures and through the roadway right-of-way in a manner that adequately drains sites and roadways and minimizes the potential for flooding and erosion to properties. Storm sewer facilities consist of curbs, gutter, intakes, manholes, and storm sewers. The placement and hydraulic capacities of storm sewer facilities should be designed to take into consideration damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.

B. Location of Storm Sewers

1. **Storm Sewers in Street Right-of-way:**

   a. Storm sewers parallel to the street and in the right-of-way should be placed behind the back of curbs, as close as practical, to fit specific manhole or intake connections.

   b. Storm sewers perpendicular to the street are to connect at each end by intakes or manholes.

   c. Storm sewers in the street right-of-way should be concrete pipe to prevent utility cuts through the pipe. This includes storm sewer service stubs equal to or greater than 12 inches in diameter, extended 10 feet outside of the right-of-way.

   d. If a type of flexible pipe is approved for use by the Engineer, it is important to take steps to protect the integrity of the trench backfill since the pipe depends on the backfill envelope for its strength. The pipe can be damaged if the backfill is disturbed.

2. **Public Storm Sewers Outside of Street Right-of-way but within Public Easement:** Storm sewers outside of the street right-of-way will be placed in a public storm sewer easement. Public storm sewer easements should have a minimum width of 20 feet or two times the depth of the sewer, whichever is greater. Additional width may be required by the Engineer to ensure proper access for maintenance purposes. When determining the width of the easement, consideration needs to be given to placement of excavated materials for the repair of the pipe.

   a. Storm sewer outlets should be concrete pipe.

   b. Upon the approval of the Engineer, flexible pipe and CMP may be used outside of the street right-of-way where the granular backfill is not likely to be disturbed by other utilities or other construction in the area.

   c. Storm sewer along a side property line should run the length of the property line and outlet past the rear property line to a receiving drainageway.

C. Pipe Materials

1. **Storm Sewer Pipes:** The approved storm sewer pipe materials are included in SUDAS Specifications Section 4020.

2. **Culverts:** The approved culvert materials are included in SUDAS Specifications Section 4030.

3. **Subdrains and Footing Drain Collectors:** The approved subdrain and footing drain collector materials are listed in SUDAS Specifications Section 4040.

D. Physical Requirements

1. **Minimum Cover over Storm Sewer Pipes:** The recommended minimum cover over storm sewer pipes should be 1 foot or as specified by the type of pipe as described in Chapter 9 - Utilities, whichever is greater. Where the clearance is less than 1 foot below the pavement, the Project Engineer will provide a design method to maintain the integrity of the pipe and pavement. For storm sewer pipe outside of the pavement, the minimum cover should be 1 foot or as specified by the type of pipe (described in Chapter 9 - Utilities), whichever is greater.

2. **Minimum Flow Line Depth for Footing Drain Sewers:** 3 feet 6 inches.

3. **Minimum Pipe Size:**

   a. **Storm Sewers:** 15 inches in diameter.

   b. **Subdrains:** 6 inches in diameter.

   c. **Footing Drain Collector Sewers in Public Right-of-way:** 8 inches in diameter.

   d. **Building Storm Sewer Stubs:** 4 inches in diameter

4. **Velocity within Storm Sewer Pipe:**

   a. Minimum flow (1/2 full pipe) = 3 fps cleaning velocity

   b. Maximum flow (1/2 full pipe) = 15 fps

5. **Velocity at Outlet of Pipe:** Energy dissipation is required when discharge velocities exceed those allowed for downstream channel. (See Tables 2F-2.03 and 2F-2.04).

   a. With flared end section, maximum of 5 fps.

   b. Maximum with flared end section, footing, and rip rap = 10 fps

   c. Maximum with energy dissipation device = 15 fps

6. **Partially Full Pipe Flow:** For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Table 2D-2.01 in Section 2D-2). The data presented assumes that the friction coefficient, Manning's "n" value, does not vary throughout the depth.

7. **Minimum Storm Sewer and Footing Drain Grades:**

   a. **Storm Sewer Mains:** Minimum grade is set by the required minimum velocity for storm sewers and footing drain sewers - 3 fps for design storm.

   b. **Cross Runs:** Minimum grade of 1%. Desired minimum velocity of 3 fps for design storm.

   c. **Building Storm Sewer Stubs:** Minimum grade of 1%.

   d. **Subdrains:** Minimum grade of 0.5%.

8. **Intakes:** See Section 2C-3.

9. **Manholes:** See Section 2C-3.

E. Horizontal Alignment

Sewer will be laid with a straight alignment between structures with the following exception: where street layouts are such that straight alignments are difficult to maintain without an increased number of structures, and where the storm sewers are 54 inches in diameter or greater, the sewers may be curved. The curvature will be factory fabricated pipe bends and should be concentric with the curvature of the street. The radius of curvature must not be less than 200 feet. The pipe manufacturer's recommended maximum deflection angle may not be exceeded.

F. Separation of Water Mains from Sewer Mains

The following comply with the Iowa Department of Natural Resources separation requirements.

1. **Horizontal Separation of Gravity Sewers from Water Mains:** Separate gravity storm sewer mains from water mains by a horizontal distance of at least 10 feet unless:
   - The top of a sewer main is at least 18 inches below the bottom of the water main, and
   - The sewer is placed in a separate trench or in the same trench on a bench of undisturbed earth at a minimum horizontal separation of 3 feet from the water main.

   When it is impossible to obtain the required horizontal clearance of 3 feet and a vertical clearance of 18 inches between sewers and water mains, the sewers must be constructed of water main materials meeting the requirements of SUDAS Specifications Section 5010, 2.01. However, provide a linear separation of at least 2 feet.

2. **Separation of Sewer Force Mains from Water Mains:** Separate storm sewer force mains and water mains by a horizontal distance of at least 10 feet unless:
   - The force main is constructed of water main materials meeting a minimum pressure rating of 150 psi and the requirements of SUDAS Specifications Section 5010, 2.01, and
   - The sewer force main is laid at least 4 linear feet from the water main.

3. **Separation of Sewer and Water Main Crossovers:** Vertical separation of storm sewers crossing under any water main should be at least 18 inches when measured from the top of the sewer to the bottom of the water main. If physical conditions prohibit the separation, the sewer may be placed not closer than 6 inches below a water main or 18 inches above a water main. Maintain the maximum feasible separation distance in all cases. The sewer and water pipes must be adequately supported and have watertight joints. Use a low permeability soil for backfill material within 10 feet of the point of crossing.

Where the storm sewer crosses over or less than 18 inches below a water main, locate one full length of sewer pipe of water main material or reinforced concrete pipe (RCP) with flexible O-ring gasket joints so both joints are as far as possible from the water main.

Storm Sewer Sizing

A. Introduction

The purpose of this section is to outline the basic hydraulic principles in order to determine the storm sewer size. The elements covered include basic flow formulas (Bernoulli Equation and Manning Equation), hydraulic losses, and hydraulic design of storm sewers. Information in this section was derived from FHWA's HEC-22 except where noted.

B. Definitions

Energy Grade Line: The energy grade line represents the total energy along a channel or conduit carrying water. For a fluid flowing without any losses due to friction (major losses) or components (minor losses) the energy grade line would be at a constant level. In practice, the energy grade line decreases along the flow due to these losses.

Hydraulic Grade Line: The hydraulic grade line equals the total head available to the fluid, minus the velocity head. Under open channel flow, the hydraulic grade line is at the water surface. Under pressure flow, the hydraulic grade line represents the level to which water would rise in piezometric pipes (or in manholes and intakes).

Pressure Head: Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Velocity Head: Velocity head is a quantity proportional to the kinetic energy flowing water expressed as a height or head of water.

C. Hydraulics of Storm Sewers

1. **Flow Assumptions:** The design procedures presented here assume that flow within each storm sewer segment is steady and uniform. This means that the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm sewers typically have a uniform pipe size within a segment, the average velocity throughout each segment is considered to be constant.

   In actual storm sewer systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based upon computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

2. **Open Channel vs. Pressure Flow:** Two design philosophies exist for sizing storm sewers under the steady uniform flow assumption. The first is referred to as open channel or gravity flow design. To maintain open channel flow, the segment must be sized so that the water surface within the conduit remains open to atmospheric pressure. For open channel flow, flow energy is derived from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). If the water surface throughout the conduit is to be maintained at atmospheric pressure, the flow depth must be less than the height of the conduit.

Pressure flow design requires that the flow in the conduit be at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within the conduit. In pressure flow, flow energy is again derived from the flow velocity, depth, and elevation. The significant difference here is that the pressure head will be above the top of the conduit, and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the hydraulic grade line.

The question of whether open channel or pressure flow should control design has been debated. For a given flow rate, a design based on open channel flow requires a larger storm sewer than those sized based on pressure flow. While it may be more expensive to construct storm drainage systems designed for open channel flow, this design procedure provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. This factor of safety is often desirable since the methods of runoff estimation are not exact, and once placed, storm drains are difficult and expensive to replace.

Under ordinary conditions, it is recommended that storm drains be sized based on a gravity flow criteria at flow full or near full. Designing for full flow is a conservative assumption since the peak flow capacity actually occurs at 93% of the full flow depth. When allowed by the Jurisdiction, pressure flow may be used. When pressure flow is allowed, special emphasis should be placed on the proper design of the joints so that they are able to withstand the pressure flow on a regular basis.

3. **Hydraulic Capacity:** A storm sewer's size, shape, slope, and friction resistance control its hydraulic capacity. These properties are all accounted for with the Manning Equation given as:

$$V = \frac{Q}{A} = \frac{1.486}{n} r^{2/3} s^{1/2}$$ **Equation 2D-2.01**

where:

V = Average velocity, ft/s
Q = Discharge, cfs
A = Cross-sectional area of flow, ft^2
n = Manning's roughness coefficient
r = hydraulic radius, ft
= A/p (note: for circular pipes flowing full, r=D/4)
p = wetted perimeter, ft
s = slope of hydraulic grade line (pipe/channel slope), ft/ft

Table 2D-2.01: Manning Coefficients for Common Storm Sewer Materials

Type of Pipe	Manning’s n
Concrete pipe	0.013
PVC pipe (smooth wall)	0.010
Polyethylene (smooth interior)	0.011
HDPE (smooth or corrugated)	0.020
CMP (2-2/3" x 1/2" corrugations)	0.024
CMP (3" x 1" corrugations)	0.027
CMP (5”x1” corrugations)	0.025
Structural Plate	0.032

Note: for additional manning coefficients, see the pipe manufacturer’s information.

D. Conservation of Energy

1. **Bernoulli Equation:** The law of conservation of energy, as expressed by the Bernoulli Equation, is the basic principle most often used in hydraulics. This equation may be applied to any conduit with a constant discharge. Friction flow formulas such as the Manning’s Equation have been developed to express the rate of energy dissipation as it applies to the Bernoulli Equation. The theorem states that the energy head at any cross-section must equal that in any other downstream section plus the intervening losses.

   Bernoulli’s equation, where the total energy at Section 1 is equal to the energy at Section 2 plus the intervening head loss, is summarized in two versions below:

For open (non-pressure) conduit flow:

$$\frac{V_1^2}{2g} + Y_1 + Z_1 = \frac{V_2^2}{2g} + Y_2 + Z_2 + h_f$$ **Equation 2D-2.02**

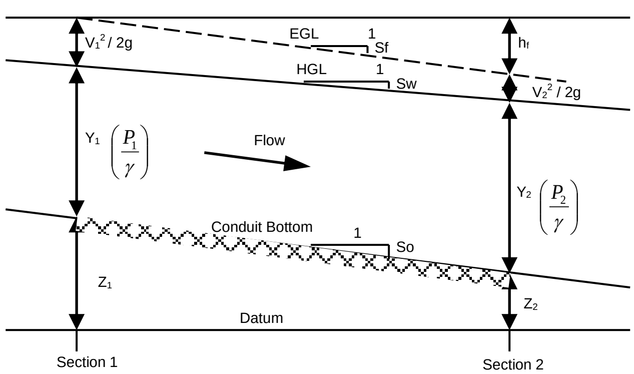
For pressure conduit flow

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_f$$ **Equation 2D-2.03**

where:

EGL = Energy grade line
HGL = Hydraulic grade line
Y = Water depth, ft
$V^2/_{2g}$ = Energy head, ft
V =Average velocity, fps
S_f = Slope of EGL
S_w = Slope of HGL
g = acceleration of gravity (32.2 fps)
$P/_{\gamma}$ = Pressure head, ft
P = Pressure at given location (lb/ft^2)
γ = Specific weight of water (62.2 lb/ft^3)
Z = Elevation relative to some datum
S_0 = Slope of bottom, ft/ft
h_f = Head loss, ft

Figure 2D-2.01: Terms Used in the Energy Equation



Source: FHWA, HEC-22

E. Hydraulic Losses

Storm sewers should be designed to convey the minor storm runoff peaks without surcharging the sewer. In situations where surcharging is a concern, the hydraulic grade line may be calculated by accounting for pipe friction losses and pipe form losses. Total hydraulic losses will include friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses are presented herein.

1. **Pipe Friction Losses:** The major head loss in a storm drainage system is due to pipe friction. Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined:

$$S_f = 0.453 \frac{Q^2 n^2}{A^2 R^{4/3}}$$ **Equation 2D-2.04**

The friction head lost through a segment is simply the hydraulic gradient multiplied by the length of the run:

$$H_f = S_f L$$ **Equation 2D-2.05**

where:

H_f = Friction head loss, ft
S_f = Friction slope, ft/ft
L = Length of outflow pipe, ft

2. **Exit Losses:** The exit loss from a storm sewer outlet is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$H_o = 1.0 \left[\left(\frac{V_o^2}{2g} \right) - \left(\frac{V_d^2}{2g} \right) \right]$$ **Equation 2D-2.06**

where:

V_o = Average outlet velocity, ft/s
V_d = Channel velocity downstream of outlet in direction of the pipe flow, ft/s
g = Acceleration due to gravity, 32.2 ft/s

Note that when $V_d = 0$, as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

3. **Bend Losses:** The bend loss coefficient (H_b) for storm sewer design (for bends in the pipe run, not in a structure) can be estimated using the following formula:

$$H_b = K_b \frac{V^2}{2g}$$ **Equation 2D-2.07**

where:

K_b = Bend loss coefficient (refer to Table 2D-2.02)

Table 2D-2.02: Bend Loss Coefficients

Bend Radius / Pipe Dia. (R/d)	**Degree of Bend**		
	22.5°	***45°***	***90°***
1	0.12	0.17	0.23
2	0.07	0.10	0.13
4 or larger	0.04	0.06	0.08

Source: Brater, King, et al.

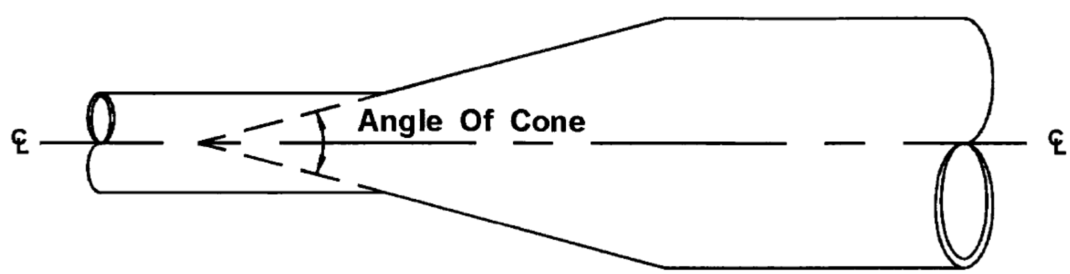
4. **Transition Losses:** A transition is a location where a conduit or channel changes size. Transitions include expansions, contractions, or both. In small storm sewers, transitions should occur within manhole or intake structures. However, in larger storm sewers, or when a specific need arises, expansions may occur within pipe runs. Contractions must always occur within a structure and never within the pipeline, regardless of pipe size.

   Energy losses due to expansions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends:

   $$H_e = K_e\left[\left(\frac{V_2^2}{2g}\right)-\left(\frac{V_1^2}{2g}\right)\right]$$ **Equation 2D-2.08**

   The head loss coefficient for a pipe expansion, K_e, is dependent upon the rate of change in diameter (angle of cone) as shown in Table 2D-2.03.

Figure 2D-2.02: Angle of Cone for Pipe Diameter Changes



Source: FHWA, HEC-22

Table 2D-2.03: Typical Values of K_e for Gradual Enlargement of Pipes in Non-pressure Flow

D_2 / D_1	**Angle of Cone**						
	10°	***20°***	***45°***	***60°***	***90°***	***120°***	***180°***
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

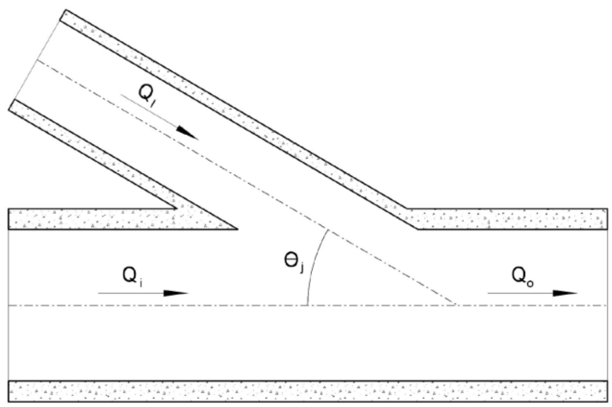
5. **Junction Losses:** A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of a manhole or other structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j = \left\{\frac{[(Q_oV_o) - (Q_iV_i) - (Q_lV_l \cos\theta_j)]}{[0.5g(A_o + A_i)]}\right\} + \frac{V_i^2}{2g} - \frac{V_o^2}{2g}$$ **Equation 2D-2.09**

where:

H_j	= Junction loss, ft
Q_o, Q_i, Q_l	= Outlet, inlet, and lateral flows respectively, ft^3/s
V_o, V_i, V_l	= Outlet, inlet, and lateral velocities, respectively, ft^3/s
A_o, A_i	= Outlet and inlet cross-sectional area, ft^2
θ	= Angle between the inflow trunk pipe and inflow lateral pipe, degrees

Figure 2D-2.03: Interior Angle Definition for Pipe Junctions



6. **Structure Losses:** A complex situation exists where a manhole or intake exists at the junction between inflow and outflow pipes. The following method provides approximate results and estimates losses across a structure by multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 2D-2.10. Table 2D-2.04 tabulates typical coefficients (K_{ah}) applicable for use in this method. Refer to HEC-22 for a detailed explanation of analyzing structure losses.

$$H_{ah} = K_{ah}\left(\frac{V_{0i}^2}{2g}\right)$$ **Equation 2D-2.10**

This approximate method estimates the necessary elevation drop across a structure required to offset energy losses through the structure. This drop is then used to establish the appropriate pipe invert elevations.

Table 2D-2.04: Head Loss Coefficients through Structures

Pipe Angled Through	K_S
90°	1.00
60°	0.85
45°	0.75
22.5°	0.45
Straight Run	0.15

7. **Structure Drop:** Where pipe size increases in a structure, the invert of the smaller sewer must be raised to maintain the same energy gradient. An approximate method of doing this is to place the 0.8 depth point of both sewers at the same elevation. When there is a change in alignment between storm sewers of 45 degrees or greater, the suggested minimum manhole drop is 0.10 foot.

F. References

Brater, King, et al. *Handbook of Hydraulics*. Seventh Ed. 1996.

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22. Third Ed. 2009.

Groundwater Barriers and Outlets

A. Introduction

When there exists a possibility that groundwater may be diverted and follow the path of the new sewer, groundwater barriers should be constructed in adequate numbers to prevent groundwater migration down sewer trenches.

Subsurface barriers are designed to prevent or control groundwater flow into, through, or from a certain location. Barriers keep fresh groundwater from coming into contact with a contaminated aquifer zone or ground water from existing areas of contamination from moving into areas of clean groundwater. Usually it is necessary to incorporate other technologies, such as pump-and-treat systems, with groundwater barriers.

B. Groundwater Barriers

The types of barriers commonly used include:
- Slurry trench walls
- Grout curtains
- Vibrating beam walls
- Bottom sealing
- Block displacement
- Sheet piles
- Sheet curtains

1. **Slurry Trench Walls:** Slurry trench walls are placed either upgradient from a waste site to prevent flow of groundwater into the site, downgradient to prevent off-site flow of contaminated water, or around a source to contain the contaminated groundwater. A slurry wall may extend through the water-bearing zone of concern, or it may extend only several feet below the water table to act as a barrier to floating contaminants. In the former case, the foundation should lie on, or preferably in, an underlying unit of low permeability so that contaminants do not flow under the wall. A slurry wall is constructed by excavating a trench at the proper location and to the desired depth, while keeping the trench filled with a clay slurry composed of a 5% to 7% by weight suspension of bentonite in water. The slurry maintains the vertical stability of the trench walls and forms a low permeability filter cake on the walls of the trench. As the slurry trench is excavated, it is simultaneously filled with a material that forms the final wall. The three major types of slurry backfill mixtures are soil bentonite, cement bentonite, and concrete. Slurry walls, under proper conditions, can be constructed to depths of about 100 feet.

   Slurry trench walls are reported to have a long service life and short construction time, cause minimal environmental impact during construction, and be a cost-effective method for enclosing large areas under certain conditions. A concern regarding the use of a slurry wall where contaminated materials are in direct contact with the wall is the long-term integrity of the wall. In such cases, the condition of the wall needs to be verified over time by groundwater monitoring.

2. **Grouting Curtains:** Grouting is the process of pressure-injecting stabilizing materials into the subsurface to fill, and thereby seal, voids, cracks, fissures, or other openings. Grout curtains are underground physical barriers formed by injecting grout through tubes. The amount of grout needed is a function of the available void space, the density of the grout, and the pressures used in setting the grout. Two or more rows of grout are normally required to provide a good seal. The grout used may be either particulate (i.e., portland cement) or chemical (i.e., sodium silicate) depending on the soil type and the contaminant present. Grouting creates an effective barrier to groundwater movement, although the degree of completeness of the grout curtain is difficult to ascertain. Incomplete penetration of the grout into the voids of the earth material permits leakage through the curtain.

3. **Vibrating Beam Walls:** A variation of the grout curtain is the vibrating beam technique for placing thin (approximately 4 inches) curtains or walls. Although this type of barrier is sometimes called a slurry wall, it is more closely related to a grout curtain since the slurry is injected through a pipe in a manner similar to grouting. A suspended I-beam connected to a vibrating driver-extractor is vibrated through the ground to the desired depth. As the beam is raised at a controlled rate, slurry is injected through a set of nozzles at the base of the beam, filling the void left by the beam's withdrawal. The vibrating beam technique is most efficient in loose, unconsolidated deposits, such as sand and gravel.

4. **Bottom Sealing:** Another method that uses grouting is bottom sealing, where grout is injected through drill holes to form a horizontal or curved barrier below the site to prevent downward migration of contaminants.

5. **Block Displacement:** Block displacement is a relatively new plume management method, in which a slurry is injected so that it forms a subsurface barrier around and below a specific mass or "block" of material. Continued pressure injection of the slurry produces an uplift force on the bottom of the block, resulting in a vertical displacement proportional to the slurry volume pumped.

6. **Sheet Piles:** Sheet pile cutoff walls have been used for many years for excavation bracing and dewatering. Where conditions are favorable, depths of 100 feet or more can be achieved. Sheet piling cutoff walls can be made of wood, reinforced concrete, or steel, with steel being the most effective material for constructing a groundwater barrier. The construction of a sheet pile cutoff wall involves driving interlocking sheet piles down through unconsolidated materials to a unit of low permeability. Individual sheet piles are connected along the edges with various types of interlocking joints. Unfortunately, sheet piling is seldom water-tight and individual plates can move laterally several to several tens of feet while being driven. Acidic or alkaline solutions, as well as some organic compounds, can reduce the expected life of the system.

7. **Sheet Curtains:** Membrane and synthetic sheet curtains can be used in applications similar to grout curtains and sheet piling. With this method, the membrane is placed in a trench surrounding or upgradient of the plume, thereby enclosing the contaminated source or diverting groundwater flow around it. Placing a membrane liner in a slurry trench application also has been tried on a limited basis. Attaching the membrane to an underlying confining layer and forming perfect seals between the sheets is difficult but necessary in order for membranes and other synthetic sheet curtains to be effective.

Source: The Pan American Center for Sanitary Engineering and Environmental Sciences, CEPIS.

C. Outlets

1. Where a storm sewer discharges into a natural channel or irrigation ditch, an outlet structure should be provided that will blend the storm sewer discharge into the natural channel flow in such a way as to prevent erosion of the bed or banks of the channel. As a minimum, all storm sewer pipes that outlet to drainageways will require flared end sections with apron guard for pipe diameters 18 inches or larger. Storm sewers 30 inches in diameter or greater require a footing at the outlet. Footings may be required for pipe diameters less than 30 inches.

2. In an instance where the discharge velocity is high (higher than those outlined in Section 2F-2, Tables 2F-2.03 and 2F-2.04) or supercritical, prevention of erosion of the natural channel bed or banks in the vicinity of the outlet requires an energy dissipating structure, such as:
   - Rip rap
   - Concrete slab
   - Gabions
   - Headwalls and wing wall with stilling basins
   - Flow transition mats

3. Outlets should drain at a receiving drainageway or connect to an existing storm sewer. Outlets should not drain across sidewalks or directly to streets. Outlets should not be located on slopes without adequate erosion protection and means of conveyance between the outlet and receiving drainageway or storm sewer. Erosion protection on a slope that does not extend beyond the outlet is often inadequate, as runoff velocity will increase down grade of the outlet.

General Information for Culvert Design

A. Introduction

A culvert is a conduit under an embankment that transports stormwater from one side of the embankment to the other through hydraulic inlet, outlet, or barrel control. The primary purpose of a culvert is to convey surface water. However, when properly designed, it may also be used to restrict flow for upstream detention and reduce downstream storm runoff peaks. Primary considerations for the final selection of any drainage structure should be based upon appropriate hydraulic principles, economy, and minimal effects on adjacent property by the resultant headwater depth and outlet velocity. The allowable headwater elevation is the maximum elevation that can be reached before damage could be caused to adjacent property or compromise the right-of-way. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The control of flow in a culvert can shift dramatically and unpredictably between inlet control, barrel control, and outlet control, causing relatively sudden rises in headwater. A critical aspect of culvert design is to determine stable and predictable performance for all expected flow levels. When the type of flow is known, the well-known equations for orifice, weir, or pipe flow and backwater profiles can be applied to determine the relationships between head and discharge (Blaisdell, 1966). Modern culvert nomographs, computer programs, and instructions are based on sound theory and extensive laboratory and field studies.

The 100 year flood is checked to determine if streets will provide access or be inundated. See Section 2A-3 that addresses access requirements for specific storms. Performance curves should be made available for all culverts for evaluating the hydraulic capacity of a culvert for various headwaters. These will display the consequence of high-flow rates at the site and any possible hazards. Sometimes a small increase in flow rate can affect a culvert design. If only the design peak discharge is used in the design, the designer cannot assess what effects any increases in the estimated design discharge will have on the culvert design. For culverts with significant headwater storage, the site should be treated as detention design, and flow should be routed.

B. Definitions

Backwater: Constriction of flow causes a rise in the normal water surface elevation upstream of the constriction. The magnitude of the rise, in feet, is called backwater.

Barrel Control: Barrel control for culvert hydraulics exists when the rise of headwater at the culvert inlet is greater than the rise from inlet or outlet control. This rise in headwater from barrel control can be a combination of barrel roughness, length, and restriction. Barrel control is rarely the control of headwater. Since the head loss due to roughness in the barrel is normally not as great as inlet head loss, the effect of barrel roughness is included as part of outlet control.

Critical Depth: Critical depth can best be illustrated as the depth of water at the culvert outlet under outlet control at which water flows are not influenced by backwater forces. Critical depth is the depth at which specific energy of a given flow rate is at a minimum. For a given discharge and cross-section geometry, there is only one critical depth.

Energy Grade Line: The energy grade line represents the total energy at any point along the culvert barrel.

Free Outlets: Free outlets are outlets with a tailwater equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Headwater: The vertical distance from the culvert invert (flow line) at the culvert entrance to the water surface elevation of the upstream channel.

Hydraulic Grade Line: The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of a culvert barrel. In a full flow, the energy grade line and the hydraulic grade line are parallel lines separated by the velocity head, except at the inlet and the outlet.

Improved Inlets: Flared, improved, or tapered inlets indicate a special entrance condition that decreases the amount of energy needed to pass the flow through the inlet and, thus increases the capacity of culverts at the inlet.

Inlet Control: With inlet control, the cross-sectional area of the culvert barrel, inlet geometry, and the amount of headwater or ponding at the entrance are the controlling design factors.

Invert: Invert refers to the inside bottom of the culvert.

Normal Flow: Normal flow occurs in the channel reach when the discharge, velocity, and depth of flow do not change throughout the reach. The water surface profile and channel bottom slope will be parallel. This type of flow will be approximated in a culvert operating on a steep slope, provided the culvert is sufficiently long.

Outlet Control: Outlet control involves the additional considerations over inlet control of the elevation of the tailwater, slope, roughness, and length of the culvert.

Steep and Mild Slope: A steep-slope culvert operation is where the computed critical depth is greater than the computed uniform depth. A mild-slope culvert operation is where critical depth is less than uniform.

Submerged Inlets: Submerged inlets are those inlets having a headwater greater than 1.2 times the diameter of the culvert or barrel height.

Submerged Outlets: Partially submerged outlets are outlets with tailwater that is higher than critical depth and lower than the height of the culvert. Submerged outlets are outlets having tailwater elevation higher that the soffit (crown) of the culvert.

Tailwater: The water depth from the culvert invert at the outlet to the water surface in the outlet swale or channel.

Uniform Flow: Uniform flow is flow in a prismatic channel of constant cross-section having a constant discharge, velocity, and depth of flow throughout the reach. This type of flow will exist in a culvert operating on a steep slope, provided the culvert is sufficiently long.

C. Site Considerations

Site considerations include the generalized shape of the embankment, bottom elevations and cross-sections along the streambed, the approximate length of the culvert, and the allowable headwater elevation. In determining the allowable headwater elevation, roadway elevations and the elevation of upstream property should be considered. The consequences of exceeding the allowable headwater need to be kept in mind throughout the design process.

D. Culvert Design Items

The following should be considered for all culvert designs where applicable:

1. Engineering aspects
   a. flood frequency
   b. velocity limitations
   c. buoyancy protection

2. Site criteria
   a. length and slope
   b. debris and siltation control
   c. culvert barrel bends
   d. ice buildup

3. Design limitations
   a. headwater limitations
   b. tailwater conditions
   c. storage – temporary or permanent

4. Design options
   a. culvert inlets
   b. inlets with headwalls
   c. wingwalls and aprons
   d. improved inlets
   e. material selection
   f. culvert skews
   g. culvert sizes and shapes
   h. twin pipe separations (vertical and horizontal)
   i. culvert clearances

5. Related designs
   a. weep holes
   b. outlet protection
   c. erosion and sediment control
   d. environmental considerations

The designer must incorporate experience and judgment to determine which of the above items listed need to be evaluated and how to design the final culvert installation.

E. Design Considerations

1. **Flood Frequencies:** See Sections 2A-1 and 2A-3 for flood design frequencies.

2. **Velocity Limitations:**

    a. **Minimum Cleaning Velocity:** 3.0 fps

    b. **Maximum Velocity:** Should be consistent with outlet conditions of a stream or waterway. The need for channel stabilization at a culvert outlet is based on exceeding the natural stability of the channel.

3. **Buoyancy Protection:** Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts greater than 24 inches in diameter. Buoyancy is more serious with steepness of the culvert slope, depth of the potential headwater (debris blockage may increase headwater), flatness of the upstream fill slope, height of the fill, large culvert skews, or mitered ends.

4. **Length and Slope:** Because the length of the culvert will affect the capacity of culverts on outlet control, the length should be kept to a minimum, and yet meet future needs and clear zones. Existing facilities should not be extended without determining the decrease in capacity that will occur. In addition, the culvert length and slope should be chosen to approximate existing topography. To the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream. The culvert entrance should match the geometry of the embankment. Future street or highway improvements need to be considered when setting the length of the culvert, especially in growth areas where rural cross-sections may be converted to urban sections, or street widening is a probability with sidewalks, utility corridors, etc.

5. **Debris Control:** In designing debris control structures, it is recommended that the publication Hydraulic Engineering Circular No. 9 titled "Debris Control Structures" (FHWA, 2005) be consulted. Debris control should be considered in the following conditions:

    a. Where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris.

    b. For culverts located in steep regions.

    c. For culverts that are under high fills.

    d. Where cleaning access is limited. However, access must be available to clean the debris-control device.

6. **Siltation:** When streams or overland flow drain through culverts and carry silt, it is important to design the culvert such that the culvert barrel will not be clogged with silt and reduce its capacity.

    a. **Barrel Slope:** The barrel slope of culverts should not have long sections of subcritical flow. This minimizes the settling of silt in the barrel. The slopes should be designed so the minimum velocity through the barrel will be no less than 3 fps for a 2 year storm frequency.

b. **Horizontal Bends:** A straight culvert alignment is desirable to avoid clogging, increased construction costs, and reduced hydraulic efficiency. However, site conditions may dictate a change of alignment. Horizontal bends may be used to avoid obstacles or realign the flow. When considering a nonlinear culvert alignment, particular attention should be given to maintenance access and erosion, sedimentation, and debris control. Certain culvert installations may encounter sedimentation problems. The most common of these problems are multi-barrel installations. Culverts with more than one barrel may be necessary for wide shallow streams and for low fills. It is well-documented that one or more of the barrels will accumulate sediment, particularly the inner barrel in a curved stream alignment – especially during times of low flow. However, self-cleaning usually occurs during periods of high discharge. This design situation should be approached cautiously with an increased effort in the field investigation stage to obtain a thorough knowledge of stream characteristics and bed-bank materials.

c. **Multiple Pipe:** To help prevent siltation in low-flow conditions where multiple pipes are used, the inlet of all but one of the multiple pipes is placed higher than the other. The lower pipe can maintain cleaning velocities, and the higher pipes help provide flow capacity for major storms. The difference in elevation between the pipes is based on the depth of flow of the lower pipe for a 2 year storm frequency. The higher pipe is therefore at or above the 2 year frequency elevation in the lower pipe.

7. **Headwater Limitations:** The allowable headwater (HW) elevation is determined from elevation of land use upstream of the culvert and the proposed or existing top of the embankment. Headwater is the depth (D) of water above the culvert inlet invert. In general, the constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.

The allowable headwater design frequency conditions should allow for or consider the following upstream controls:

- Reasonable freeboard (see Section 2A-3 for maximum allowable headwater depth).
- Upstream property damage
- Elevations established to delineate floodplain zoning
- Low point in the road grade that is not at a culvert location
- Ditch elevation of the terrain that will permit flow to divert around culvert
- Follow recommended HW/D design criteria:
  - For drainage facilities with cross-sectional area equal to or less than 30 square feet, HW/D is equal to or less than 1.5
  - For drainage facilities with cross-section area greater than 30 square feet, HW/D is equal to or less than 1.2
- The headwater should be checked for the 100 year flood to ensure compliance with floodplain criteria.
- The maximum acceptable outlet velocity should be identified. The headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided where acceptable velocities are exceeded.

If there is insufficient headwater elevation available to convey the required discharge, it will be necessary to use a larger culvert, lower inlet invert, irregular cross-section such as pipe arches or multiple pipes, improved inlet if in inlet control, multiple barrels, or a combination of these measures. If the inlet is lowered, special consideration must be given to scour and sedimentation at the entrance.

8. **Tailwater Conditions:** The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharges. At times, there may be a need for calculating backwater curves to establish the tailwater conditions. If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined. Tailwater elevations can determine whether a culvert will operate with a free outfall or under submerged conditions. For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined.

   If an upstream culvert outlet is located near a downstream culvert inlet or other control, the headwater elevation of the downstream control may establish the design tailwater depth for the upstream culvert. If the culvert discharges to a lake, pond, or other major water body, the expected high-water elevation of the particular water body may establish the culvert tailwater.

9. **Storage - Temporary or Permanent:** If storage is being assumed upstream of the culvert, consideration should be given to the following.

   a. The total area of flooding.

   b. The average time that bankfull stage is exceeded for the design flood; up to 48 hours in rural areas or 6 hours in urban areas.

   c. Availability of the storage area for the life of the culvert through the purchase of right-of-way or easement.

10. **Weep Holes:** Weep holes are sometimes used to relieve uplift pressure. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent formation of piping channels. The filter material should be designed as underdrain filter so that it will not become clogged and so that piping cannot occur through the pervious material and the weep hole. Plastic woven filter cloth would be placed over the weep hole in order to keep the pervious material from being carried into the culvert. If weep holes are used to relieve uplift pressure, they should be designed in a manner similar to underdrain systems.

11. **Erosion Control at Inlet and Outlet:** Energy dissipation will be required for velocities higher than those outlined in Tables 2F-2.03 and 2F-2.04. Gabions or other erosion prevention or energy dissipation devices may be required.

12. **Erosion Control along Channel:** See Chapter 7 - Erosion and Sediment Control for specific information on channel/ditch lining. When pavement or rip rap for side slope inverts are not used, nets, meshes, or geo-grids placed along the toe of the backslope of a paved channel bottom help prevent erosion of the bank and undermining of paved channels.

13. **Environmental Considerations:** In addition to controlling erosion, siltation, and debris at the culvert site, care must be exercised in selecting the location of the culvert site. Environmental considerations are an important aspect of the culvert design. Using good hydraulic engineering, a site should be selected that will allow the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.

14. Horizontal Culvert Clearances:

a. Small culverts (30 inches in diameter or less) should use an end section or a sloped headwall.

b. Culverts greater than 30 inches in diameter should receive one of the following treatments:
   1) Extend to appropriate clear zone distance per AASHTO Roadside Design Guide
   2) When installing a grate to prevent entry, make sure to check the potential consequences of clogging and flooding.

15. Separation of Multi-pipe Culverts: In order to provide proper spacing between multi-pipe culverts, the following should be considered:

a. Without Aprons: If multi-pipe culverts are placed without aprons or footings, the distance between the centerline of each pipe should be 1 1/2 times the pipe diameter, but no less than 1 foot between the outside wall of each pipe. This separation allows room for compaction between the culverts. If a cutoff wall or barrier wall of low-permeability clay soil at least 2 feet thick is not available at the inlet and outlet to protect the pipe backfill, then consideration should be given to the use of flowable mortar as a means of pipe backfill.

b. With Curtain Walls: The distance between the centerline of each pipe culvert with curtain walls equals the diameter plus 2 feet (allows for proper reinforcement placement in the footing).

c. With Aprons: The separation between multi-pipe culverts with aprons is based on the distance need between aprons. This distance should be a minimum of 2 feet from the end of the apron for concrete and reinforcement placement to tie the aprons together. A preferable distance of 4 to 6 feet should be used when earth fill is used.

F. Pipe Material

1. RCP - Minimum strength Class III under all streets and entrance pavement and Class V under railroad tracks and pipes to be jacked.

2. Use of CMP and multi-plate gauge is at the discretion of the Jurisdictional Engineer.

G. Pipe Culvert Sizes

1. **Entrance Pipes:** Minimum 18 inches in diameter

2. **Street or Roadway Pipe**: Minimum 24 inches in diameter

H. Culvert Inlets

Selection of the type of inlet is an important part of the culvert design, particularly with inlet control. Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. All the methods described in this chapter directly or indirectly use inlet coefficients. See Table 2E-1.01.

1. **Inlets with Headwalls:** Headwalls may be used for a variety of reasons:
   - Increasing the efficiency of the inlet
   - Providing embankment stability
   - Providing embankment protection against erosion
   - Providing protection from buoyancy
   - Shortening the length of the required structure

   The relative efficiency of the inlet depends on the pipe material. Headwalls are usually required for all metal culverts and where buoyancy protection is necessary. Corrugated metal pipe in a headwall is essentially square-edged with an inlet coefficient of approximately 0.5. For tongue-and-groove or bell-and-spigot concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall.

2. **Wingwalls and Aprons:** Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable, or where the culvert is skewed to the normal channel flow. Little increase in hydraulic efficiency is realized with the use of normal wingwalls, regardless of the pipe material used and therefore, the use should be justified for other reasons. Wingwalls can be used to increase hydraulic efficiency if designed as a side-tapered inlet.

   If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

Table 2E-1.01: Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls:	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded [radius = 1/12 depth]	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7º or 45º bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projected from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
End-section[1] conforming to fill slope	0.5
Beveled edges, 33.7º or 45º bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls):	
Square-edged on three edges	0.5
Rounded on three edges to radius of 1/12 depth or beveled edges on three sides	0.2
Wingwalls at 30º to 75º to barrel:	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 depth or beveled top edge	0.2
Wingwalls at 10º or 25º to barrel:	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

[a] End-section conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall inlet and outlet controls. Some end-sections, incorporating a closed taper in their design, have superior hydraulic performance.

Source: From Federal Highway Administration, Hydraulic Design of Improved Inlets for Culverts, Hydraulic Engineering Circular No. 13, 1972.

I. Roadway or Street Overtopping

To complete the culvert design, roadway or street overtopping should be analyzed. See Section 2A-3 for allowable depth for major storms and cross-street flow allowable depths. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial-and-error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step 1: Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Inlet- and outlet-control headwaters should be calculated.

Step 2: Combine the inlet- and outlet-control performance curves to define a single performance curve for the culvert.

Step 3: When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and the equation below to calculate flow rates across the roadway.

$$Q = C_d L (HW)^{1.5}$$ **Equation 2E-1.01**

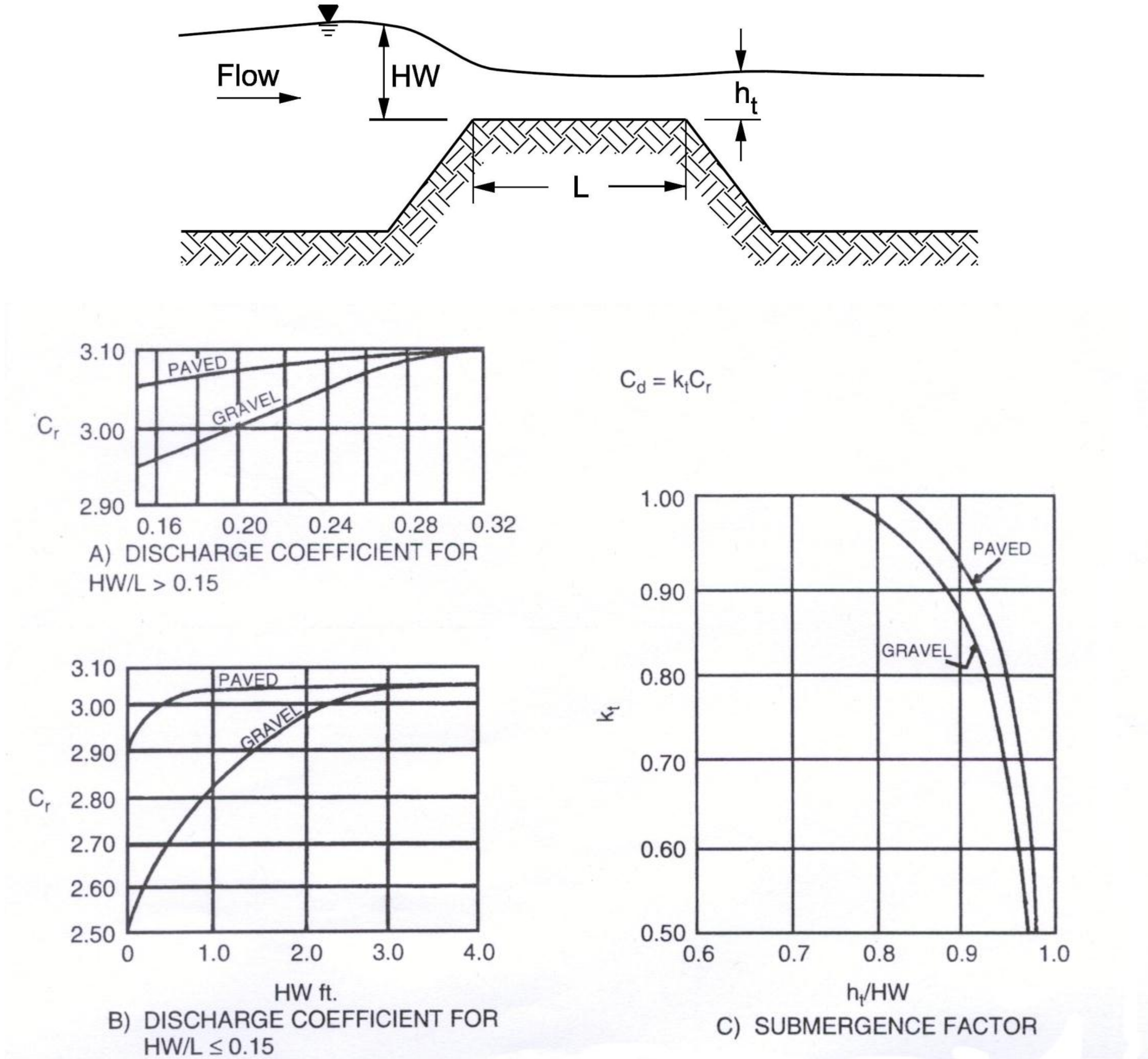
where:

Q = overtopping flow rate, cfs
C_d = overtopping discharge coefficient
L = length of roadway, ft
HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown, ft

Step 4: See Figure 2E-1.01 for guidance in determining a value for C_d.

Step 5: Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Figure 2E-1.01: Determination of Overtopping Discharge Coefficient



Source: Debo & Reese

J. Storage Routing

A significant storage capacity behind an embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert and its size may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the embankment. Routing procedures are outlined in HDS No. 5 (FHWA, 1985). In addition, the HEC-RAS program may be used to analyze backwater conditions upstream of the culvert.

Flood routing design procedures through a culvert are the same as for a reservoir or detention basin. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed.

K. References

Debo T., Reese A. *Municipal Stormwater Management.* Second Ed. 2003.

Federal Highway Administration. Hydraulic Engineering Circular No. 9. *Debris Control Structures*. 2005.

Federal Highway Administration. Hydraulic Engineering Circular No. 13. *Hydraulic Design of Improved Inlets for Culverts*. 1972.

The American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide.*

Culvert Hydraulics

A. Culvert Flow Controls and Equations

Figure 2E-2.01 depicts the energy grade line and the hydraulic grade line for full flow in a culvert barrel. The energy grade line represents the total energy at any point along the culvert barrel. Headwater is the depth from the inlet invert to the energy grade line. The hydraulic grade line is the depth to which water would rise in the vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel straight lines separated by the velocity head lines except in the vicinity of the inlet where the flow passes through a contraction.

The headwater and tailwater conditions as well as the entrance, friction, and exit losses are also shown in Figure 2E-2.01. When equating the total energy at sections 1 and 2 (see Figure 2E-2.01), upstream and downstream of the culvert barrel in the figure, the following relationship results:

$$\mathrm{HW_0} + \frac{\mathrm{V_1^2}}{2\mathrm{g}} = \mathrm{TW} + \frac{\mathrm{V_1^2}}{2\mathrm{g}} + \mathrm{H}$$ **Equation 2E-2.01**

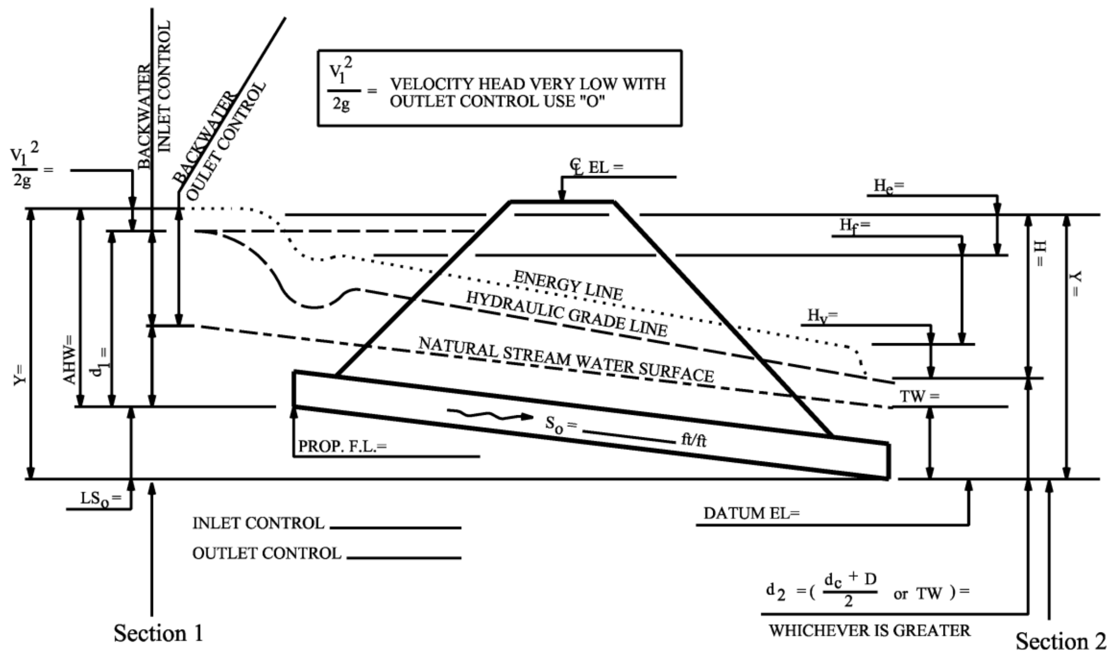
where:

H = sum of all losses = $H_e + H_f + H_v;\ H = \left[1 + K_e + \frac{29n^2L}{R^{1.33}}\right]\frac{V^2}{2g}$ **Equation 2E-2.02**

where:

V = the average velocity in the culvert barrel, ft/s
g = acceleration of gravity, ft/s (32.2)
K_e = inlet loss coefficient (see Section 2E-1, Table 2E-1.01)
R = hydraulic radius (cross sectional area of the fluid in the culvert divided by the wetted perimeter)
H_e = entrance head loss $= (K_e)\frac{V^2}{2g}$ **Equation 2E-2.03**
H_f = barrel friction head loss $= \left(\frac{29n^2L}{R^{1.33}}\right)\frac{V^2}{2g}$ **Equation 2E-2.04**
H_v = velocity head loss $= \frac{V^2}{2g}$ **Equation 2E-2.05**

Figure 2E-2.01: Full Flow Energy and Hydraulic Grade Line



Source: Adapted from *Hydraulic Design of Highway Culverts*, FHWA

B. Inlet and Outlet Control

The design procedures contained in this section are for the design of culverts for a constant discharge considering inlet and outlet control. Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert is operating on a mild slope. Inlet control usually occurs if the culvert is operating on a steep slope.

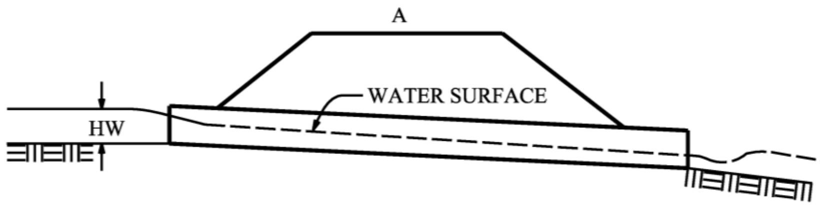
For inlet control, the entrance characteristics of the culvert are such that the entrance headlosses are predominant in determining the headwater of the culvert. The barrel will carry water through the culvert more efficiently than the water can enter the culvert. Proper culvert design and analysis requires checking for inlet and outlet control to determine which will govern particular culvert designs. For outlet control, the headlosses due to tailwater and barrel friction are predominant in controlling the headwater of the culvert. The entrance will allow the water to enter the culvert faster than the backwater effects of the tailwater, and barrel friction will allow it to flow through the culvert.

1. **Inlet Control:** Since the control is at the upstream end in inlet control, only the headwater and the inlet configuration affect the culvert performance. The headwater depth is measured from the invert of the inlet control section to the surface of the upstream pool. The inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area, but for tapered inlets, the face area is enlarged, and the control section is at the throat.

   Examples of inlet control:
   Figures 2E-2.01A through 2E-2.01D depict several different examples of inlet control flow. The type of flow depends on the submergence of the inlet and outlet ends of the culvert. In all of these examples, the control section is at the inlet end of the culvert. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

a. Figure 2E-2.01A depicts a condition where neither the inlet nor the outlet end of the culvert is submerged. The flow passes through critical depth just downstream of the culvert entrance and the flow in the barrel is supercritical. The barrel flows partly full over its length, and the flow approaches normal depth at the outlet end.

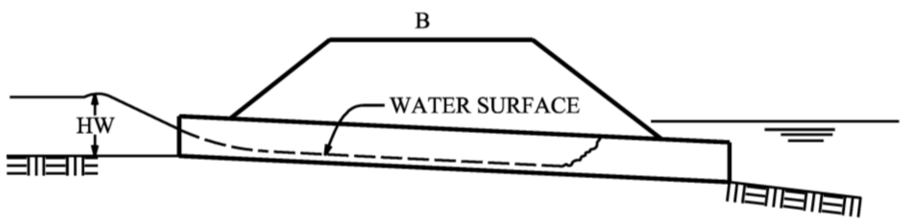
Figure 2E-2.01A: Inlet/Outlet Unsubmerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

b. Figure 2E-2.01B shows that submergence of the outlet end of the culvert does not assure outlet control. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.

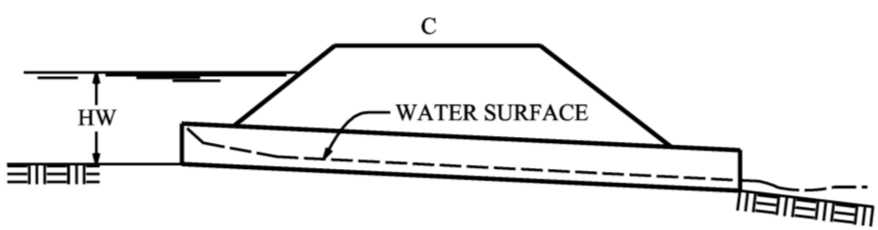
Figure 2E-2.01B: Outlet Submerged, Inlet Unsubmerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

c. Figure 2E-2.01C is a more typical design situation. The inlet end is submerged and the outlet end flows freely. Again, the flow is supercritical and the barrel flows partly full over its length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching normal depth at the downstream end of the culvert.

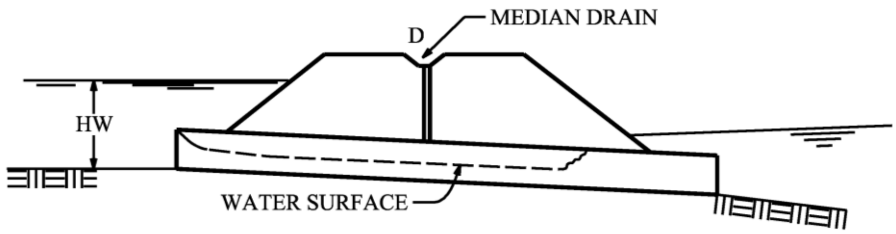
Figure 2E-2.01C: Inlet Submerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

d. Figure 2E-2.01D is an unusual condition illustrating the fact that even submergence of both the inlet and the outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. The median inlet provides ventilation of the culvert barrel. If the barrel were not ventilated, sub-atmospheric pressures could develop which might create an unstable condition during which the barrel would alternate between full flow and partly full flow.

Figure 2E-2.01D: Inlet/Outlet Submerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

2. **Outlet Control:** All of the factors influencing the performance of a culvert inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation affect culvert performance in outlet control.

   The barrel roughness is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as the Manning n value.

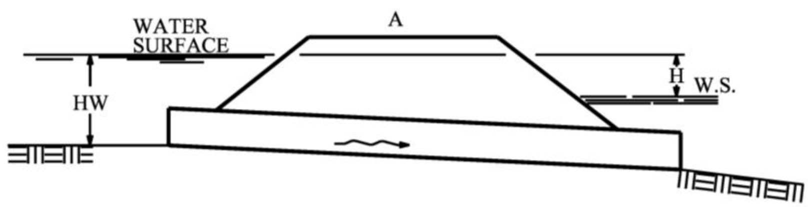
   The barrel area and barrel shape are self-explanatory. The barrel length is the total culvert length from the entrance to the exit of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of the barrel length is usually necessary to begin the design process. The barrel slope is the actual slope of the culvert barrel. The barrel slope is often the same as the natural stream slope. However, when the culvert inlet is raised or lowered, the barrel slope is different from the stream slope.

   The tailwater elevation is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define tailwater elevation.

   Hydraulics of outlet control:
   Full flow in the culvert barrel, as depicted in Figure 2E-2.02A, is the best type of flow for describing outlet control hydraulics. Outlet control flow conditions can be calculated based on energy balance. The total energy (HL) required to pass the flow through the culvert barrel is made up of the entrance loss (He), the friction loss through the barrel (H_f), and the exit loss (H_o). Other losses, including bend losses (H_b), losses at junctions (H_j), and losses at gates (H_g) should be included as appropriate.

   a. Figure 2E-2.02A represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations, but seldom actually exists.

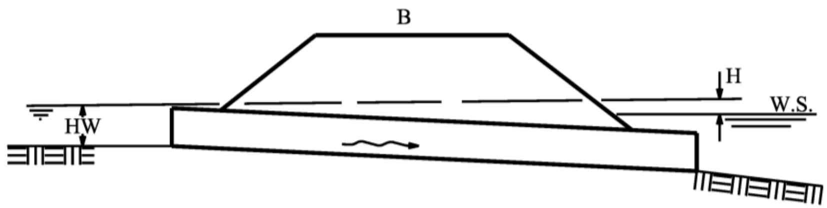
Figure 2E-2.02A: Inlet/Outlet Submerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

b. Figure 2E-2.02B depicts the outlet submerged with the inlet unsubmerged. For this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts to the culvert.

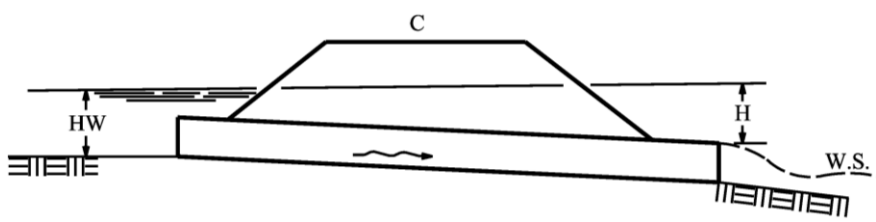
Figure 2E-2.02B: Outlet Submerged, Inlet Unsubmerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

c. Figure 2E-2.02C shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsumberged. This is a rare condition. It requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are usually high under this condition.

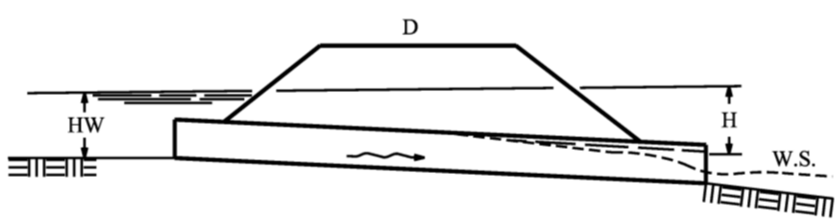
Figure 2E-2.02C: Inlet Submerged, Outlet Unsubmerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

d. Figure 2E-2.02D is more typical. The culvert entrance is submerged by the headwater and the outlet end flows freely with the low tailwater. For this condition, the barrel flows partly full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream from the outlet.

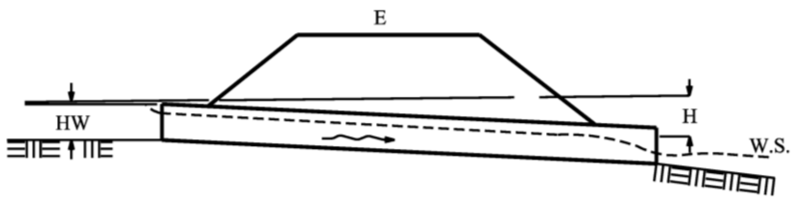
Figure 2E-2.02D: Inlet Submerged, Outlet Partially Submerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

e. Figure 2E-2.02E is also typical, with neither the inlet nor the outlet end of the culvert submerged. The barrel flows partly full over its entire length, and the flow profile is subcritical.

Figure 2E-2.02E: Inlet Unsubmerged, Outlet Unsubmerged



Source: *Hydraulic Design of Highway Culverts*, FHWA

C. Software Versus Nomographs

Culvert calculations utilizing the nomograph procedure are tedious and time consuming. Complex interactions between the headwater, tailwater, inlet control, and outlet control require initial assumptions and numerous trial and error iterations to arrive at a final design.

The designer may prefer to use culvert design software to assist in improving efficiency. HY8* Culvert Analysis Microcomputer Program (www.fhwa.dot.gov) or the Iowa DOT Culvert Program (www.iowadot.gov) are two publicly available programs that may be downloaded for free. When using the Iowa DOT Culvert Program, the Rational Method or the TR-55 Method should be used rather than the Iowa Runoff Curve to more accurately reflect urban hydrology. Proprietary design software may also be utilized.

D. Use of Inlet and Outlet Control Nomographs

The use of nomographs requires a trial-and-error solution. The solution provides reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional separate computations beyond what can be obtained from the nomographs.

Figures 2E-2.07 and 2E-2.08 show examples for inlet-control nomographs that can be used to design concrete pipe culverts. Figures 2E-2.09 through 2E-2.11 show examples for outlet-control nomographs. For culvert designs not covered by these nomographs, refer to the complete set of nomographs given in Municipal Stormwater Management, Second edition, 2003 by Thomas N. Debo, Andrew J. Reese. Following is the design procedure that requires the use of inlet- and outlet-control nomographs:

Step 1: List design data

- Q = discharge (cfs)
- L = culvert length (ft)
- S = culvert slope (ft/ft)
- K_e = inlet loss coefficient
- V = velocity (ft/s)
- TW = tailwater depth (ft)
- HW = allowable headwater depth for the design storm (ft)

Step 2: Determine trial culvert size by assuming a trial velocity 3-5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

Step 3: Find the actual HW for the trial-size culvert for inlet and outlet control.

a. For inlet control, enter inlet-control nomograph with D and Q and find HW/D for the proper entrance type. Compute HW, and, if too large or too small, try another culvert size before computing HW for outlet control.

b. For outlet control, enter the outlet-control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.

c. To compute HW, connect the length of the scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the following equation:

$$HW = H + h_0 - LS$$ **Equation 2E-2.06**

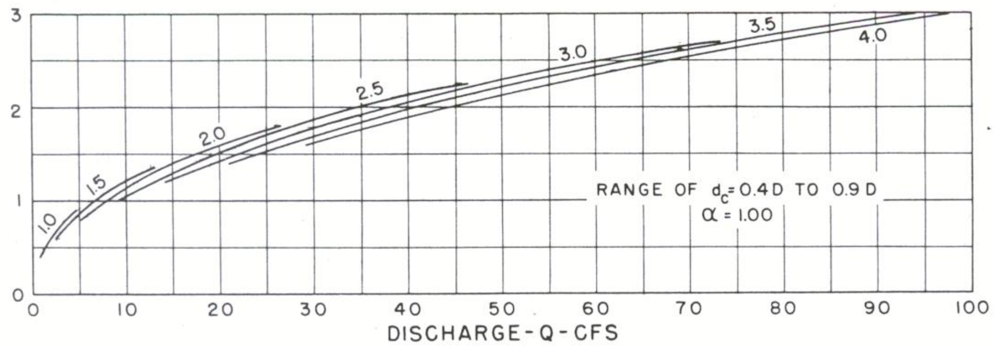
Where:

h_0 = ½ (critical depth +D), or tailwater depth , whichever is greater

Step 4: Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control. If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Because the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

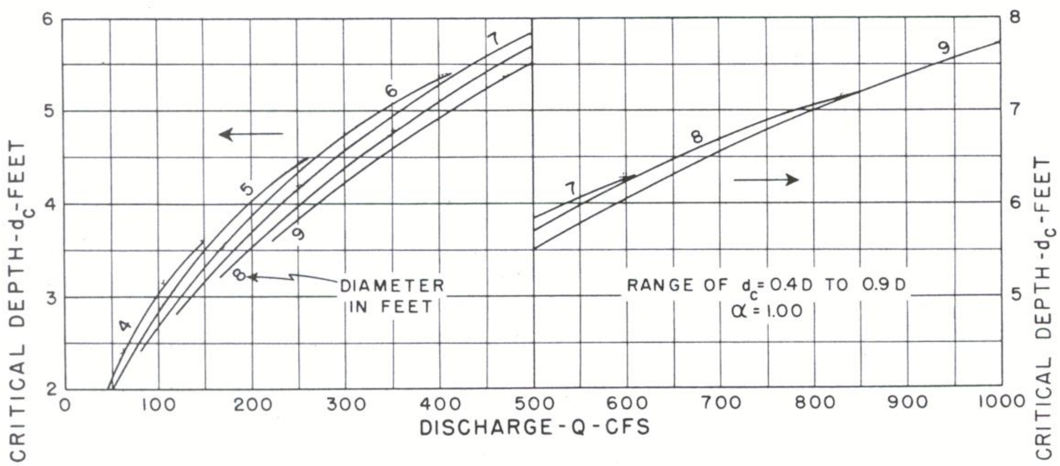
Step 5: Calculate exit velocity and expected streambed scour to determine if an energy dissipater is needed. The stream degradation may be a pre-existing condition, and the reasons and rate of degradation need to be determined. The culvert cross-sectional area may need to be increased and culvert invert initially buried if stream degradation is probable. A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Curves with length intervals of 25-50 feet are usually satisfactory for design purposes. Such computations are made much easier by available computer programs.

Figure 2E-2.03A: Critical Depth Circular Pipe, Discharge = 0 to 100 cfs



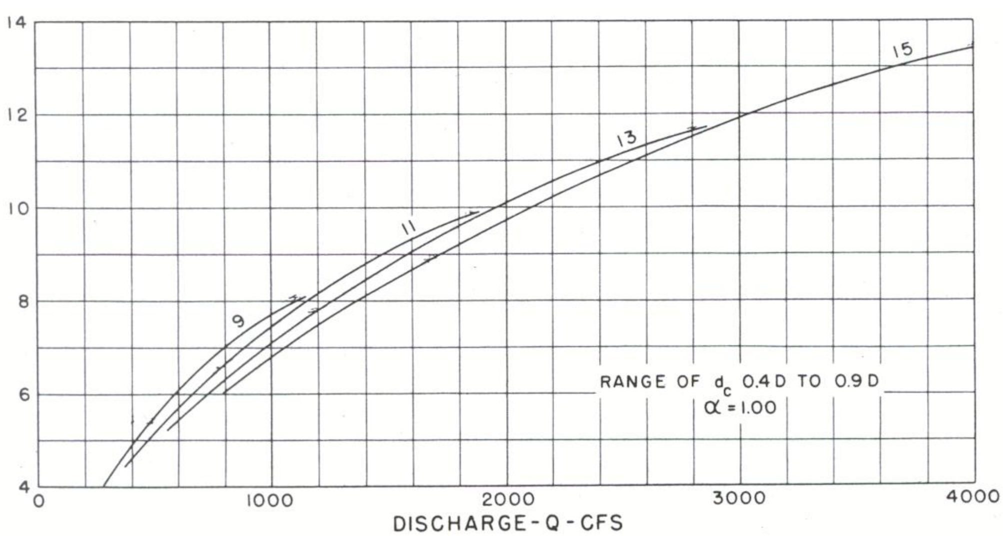
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.03B: Critical Depth Circular Pipe, Discharge = 0 to 1000 cfs



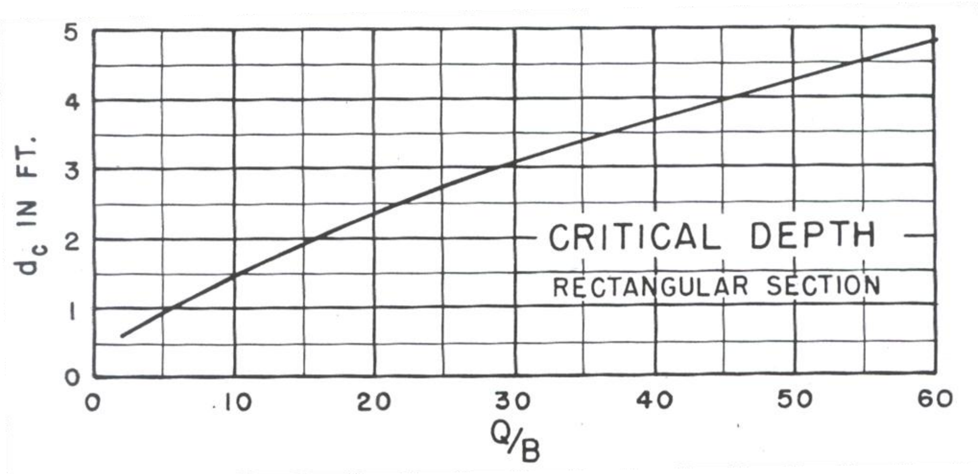
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.03C: Critical Depth Circular Pipe, Discharge = 0 to 4000 cfs



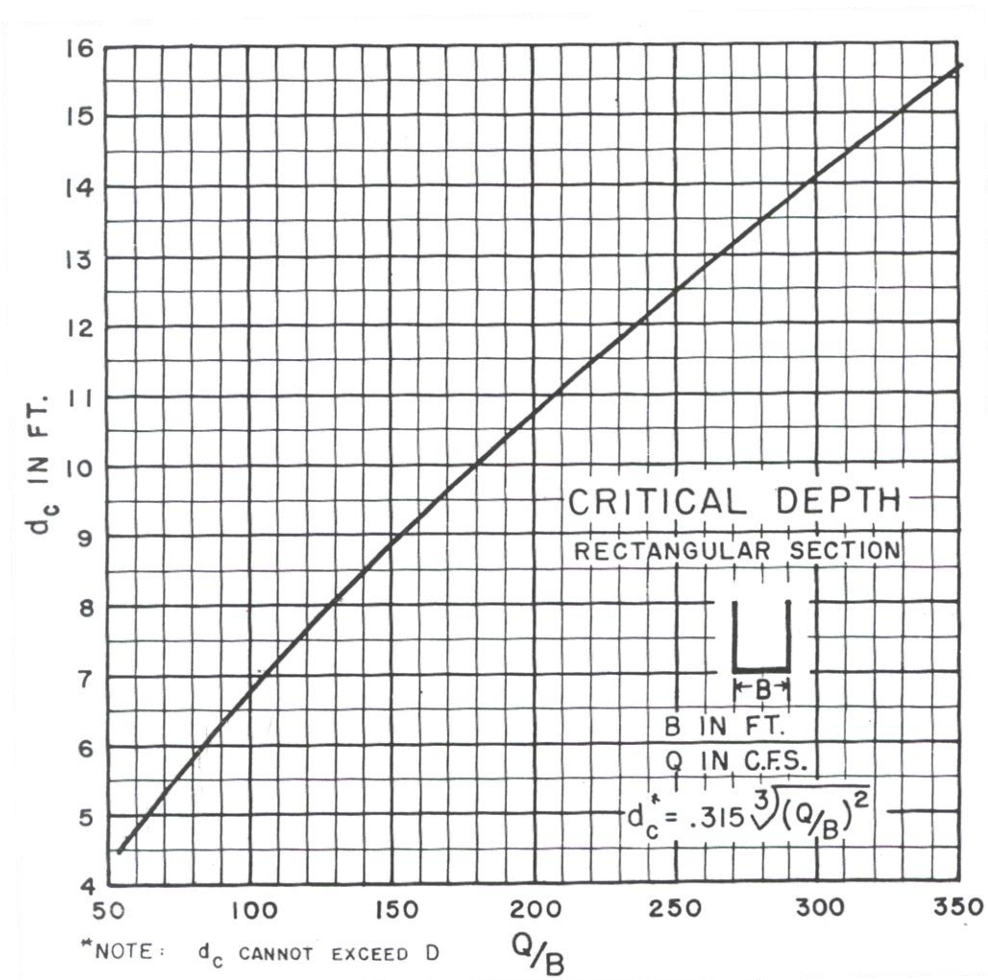
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.04A: Critical Depth Box Culvert, Q/B = 0 to 60 cfs



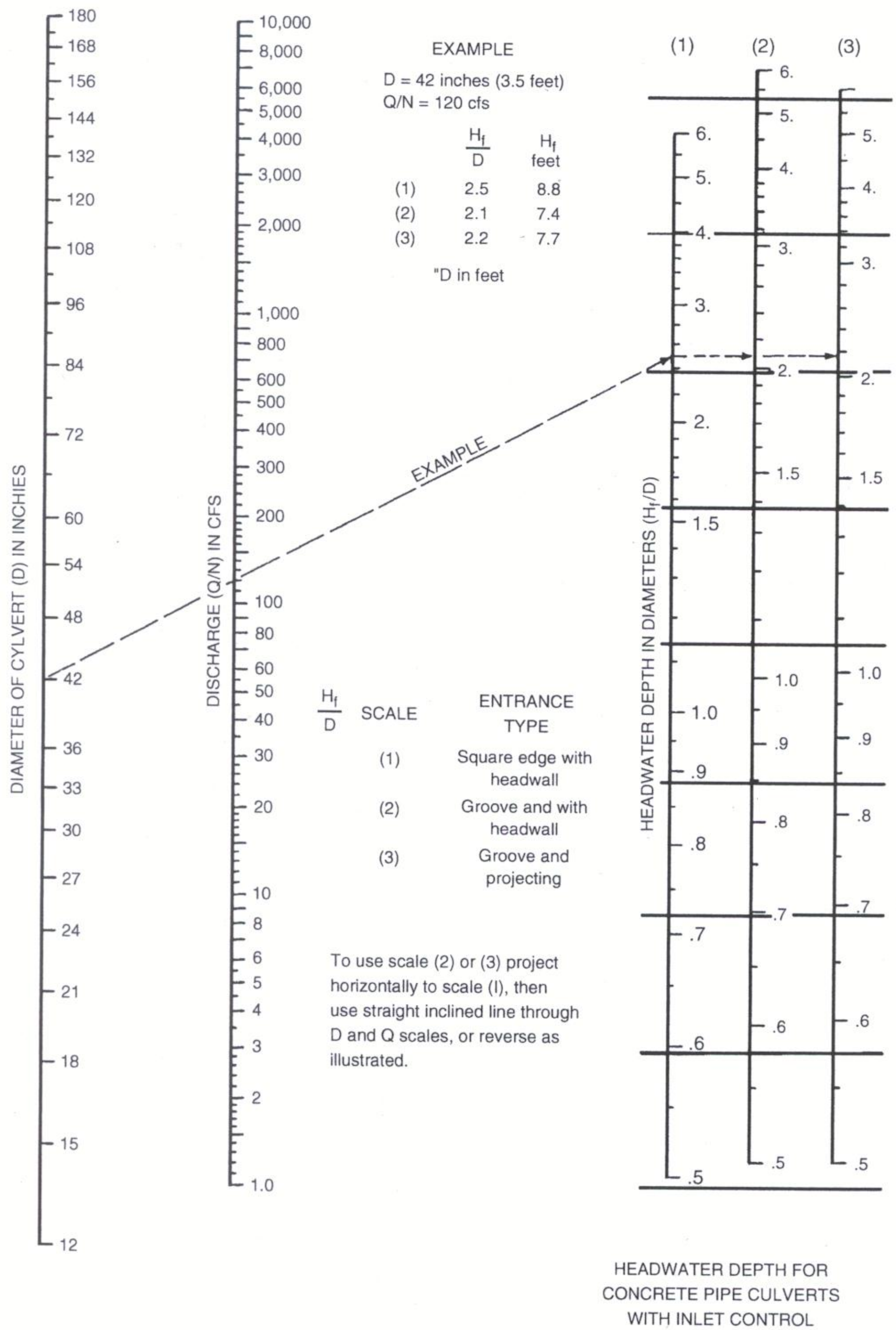
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.04B: Critical Depth Box Culvert, Q/B = 50 to 350 cfs



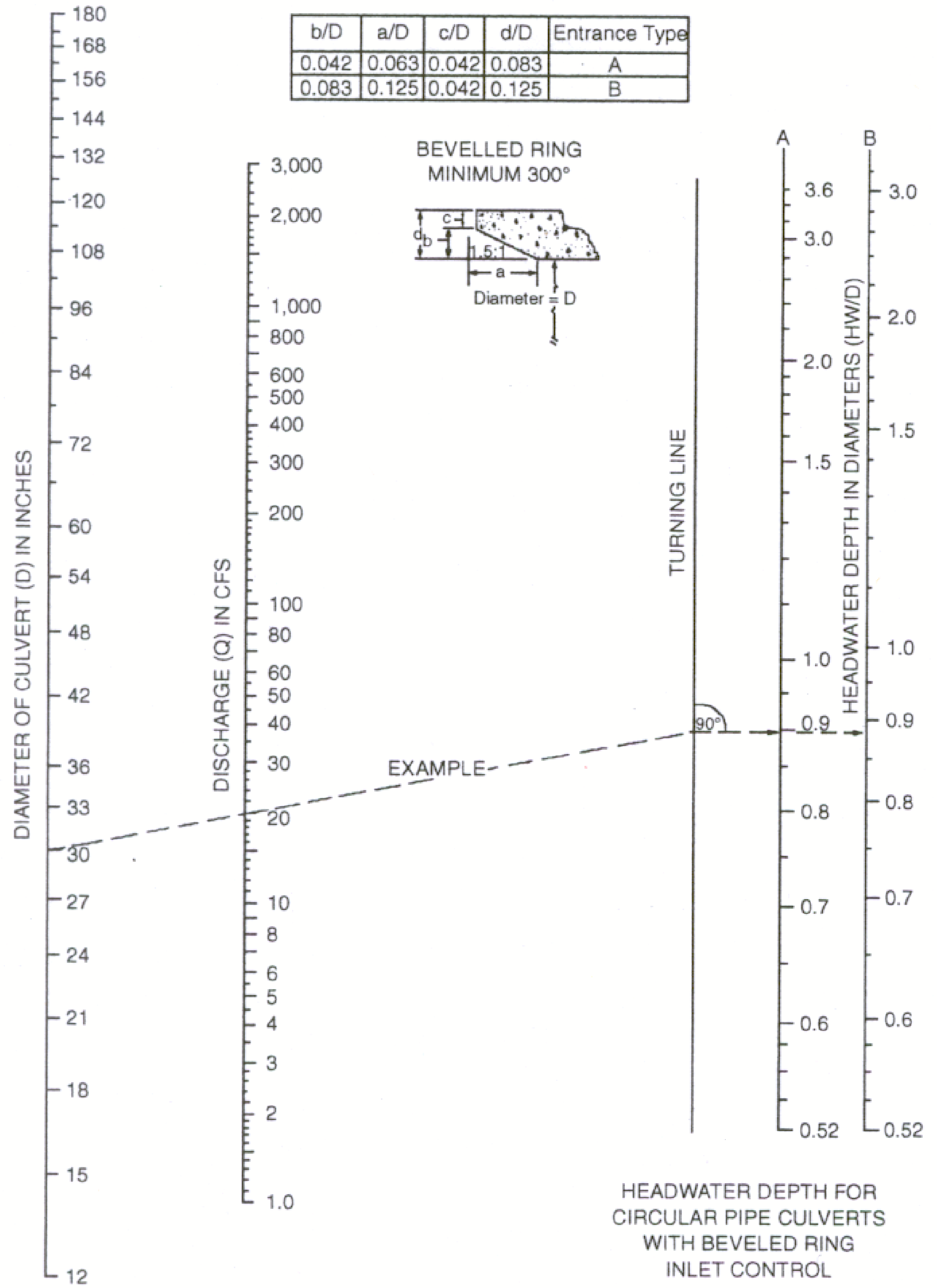
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.05: Inlet Control Nomograph



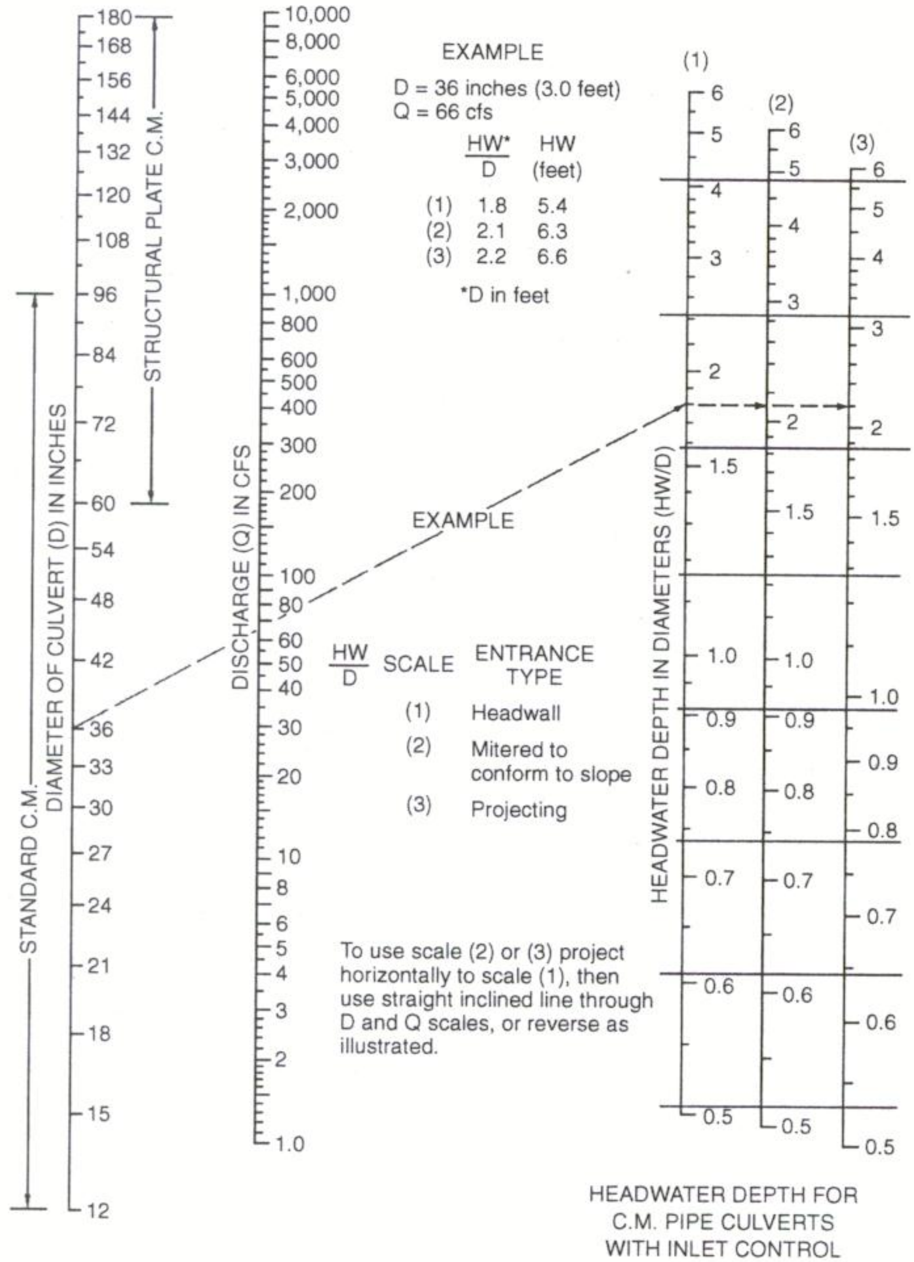
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.06: Inlet Control Nomograph



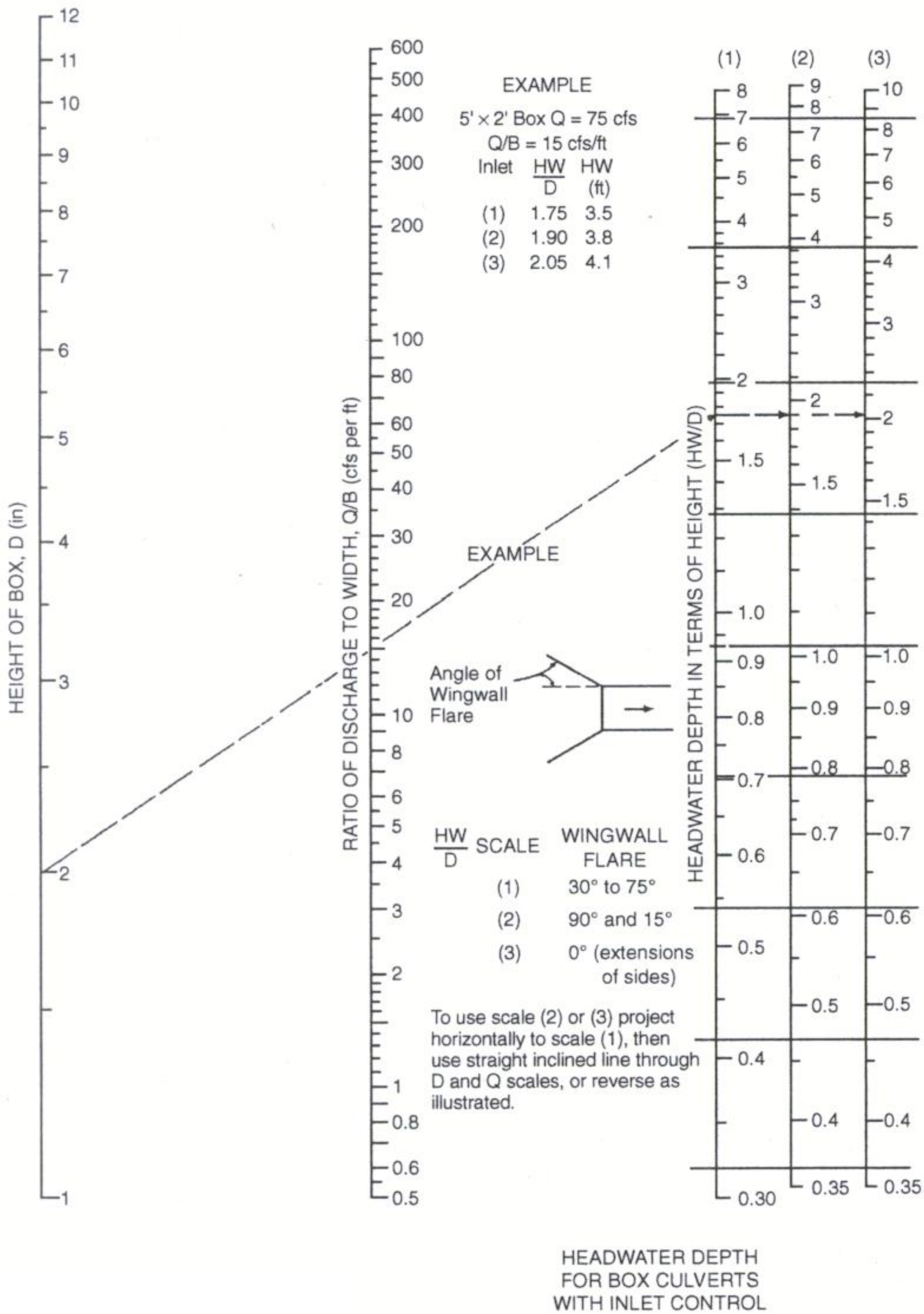
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.07: Inlet Control Nomograph



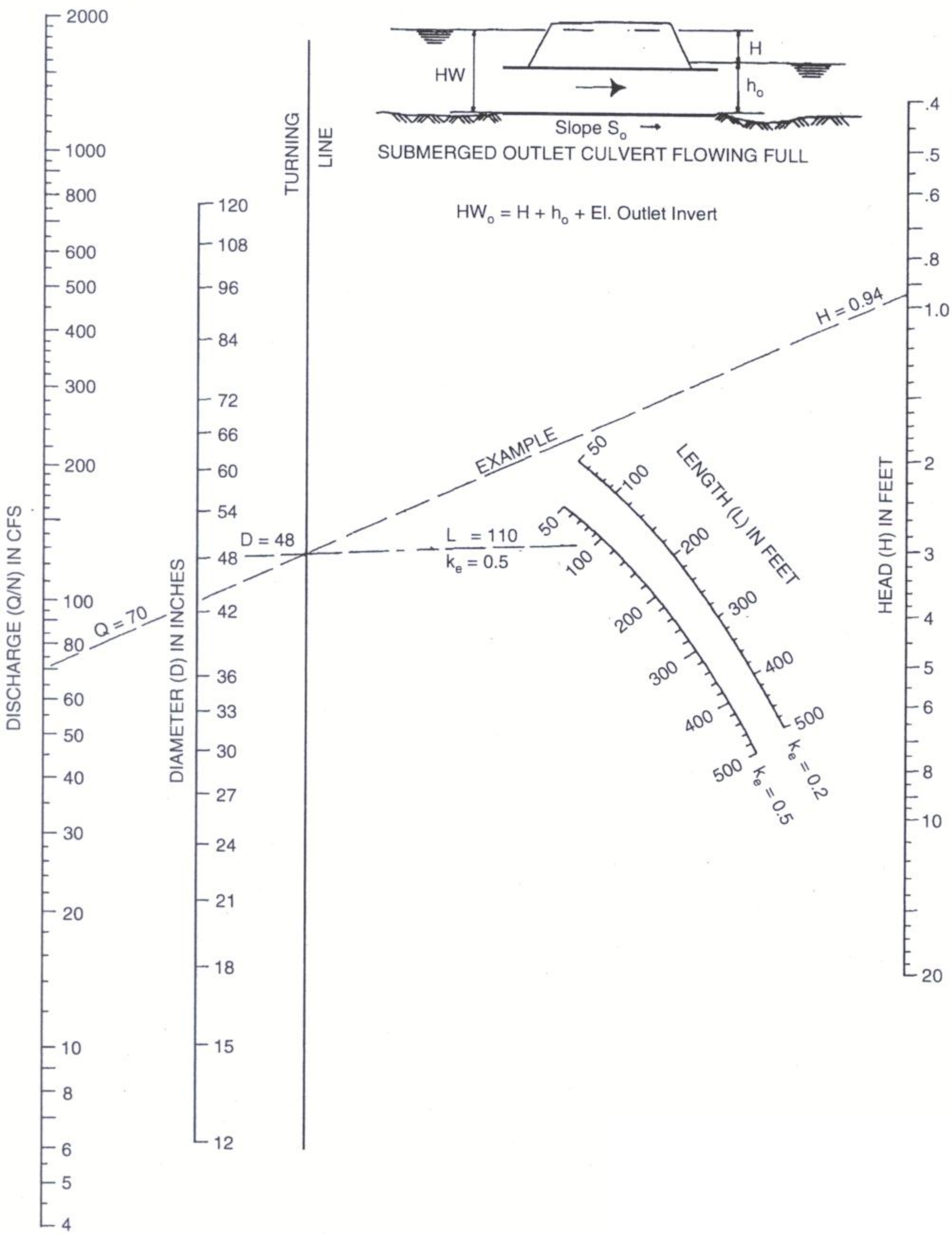
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.08: Inlet Control Nomograph



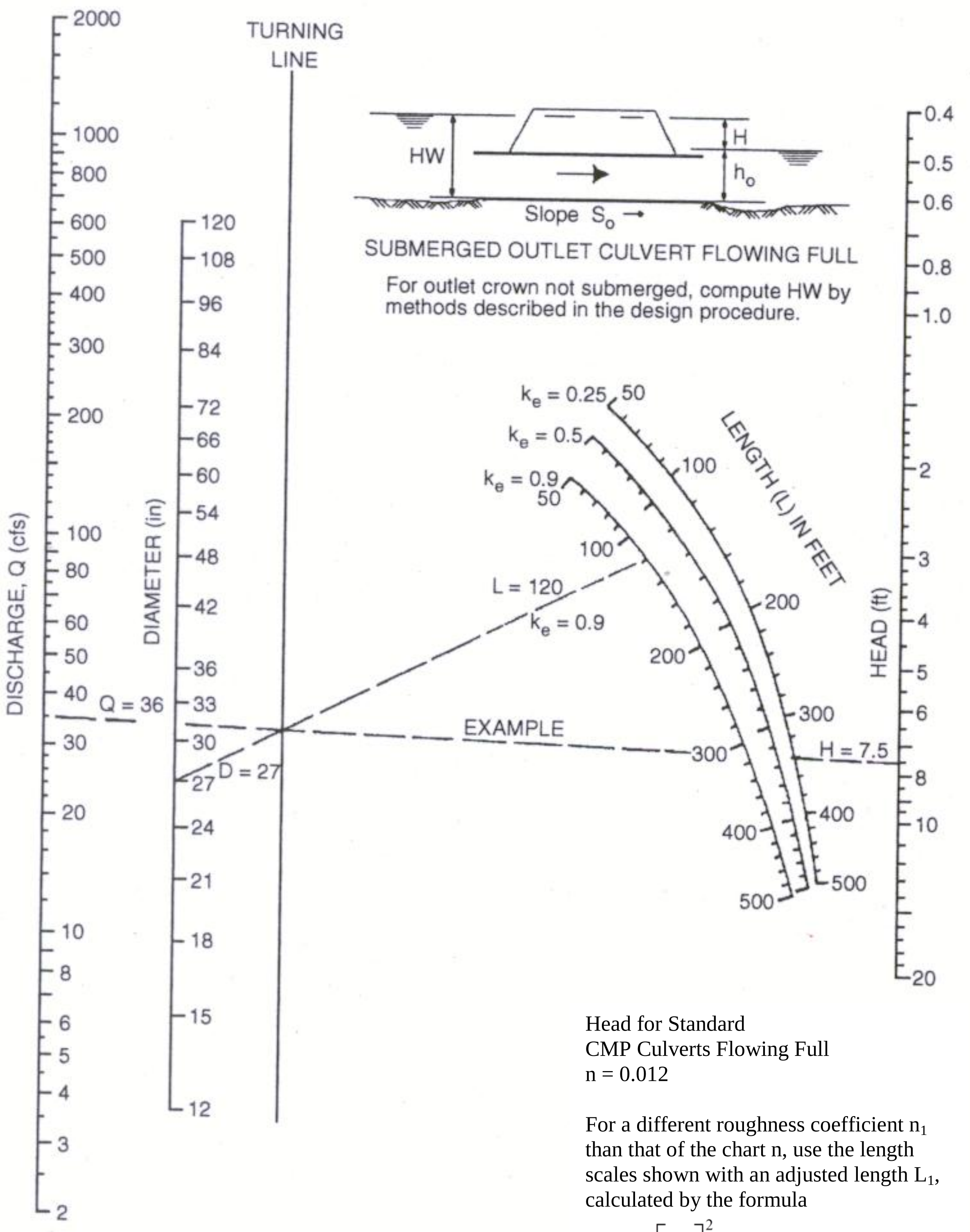
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.09: Outlet Control Nomograph



Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.10: Outlet Control Nomograph



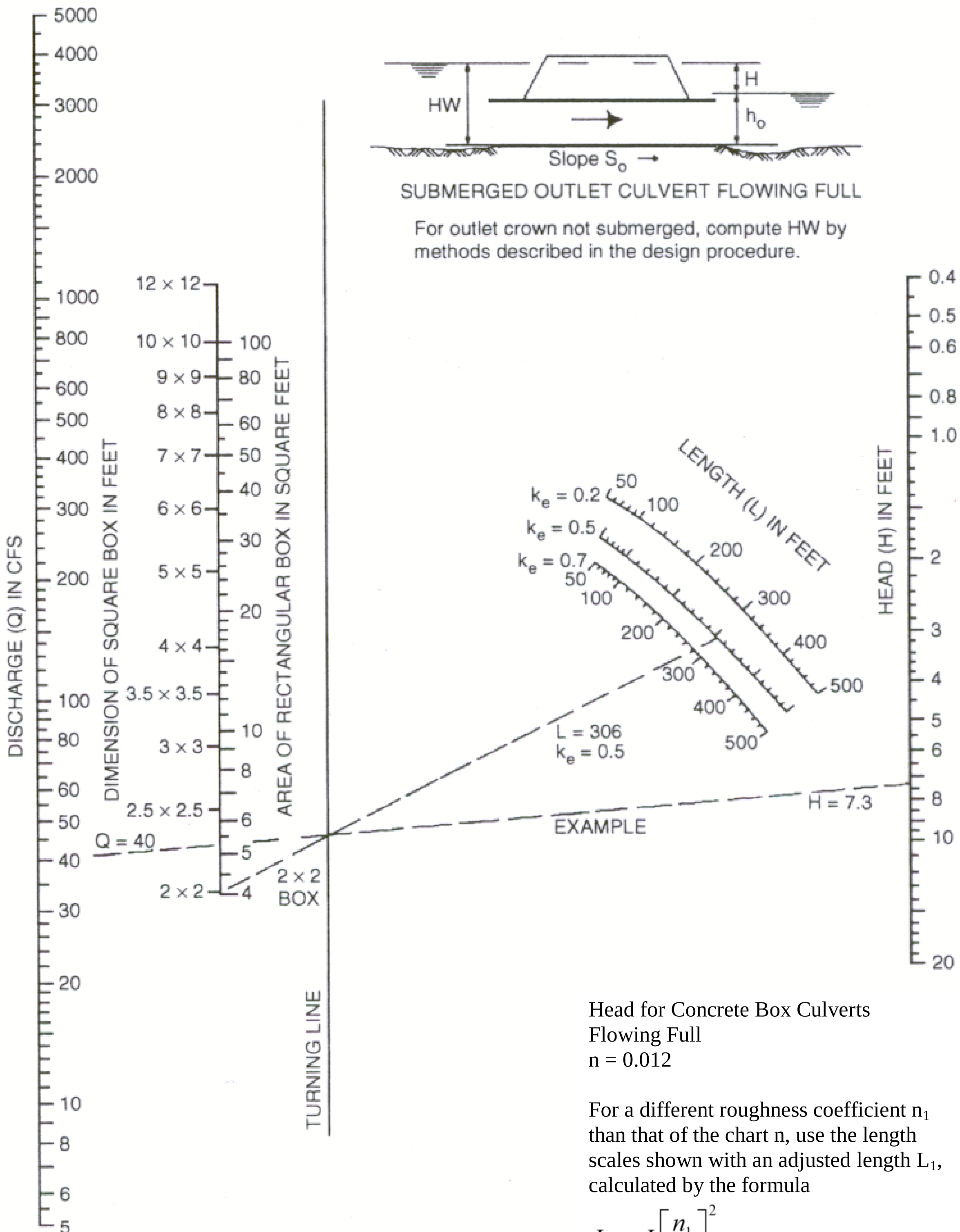
Head for Standard
CMP Culverts Flowing Full
n = 0.012

For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula

$$L_1 = L\left[\frac{n_1}{n}\right]^2$$

Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.11: Outlet Control Nomograph



Head for Concrete Box Culverts
Flowing Full
n = 0.012

For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula

$$L_1 = L\left[\frac{n_1}{n}\right]^2$$

Source: *Hydraulic Design of Highway Culverts*, FHWA

E. Culvert Design Example

The following example problem illustrates the procedures to be used in designing culverts using the nomographs. The example problem is as follows: Size a culvert given the following design conditions.

Input Data

- Discharge for 10 year flood = 70 cfs
- Discharge for 100 year flood = 176 cfs
- Allowable H_w for 10 year discharge = 4.5 feet
- Allowable H_w for 100 year discharge = 7.0 feet
- Length of culvert = 100 feet
- Natural channel invert elevations – inlet = 15.50 feet, outlet = 15.35 feet
- Culvert slope = 0.0015 feet per feet
- Tailwater depth for 10 year discharge = 3.0 feet
- Tailwater depth for 100 year discharge = 4.0 feet
- Tailwater depth is the normal depth in downstream channel
- Entrance type = groove end with headwall

Step 1: Assume a culvert velocity of 5 feet per second
Required flow area = 70 cfs/5 feet per second = 14 sq ft (for the 10 year flood).

Step 2: The corresponding culvert diameter is about 48 inches. This can be calculated by using the formula for area of a circle:
Area = $(3.14\ D^2)/4$ or D = $(\text{Area times } 4/3.14)^{0.5}$

Therefore: D = $[(14 \text{ sq ft x } 4) / 3.14]^{0.5}$ x 12 inches per feet = 50.7 inches

Step 3: A grooved-end culvert with a headwall is selected for the design. Using the inlet-control nomograph, with a pipe diameter of 48 inches and a discharge of 70 cfs; read an HW/D value of 0.93.

Step 4: The depth of headwater (HW) is (0.93) x (4) = 3.72 feet, which is less than the allowable headwater of 4.5 feet.

Step 5: The culvert is checked for outlet control. With an entrance loss coefficient K_e of 0.20, a culvert length of 100 feet, and a pipe diameter of 48 inches, an H value of 0.77 feet is determined. The headwater for outlet control is computed by the equation:
$HW = H + h_o – LS$

For the tailwater depth lower than the top of culvert, $h_o = T_w$ or 1/2 (critical depth in culvert + D), whichever is greater.
h_o = 3.0 feet or h_o = 1/2 (2.55 + 4.0) = 3.28 feet

The headwater depth for outlet control is:
$HW = H + h_o – LS$

HW = 0.77 + 3.28 – (100) x (0.0015) = 3.90 feet

Step 6: Because HW for outlet control (3.90 feet) is greater than the HW for inlet control (3.72 feet), outlet control governs the culvert design. Thus, the maximum headwater expected for a 10 year recurrence flood is 3.90 feet, which is less than the allowable headwater of 4.5 feet.

Step 7: The performance of the culvert is checked for the 100 year discharge. The allowable headwater for a 100 year discharge is 7 feet; critical depth in the 48 inch diameter culvert for the 100 year discharge is 3.96 feet. For outlet control, an H value of 5.2 feet is read from the outlet-control nomograph. The maximum headwater is:

$HW = H + h_o – LS$

$HW = 5.2 + 4.0 – (100) \times (0.0015) = 9.05$ ft

This depth is greater than the allowable depth of 7 feet; thus, a larger size culvert must be selected. Repeat steps 1-7 as necessary.

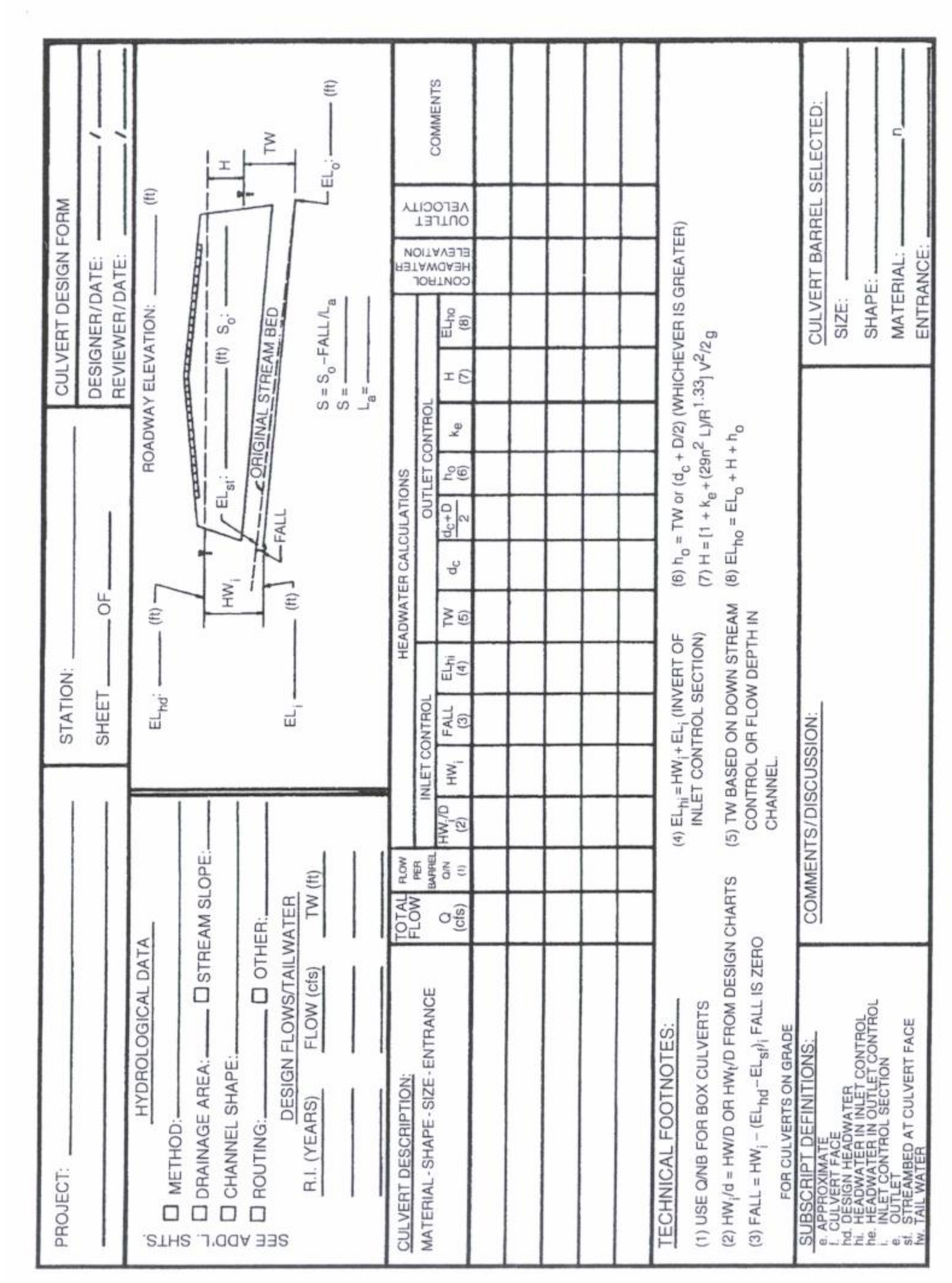
Step 8: A 54 inch diameter culvert is tried and found to have a maximum headwater depth of 3.74 feet for the 10 year discharge and of 6.97 feet for the 100 year discharge. These values are acceptable for the design conditions.

Step 9: Estimate outlet exit velocity. Because this culvert is on outlet control and discharges into an open channel downstream, the culvert will be flowing full at the flow depth in the channel. Using the 100 year design peak discharge of 176 cfs and the area of a 54 inch or 4.5 foot diameter culvert, the exit velocity will be Q = VA. Therefore:
$V = 176 / (\pi(4.5)^2 /4 = 11.8$ ft/s.

With this high velocity, some energy dissipater may be needed downstream from this culvert for streambank protection.

Step 10: The designer should check minimum velocities for low-frequency flows if the larger storm event (100 year) controls culvert design. Note: Figure 2E-2.12 provides a convenient form to organize culvert design calculations.

Figure 2E-2.12: Culvert Design Calculation



PROJECT: ______ STATION: ______ SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER/DATE: ______ / ______

REVIEWER/DATE: ______ / ______

HYDROLOGICAL DATA

SEE ADD'L. SHTS.

☐ METHOD: ______

☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______

☐ CHANNEL SHAPE: ______

☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: ______ (ft)

EL_{hd}: ______ (ft)

EL_i: ______ (ft)

HW_i FALL EL_{sf}: ______ (ft) S_o: ______ ORIGINAL STREAM BED H TW

EL_o: ______ (ft)

$S = S_o - FALL/L_a$

$S =$ ______

$L_a =$ ______

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/d = HW/D$ OR HW_f/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (29n^2 L)/R^{1.33}] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAIL WATER

COMMENTS/DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

Source: *Hydraulic Design of Highway Culverts*, FHWA

F. References

U.S. Department of Transportation. *Hydraulic Design of Highway Culverts*. Hydraulic Design Circular No. 5. 2005.

Channel Types and Structures

A. Introduction

The flow of water in an open channel is a common event in Iowa, whether in a natural channel or an artificial channel. Its movement is a difficult problem when everything is considered, especially with the variability of natural channels. However, in many cases the major features can be expressed in terms of only a few variables, whose behavior can be described adequately by a simple theory. The principal forces at work are those of inertia, gravity, and viscosity, each of which plays an important role.

B. Channel Types

Where open channel concepts are given approval by the Jurisdictional Engineer, the following design criteria should be used. The governing criteria for the selection of the channel type are based on the hydraulic carrying capacity of the channel from the area runoff.

1. Type I Channel:

a. Width at top of channel = 15 feet or less.

b. Minimum radius of curvature at centerline:
   1) Slopes greater than 3 feet/mile - 400 feet radius
   2) Slopes less than 3 feet/mile - 300 feet radius
   3) Curve protected with rip rap 75 feet radius

c. Maximum side slope = 1 vertical to 3 horizontal.

d. Minimum channel bottom = 4 feet.

e. For maximum velocity, see Section 2F-2, Tables 2F-2.03 and 2F-2.04.

f. Invert protection maybe required such as a concrete lined channel (cunette).

2. Type II Channel:

a. Width at top of channel = 15 feet to 35 feet.

b. Minimum radius of curvature at centerline:
   1) Slopes greater than 3 feet/mile - 600 feet radius
   2) Slopes less than 3 feet/mile - 500 feet radius
   3) Curve protected with rip rap - 100 feet radius

c. Maximum side slope = 1 vertical to 4 horizontal.

d. Minimum channel bottom = 6 feet.

e. For maximum velocity, see Section 2F-2, Tables 2F-2.03 and 2F-2.04.

f. Invert protection may be required such as a concrete lined channel (cunette).

3. Type III Channel:

a. Width at top of channel = 35 feet or greater.

b. Minimum radius of curvature at centerline:
   1) Slopes greater than 3 feet/mile - 700 feet radius
   2) Slopes less than 3 feet/mile - 600 feet radius
   3) Curve protected with rip rap - 200 feet radius

c. Paved concrete channel (cunette) required. Minimum width is 6 feet.

d. Maximum paved or rip rap side slope invert = 1/1 at depth established for 2 year frequency. If nets, meshes, or geo-grids are used adjacent to a paved channel bottom (no paved or rip rap side slope invert) the adjacent sideslope will not exceed 4% and have a minimum width of 2 feet on each side of the paved channel bottom.

e. Maximum side slope floodway = 1 vertical to 4 horizontal.

f. Maximum velocity in floodway = See Section 2F-2, Tables 2F-2.03 and 2F-2.04.

C. Drop Structures for Open Channel Flow

1. The use of channel drops is required when the channel would otherwise be too steep for design conditions. All drops should be designed to protect the upstream and downstream channel from erosion. Drop structure analysis may be required to determine the length of hydraulic jump and adequate erosion control measures.

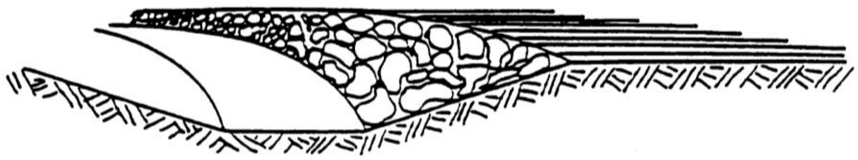
2. Vertical drops should be constructed of concrete or gabions (see Figure 2F-1.02 for example).

3. Sloped drops should be constructed of concrete, gabions, or rip rap. Rip rap drops should have a minimum of 6 inches thick gravel base and may require grouting. Engineering fabric under rip rap may be required depending on soil conditions.

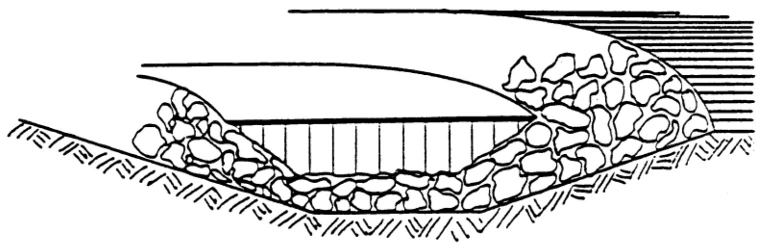
4. At drop structures, both the channel bottom and banks should be protected from erosion.

Figure 2F-1.01: Sample Channel Cross-Sections

TYPE 1 CHANNEL



TYPE 2 CHANNEL



TYPE 3 CHANNEL

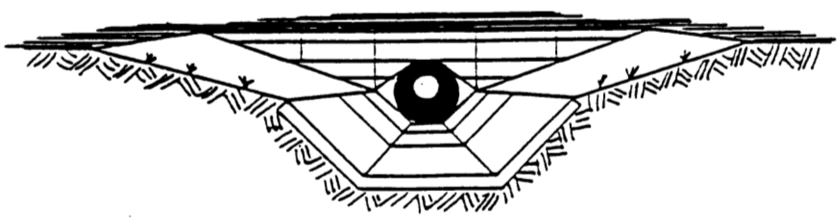
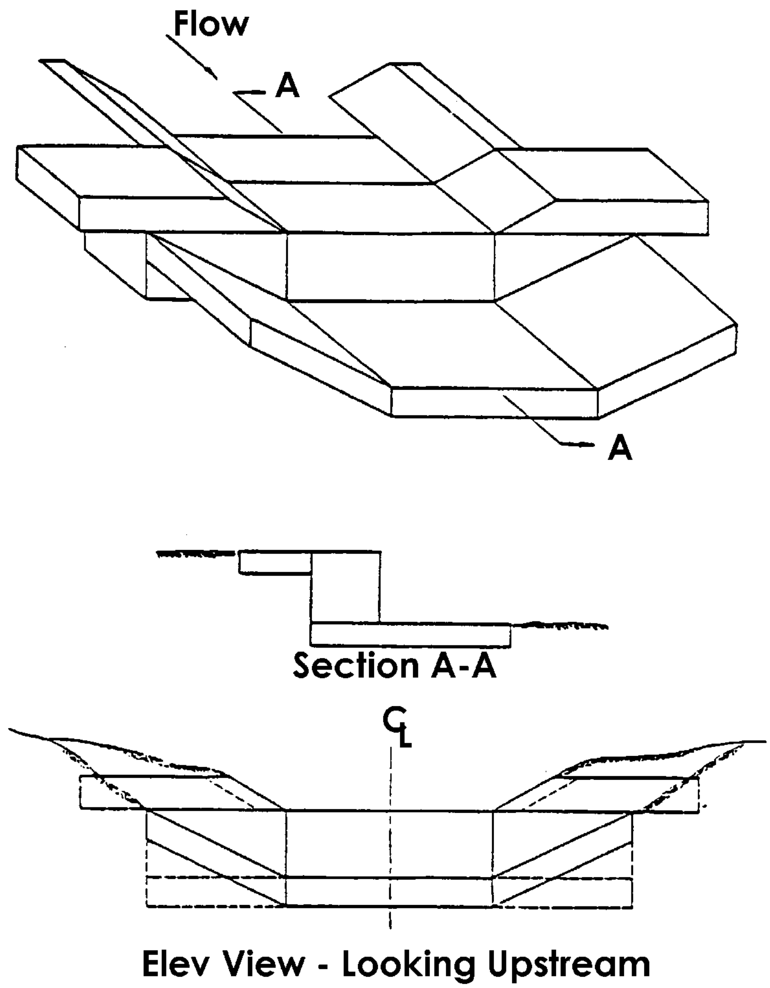


Figure 2F-1.02: Example Drop Structure for Open Channel Flow



Open Channel Flow

A. Introduction

The beginning of any channel design or modification is to understand the hydraulics of the stream. The procedures for performing uniform flow calculations aid in the selection or evaluation of appropriate depths and grades for natural or man-made channels. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's equation.

All the methods described herein will be based on the conservation of mass, momentum and energy (in the form of Bernoulli's theorem), and the Manning formula for frictional resistance. Steady uniform flow and steady non-uniform flow are the types of flow addressed in this section.

B. Definitions

Critical Flow: The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge, when the specific energy is held constant.

Froude Number: The Froude number is an important dimensionless parameter in open-channel flow. It represents the ratio of inertia forces to gravity forces. This expression for Froude number applies to any single-section channel of nonrectangular shape.

Hydraulic Jump: Hydraulic jumps occur at abrupt transitions from supercritical to subcritical flow in the flow direction. There are significant changes in the depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at stormwater management structures.

Kinetic Energy Coefficient: As the velocity distribution in a river varies from a maximum at the design portion of the channel to essentially zero along the banks, the average velocity head.

Normal Depth: For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in a steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in a steady, uniform flow and is computed from Manning's equation.

Specific Energy: Specific energy (E) is the energy head relative to the channel bottom. If the channel is not too steep (slope less than 10%), and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head. The kinetic energy correction coefficient is taken to have a value of one for turbulent flow in prismatic channels but may be significantly different from one in natural channels.

Steady and Unsteady Flow: A steady flow is when the discharge passing a given cross-section is constant with respect to time. When the discharge varies with time, the flow is unsteady. The maintenance of steady flow requires that the rates of inflow and outflow be constant and equal.

Subcritical Flow: Depths of flow greater than critical depths, resulting from relatively flat slopes. Froude number is less than one. Flow of this type is most common in flat streams.

Supercritical Flow: Depths of flow less than critical depths resulting from relatively steep slopes. Froude number is greater than one. Flow of this type is most common is steep streams.

Total Energy Head: The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The curve of the energy head from one cross-section to the next defines the energy grade line.

Uniform Flow and Non-uniform Flow: A non-uniform flow is one in which the velocity and depth vary over distance, while they remain constant in uniform flow. Uniform flow can occur only in a channel of constant cross-section, roughness, and slope in the flow direction; however, non-uniform flow can occur in such a channel or in a natural channel with variable properties.

C. Uniform Flow (Manning's Equation)

1. **Manning's Equation:** The normal depth is used to design artificial channels in a steady, uniform flow and is computed from Manning's equation:

$$Q = AV = \frac{1.486}{n}\left(AR^{2/3}\right)\left(s^{1/2}\right)$$

Equation 2F-2.01

where:

V = Channel velocity, ft/s (see Tables 2F-2.03 and 2F-2.04 for permissible velocities)
Q = Discharge, cfs
A = Cross-sectional area of flow, ft^2
n = Manning's roughness coefficient (see Section 2B-3)
R = hydraulic radius, ft = A/P
P = wetted perimeter, ft
s = slope of hydraulic grade line (pipe/channel slope), ft/ft

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. If the normal depth computed from Manning's equation is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

Strictly speaking, uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes in highway engineering, however, the Manning equation can be applied to most streamflow problems by making judicious assumptions. When the requirements for uniform flow are met, the depth (d_n) and the velocity (V_n) are said to be normal and the slopes of the water surface and channel are parallel. For practical purposes, in open channel design, minor undulations in streambed or minor deviations from the mean (average) cross-section can be ignored as long as the mean slope of the channel can be represented as a straight line.
The Manning equation can readily be solved either graphically or mathematically for the average velocity in a given channel if the normal depth is known, because the various factors in the equation are known or can be determined (the hydraulic radius can be computed from the normal depth in a given channel). Discharge (Q) is then the product of the velocity and the area of flow (A).

2. **Continuity Equation:** The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form:

$$Q = A_1V_1 = A_2V_2$$ **Equation 2F-2.02**

where:

A = flow cross-sectional area, ft^2
V = mean cross-sectional velocity, ft/s (measured perpendicular to cross-section)

The subscripts 1 and 2 refer to successive cross-sections along the flow path. The continuity equation can be used with Manning's equation to obtain steady uniform flow velocity as:

$$V = \frac{Q}{A} = \frac{1.49\left(R^{2/3}\right)\left(A^{1/2}\right)}{n}$$ **Equation 2F-2.03**

D. Energy Flow

Flowing water contains energy in two forms, potential and kinetic. The potential energy at a particular point is represented by the depth of the water plus the elevation of the channel bottom above a convenient datum plane. The kinetic energy, in feet, is represented by the velocity head:

Kinetic energy = $\frac{V^2}{2g}$ **Equation 2F-2.04**

In channel flow problems it is often desirable to consider the energy content with the channel bottom. This is called the specific energy or specific head and is equal to the depth of water plus the velocity head:

Specific energy = $d + \frac{V^2}{2g}$ **Equation 2F-2.05**

At other times it is desirable to use the total energy content (total head), which is the specific head plus the elevation of the channel bottom above a selected datum. For example, total head may be used in applying the energy equation, which states that the total head (energy) at one point in a channel carrying a flow of water is equal to the total head (energy) at any point downstream plus the energy (head) losses occurring between the two points. The energy (Bernoulli) equation is usually written:

$$d_1 + \frac{V_1^2}{2g} + Z_1 = d_2 + \frac{V_2^2}{2g} + Z_2 + h_{loss}$$ **Equation 2F-2.06**

In this equation, cross-section 2 (subscript 2) is downstream from cross-section 1 (subscript 1), Z is the elevation of channel bottom, and h_{loss} represents loss of head between cross-sections 1 and 2. A convenient way of showing specific head is to plot the water surface and the specific head lines above a profile of the channel bottom (see Figure 2F-2.01).

Note in Figure 2F-2.01 that the line obtained by plotting velocity head above the water surface is the same line as that obtained by plotting specific head above the channel bottom. This line represents the total energy, potential and kinetic, of the flow in the channel, and is called the "total head line" or "total energy line."

The slope (gradient) of the energy line is a measure of the friction slope or rate of energy head loss due to friction. Under uniform flow, the energy line is parallel to the water surface and to the streambed. For flow to occur in a channel, the total head or energy line must slope negatively (downward) in the direction of flow.

Figure 2F-2.01: Channel Flow Terms

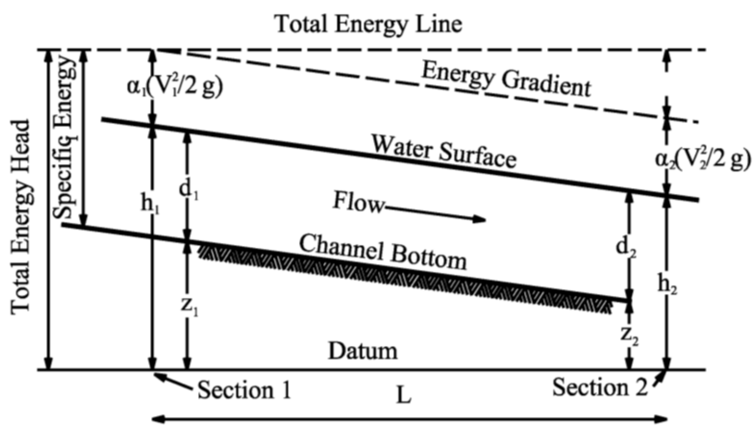
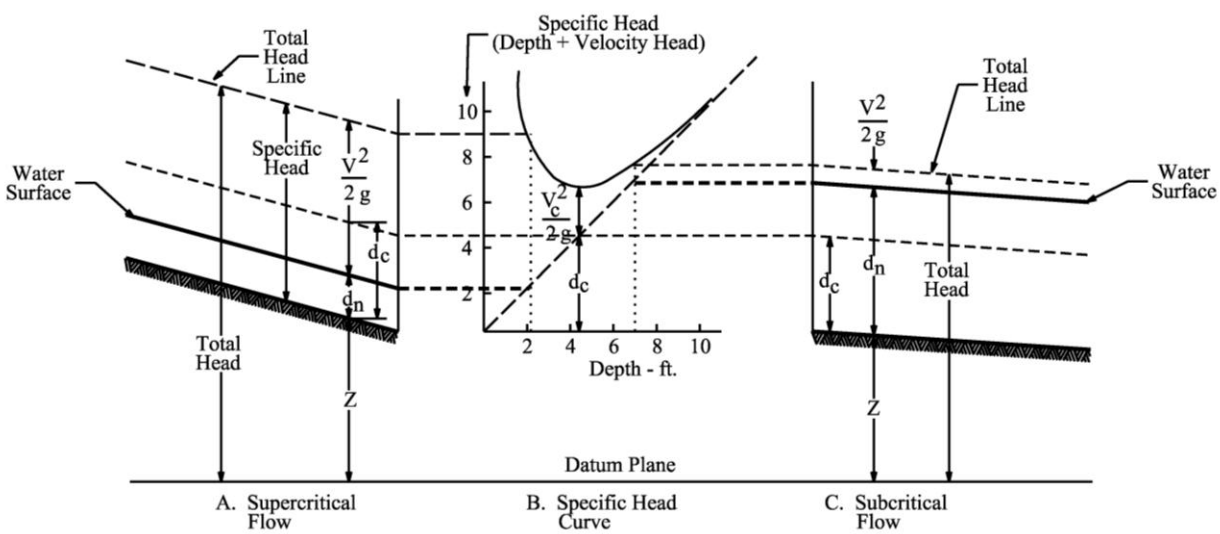


Figure 2F-2.02: Definition Sketch of Specific Head



Source: *Design Charts for Open-Channel Flow*, FHWA

1. **Critical Flow:** The relative values of the potential energy (depth) and the kinetic energy (velocity head) are important in the analysis of open-channel flow. Consider, for example, the relation of the specific head, $d + \frac{V^2}{2g}$, and the depth of a given discharge in a given channel that can be placed on various slopes. Plotting values of specific head as ordinates and of the corresponding depth as abscissa will result in a specific-head curve such as that shown in Figure 2F-2.02. The straight diagonal line is drawn through points where depth and specific head are equal. The line thus represents the potential energy, and the ordinate interval between this line and the specific head curve is the velocity head for the particular depth. A change in the discharge or in the channel size or shape will change the position of the curve, but its general shape and location above and to the left of the diagonal line will remain the same.

Note that the ordinate at any point on the specific head curve represents the total specific energy at that point. The lowest point on the curve represents flow with the minimum energy content. The depth at this point is known as critical depth (d_c) and the corresponding velocity is the critical velocity (V_c). With uniform flow, the channel slope at which critical depth occurs is known as the critical slope (S_c). The magnitude of critical depth depends only on the discharge and the shape of the channel, and is independent of the slope or channel roughness. Thus, in any given size and shape of channel, there is only one critical depth for a particular discharge. Critical depth is an important value in hydraulic analysis because it is a control in reaches of non-uniform flow whenever the flow changes from subcritical to supercritical.

Typical occurrences of critical depths are:

a. Entrance to a restrictive channel, such as a culvert or flume, on a steep slope

b. At the crest of an overflow dam or weir

c. At the outlet of a culvert or flume discharging with a free fall or into a relatively wide channel or a pond in which the depth is not enough to submerge critical depth in the culvert or flume.

2. Critical Depth Calculations:

a. The general equation for determining critical depths on the discharge rate and channel geometry is:

$$\frac{Q^2}{g} = \frac{A^3}{T}$$ **Equation 2F-2.07**

where:

g = acceleration of gravity, ft/s^2 (32.2)
A = cross-sectional area, ft^2
T = top width of water surface, ft

A trial and error procedure is needed to solve Equation 2F-2.07. The following guidelines are presented for evaluating critical flow conditions of open channel flow:

1) A normal depth of uniform flow within about 10% of critical depth is unstable (relatively large depth changes are likely for small changes in roughness, cross-sectional area, or slope) and should be avoided in design, if possible.
2) If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
3) If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
4) If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.
5) If an unstable critical depth cannot be avoided in design, the least favorable type of flow should be assumed for the design.

b. The Froude number, Fr, calculated by the flowing equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = \frac{V}{\left(\frac{gA}{T}\right)^{1/2}}$$ **Equation 2F-2.08**

where:

Fr = Froude number (dimensionless)
V = velocity of flow, ft/s
g = acceleration of gravity, ft/s^2 (32.2)
T = top width of flow, ft

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

3. **Critical Slope:** Critical slope is that channel slope for a particular channel and discharge, at which the normal depth for uniform flow will be the same as the critical depth. Critical slope varies with both the roughness and geometric shape of the channel and with the discharge. For large circular cross-section pipes, and for pipe-arch and oval pipe sections, a direct reading can be made on the part-full flow charts for critical depth, specific head, and critical slope (for certain values of n).

4. **Supercritical Flow:** Points on the left of the flow point of the specific head curve [Figure 2F-2.02 (B)] are for channel slopes steeper than critical (supercritical or steep slopes), and indicate relatively shallow depths and high velocities [Figure 2F-2.02 (A)]. Such flow is called supercritical flow. It is difficult to handle because violent wave action occurs when either the direction of flow or the cross-section is changed. Flow of this type is common in steep streams. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth.

5. **Subcritical Flow:** Points on the right of the low point of the specific head curve [Figure 2F-2.02 (B)] are for slopes flatter than critical (subcritical or mild slopes) and indicate relatively large depths with low velocities [Figure 2F-2.02 (C)]. Such flow is called subcritical flow. It is relatively easy to handle through transitions because the wave actions are tranquil. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either critical depth or the water surface elevation in a pond or larger downstream channel. Figures 2F-2.02 (A) and 2F-2.02 (C) indicate the relationship of supercritical and subcritical flows, respectively, to the specific head curve.

E. Non-uniform Flow

Flow that varies in depth and velocity along the channel is called non-uniform. Truly uniform flow rarely exists in either natural or man-made channels, because changes in channel section, slope, or roughness cause the depths and average velocities of flow to vary from point to point along the channel, and the water surfaces will not be parallel to the streambed. Although moderate non-uniform flow actually exists in a generally uniform channel, it is usually treated as uniform flow in such cases. Uniform flow characteristics can readily be computed and the computed values are usually close enough to the actual for all practical purposes. The types of non-uniform flow are innumerable, but certain characteristic types are more common.

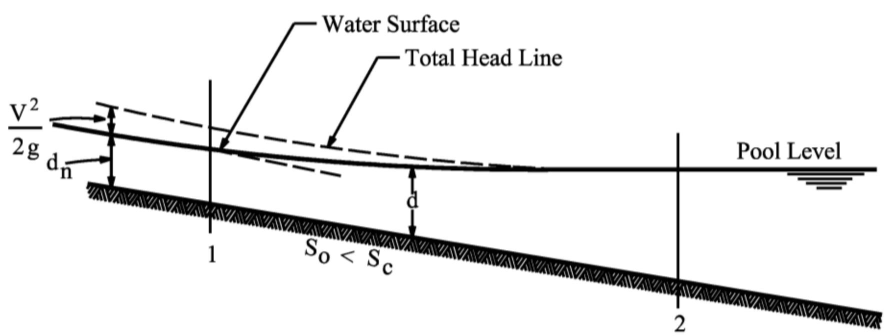
With subcritical flow, a change in channel shape, slope, or roughness affects the flow for a considerable distance upstream, and thus the flow is said to be under downstream control. If an obstruction, such as a culvert, causes ponding, the water surface above the obstruction will be a smooth curve asymptotic to the normal water surface upstream and to the pool level downstream (see Figure 2F-2.03).

Another example of downstream control occurs where an abrupt channel enlargement, as at the end of a culvert not flowing full, or a break in grade from a mild to a steep slope, causes a drawdown in the flow profile to critical depth. The water surface profile upstream from a change in section or a break in channel slope will be asymptotic to the normal water surface upstream, but will drop away from the normal water surface on approaching the channel change or break in slope. In these two examples, the flow is non-uniform because of the changing water depth caused by changes in the channel slope or channel section. Direct solution of open-channel flow by the Manning equation or by the charts in this section is not possible in the vicinity of the changes in the channel section or channel slope. With supercritical flow, a change in the channel shape, slope, or roughness cannot be reflected upstream except for very short distances. However, the change may affect the depth of flow at downstream points; thus, the flow is said to be under upstream control.

Most problems in highway drainage do not require the accurate computation of water surface profiles. However, the designer should know that the depth in a given channel may be influenced by conditions either upstream or downstream, depending on whether the slope is steep (supercritical) or mild (subcritical).

Figure 2F-2.03 shows a channel on a mild slope, discharging into a pool. The vertical scale is exaggerated to illustrate the case more clearly. Cross-section 1 is located at the end of uniform channel flow in the channel and cross-section 2 is located at the beginning of the pool. Depth 2 is located at the beginning of the pool. The depth of flow (d) between sections 1 and 2 is changing and the flow is non-uniform. The water surface profile between the sections is known as backwater curve and is characteristically very long.

Figure 2F-2.03: Water Surface Profile in Flow from a Channel to a Pool

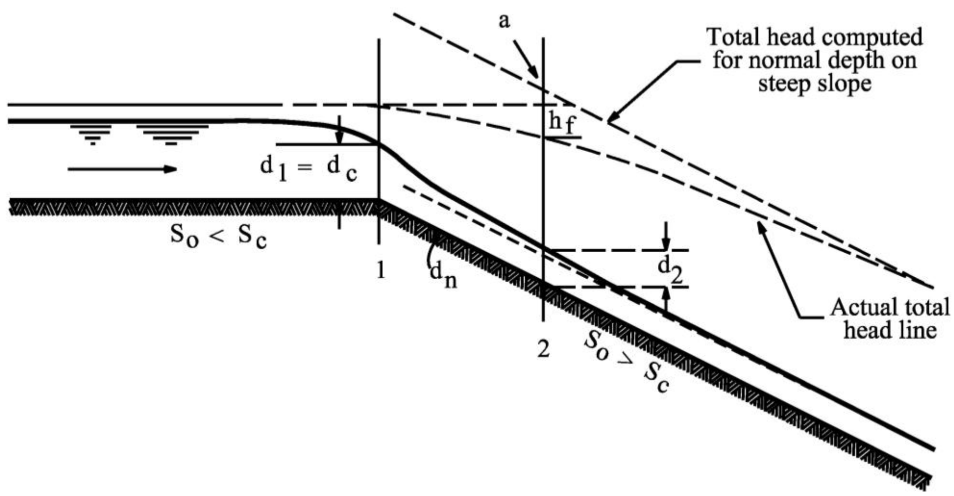


Source: *Design Charts for Open-Channel Flow,* FHWA

Figure 2F-2.04 shows a channel in which the slope changes from subcritical to supercritical. The flow profile passes through critical depth near the break in slope (section 1). This is true whether the upstream slope is mild, as in the sketch, or whether the water above section 1 is ponded, as would be the case if section 1 were the crest of the spillway of a dam. If, at section 2, the total head were computed, assuming normal depth on the steep slope, it would plot (point a on the sketch) above the elevation of the total head at section 1. This is physically impossible, because the total head line must slope downward in the direction of flow. The actual total head line will take the position shown, and have a slope approximately equal to S_c at section 1 and approaching slope S_0 farther downstream. The drop in the total head line h_f between sections 1 and 2 represents the loss in energy due to friction. At section 2 the actual depth d_2 is greater than d_n because sufficient acceleration has not occurred and the assumption of normal depth at this point would clearly be in error. As section 2 is moved downstream so that total head for the normal depth drops below the pool elevation above section 1, the actual depth quickly approaches the normal depth for the steep channel. This type of water surface curve (section 1 to section 2) is characteristically much shorter than the backwater curve discussed in the previous paragraph.

Another common type of non-uniform flow is the drawdown curve to critical depth which occurs upstream from section 1 (Figure 2F-2.04) where the water surface passes through the critical depth. The depth gradually increases upstream from critical depth to normal depth, provided the channel remains uniform through a sufficient length. The length of the drawdown curve is much longer than the curve from critical depth to normal depth in a steep channel.

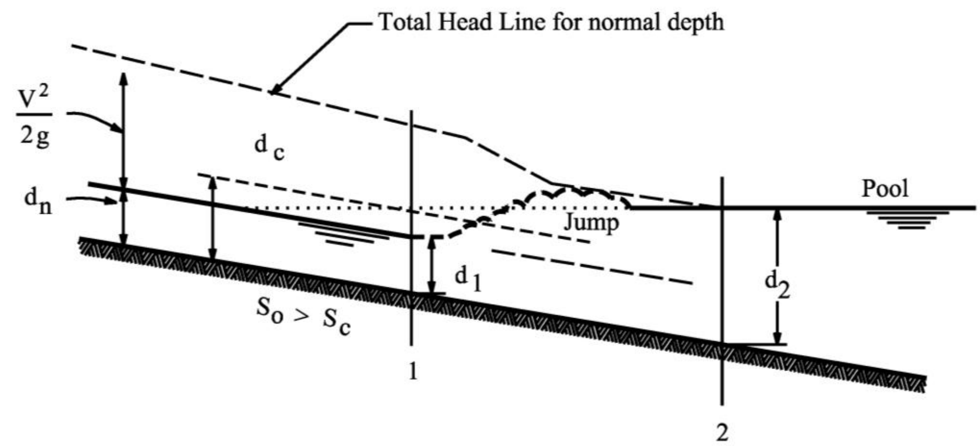
Figure 2F-2.04: Water Surface Profile in Changing from Subcritical to Supercritical Channel Slope



Source: *Design Charts for Open-Channel Flow*, FHWA

Figure 2F-2.05 shows a special case for a steep channel discharging into a pool. A hydraulic jump makes a dynamic transition from the supercritical flow in a pool. This situation differs from that shown in Figure 2F-2.03 because the flow approaching the pool in Figure 2F-2.05 is supercritical and the total head in the approach channel is large relative to the pool depth. In general, the supercritical flow can be changed to subcritical flow only by passing through a hydraulic jump. The violent turbulence in the jump dissipates energy rapidly, causing a sharp drop in the total head line between the supercritical and subcritical states of flow. A jump will occur whenever the ratio of the depth $d_1$in the approach channel to the depth d_2 in the downstream channel reaches a specific value. Note in Figure 2F-2.05 that normal depth in the approach channel persists well beyond the point where the projected pool level would intersect the water surface of the channel at normal depth. Normal depth can be assumed to exist on the steep slope upstream from section 1, which is located about at the toe of the jump.

Figure 2F-2.05: Water Surface Profile Illustrating Hydraulic Jump



Source: *Design Charts for Open-Channel Flow*, FHWA

F. Hydraulic Jump

1. **General:** The hydraulic jump consists of an abrupt rise of the water surface in the region of impact between rapid and tranquil flows. Flow depths before (supercritical depth, d_1) and after (subcritical depth, d_2) the jump are less than and greater than critical depth, respectively. The depth d_1 is calculated based on the hydraulics of the channel. The depth d_2 is calculated as shown in part 2. The zone of impact of the jump is accompanied by large-scale turbulence, surface waves, and energy dissipation. The hydraulic jump in a channel may occur at locations such as:

   a. The vicinity of a break in grade where the channel slope decreases from steep to mild.

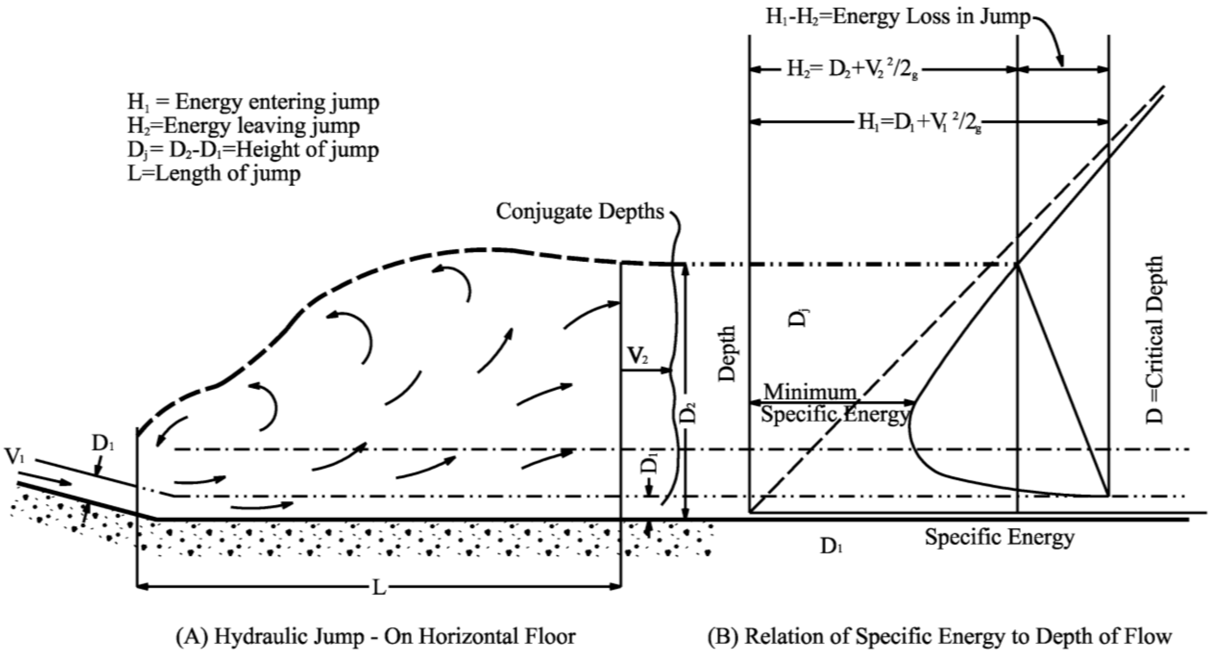
   b. A short distance upstream from channel constrictions such as those caused by bridge piers.

   c. A relatively abrupt converging transition.

   d. A channel junction where rapid flow occurs in a tributary channel and tranquil flow in the main channel.

   e. Long channels where high velocities can no longer be sustained on a mild slope.

Figure 2F-2.06: Hydraulic Jump and Depth of Flow



2. **Hydraulic Jump Computations:** The method for calculating the length of the hydraulic jump and the resulting flow depth and velocity downstream of the jump is discussed in detail in FHWA's Hydraulic Engineering Circular No. 14 (HEC-14), Hydraulic Design of Energy Dissipators for Culverts and Channels. Due to the complex energy calculations required to analyze hydraulic jumps, the use of appropriate hydraulic design software is encouraged.

Table 2F-2.03: Permissible Velocities for Channels with Erodible Linings, Based on Uniform Flow in Continuously Wet, Aged Channels

Soil Type or Lining (earth; no vegetation)	**Maximum Permissible Velocities for…**		
	Clear Water (fps)	***Water Carrying Fine Silts (fps)***	***Water Carrying Sand and Gravel (fps)***
Fine sand (non-colloidal)	1.5	2.5	1.5
Sandy loam (non-colloidal)	1.7	2.5	2.0
Silt loam (non-colloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay	3.7	5.0	3.0
Graded, loam to cobbles (non-colloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (non-colloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (non-colloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0
Fabric and excelsior mat	7.0	7.0	7.0
Dry rip rap/gabions	10.0	10.0	10.0
Concrete pilot channel	Use grass permissible velocity - Table 2F-2.04		

Table 2F-2.04: Permissible Velocities for Channels Lined with Uniform Stands of Various Grass Covers, Well Maintained[1]

Cover	Slope Range (percent)	Permissible Velocity on…	
		Erosion Resistant Soils (fps)	***Easily Eroded Soils (fps)***
Bermudagrass	0 to 5	8	6
	5 to 10	7	5
	Over 10	6	4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0 to 5	7	5
	5 to 10	6	4
	Over 10	5	3
Grass mixture	0 to 5	5	4
	5 to 10	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0 to 5	3.5	2.5
Common lespedeza[2] Sudangrass	0 to 5[3]	3.5	2.5

[1] Use velocities of 5 fps only where good covers and proper maintenance can be obtained.
[2] Annuals, used on mild slopes or as temporarily protection until permanent covers are established.
[3] Use on slopes steeper than 5% is not recommended.

Source: From *Handbook of Channel Design for Soil and Water Conservation*

G. References

U.S. Department of Transportation. *Design Charts for Open-Channel Flow*. Hydraulic Design Series No. 3. 1961.

U.S. Soil Conservation Service. *Handbook of Channel Design for Soil and Water Conservation*. 1947.

General Information for Detention Practices

A. Introduction

Storm runoff detention is considered a viable method to reduce runoff impacts. Temporarily detaining a specified volume of runoff can significantly reduce downstream flooding, as well as pipe and channel requirements in urban areas. The main purpose of a detention facility is to store the excess storm runoff associated with increased basin imperviousness and discharge this excess at a rate similar to the rate experienced from the basin without development.

1. Excess storm runoff will be judged in comparison to the site in its pre-developed condition and should include all increases in stormwater resulting from any of the following:

    a. An increase in the impervious surface of the site, including all additions of buildings, roads and parking lots.

    b. Changes in soil absorption caused by compaction during development.

    c. Modifications in contours, including the filling or draining of small depressional areas, alterations of drainageways, or regrading of slopes.

    d. Site clearing.

    e. Alteration of drainageways or installation of collection systems to intercept street flows or to replace swales or other drainageways.

    f. Alteration of subsurface flows, including any groundwater dewatering or diversion practices such as curtain drains.

    g. Any increase in runoff that occurs by piping building downspouts that previously discharged to splash blocks.

2. Pre-developed condition means those hydraulic and hydrologic site characteristics existing prior to the development being proposed and includes all the natural storage areas and drainageways plus existing farm drainage tiles and highway drainage structures. The Jurisdictional Engineer may require the pre-developed condition to be considered in a natural state (without any man-made development) if drainage problems are occurring down stream due to existing development at the proposed site or in the basin.

3. Developed condition means those hydraulic and hydrologic site characteristics that occur following the completion of the proposed development that may result in excess runoff.

4. Post-developed peak runoff is expected to exceed pre-developed runoff from a similar storm event. Even if calculated time of concentration or curve number tables suggest lower post-developed runoff, developed sites generally have more impervious areas, compacted soils, change in soil horizon, and differing vegetation from undeveloped conditions. There may be exceptions, but careful consideration of the hydrologic method and sufficient engineering judgment are necessary to ensure calculated results meet reasonable expectations.

B. Storm Detention Regulations

The developer, subdivider, or applicant should construct stormwater detention facilities designed by a Professional Engineer licensed in the State of Iowa that meets the criteria of this section. Storm basins will follow Iowa Department of Natural Resources Rules and Regulations as described in the Iowa Administrative Code, Title V, Chapter 70.

1. Conditions that Require an Iowa DNR Permit:

a. Dams: Approval by the department for construction, operation, or maintenance of a dam in the floodway or floodplain of any water source will be required when the dimensions and effects of such dams exceed the thresholds established by this rule:

1) Any dam designed to provide a sum of permanent and temporary storage exceeding 50 acre-feet at the top of dam elevation, or 25 acre-feet if the dam does not have an emergency spillway, and which has height of 5 feet or more.
2) Any dam designed to provide permanent storage in excess of 18 acre-feet and has a height of 5 feet or more.
3) Any dam across a stream draining more than 10 square miles (rural only).
4) Any dam located within one mile of an incorporated municipality, if the dam has a height of 10 feet or more, stores 10 acre-feet or more at the top of the dam elevation, and is situated such that the discharge from the dam will flow through the incorporated areas.

b. Low Head Dams: Any low head dam on a stream draining two or more square miles in an urban area, or 10 or more square miles in a rural area.

c. Levees or Dikes: Approval by the department for construction, operation, and maintenance of levees or dikes will be required in the following instances:

1) Rural Areas: In rural areas, any levees or dikes located on the floodplain or floodway of any stream or river draining more than 10 square miles.

2) Urban Areas: In urban areas, any levee or dike along any river or stream draining more than two square miles.

2. Design Storm: The design storm is the rainfall event having a return frequency of 100 years, unless higher frequencies are required by the Department of Natural Resources or the Jurisdiction. Design storm duration is that critical duration of rainfall requiring the greatest detention volume, or, based on the nature of the watershed, the critical duration would be the storm that causes the greatest downstream impact.

3. Release Requirements:

a. Release Rate: In an effort to mimic the pre-developed hydrology of a drainage area, maximum post-development release rates have been established based upon pre-developed conditions. These restrictions aid in the reduction of down-stream flooding and reduce the cost of downstream storm conveyance infrastructure.

1) **General:** The major storm drainage system should be designed to reduce the risk of substantial damage to the primary structure from storm runoff expected from the major storm. The effects of the major storm on the minor drainage system should be noted.
   2) **2 Year Pre-developed:** After development, the release rate of runoff for rainfall events having an expected return frequency of two years should not exceed the existing, pre-developed peak runoff rate from that same storm.
   3) **5 Year Pre-developed:** For rainfall events having an expected return frequency of 5, 10, 25, 50, and 100 years, the rate of runoff from the developed site should not exceed the existing, pre-developed peak runoff from a 5 year frequency storm of the same duration. Allowable discharge rate may be restricted due to downstream capacity.
   4) **Upstream Pass-through:** Detention of runoff generated by upstream land is not required on the new development site. Release of runoff generated off-site and routed through the detention basin should not be made in such a manner as to increase the combined off-site and on-site release rate.
   5) **Staged Discharge:** Because the allowable release rate varies depending on the storm frequency, multiple outlets or a multi-stage control structure may be necessary to comply with these requirements. This is especially true for sites with off-site pass-through as demonstrated in the following example.

b. Release Rate Example:

1) A 10 acre site has a critical storm duration of 6 hours after development.
2) The peak rate of runoff generated by the site for the pre-developed 2 year, 6 hour storm is 8.5 cfs.
3) The peak rate of runoff generated by the site for the pre-developed 5 year, 6 hour storm is 12 cfs.
4) The site receives off-site runoff from a 5 acre upstream area. The off-site area has the following runoff properties:

Allowable Runoff, cfs	**Return Period**					
	2 yr	***5 yr***	***10 yr***	***25 yr***	***50 yr***	***100 yr***
Offsite runoff	4.25	6	7	8.5	9.5	11

5) Taking into consideration the offsite contributing area, the maximum release rate for a given storm event is summarized in the following table:

Allowable Runoff, cfs	**Return Period**					
	2 yr	***5 yr***	***10 yr***	***25 yr***	***50 yr***	***100 yr***
Release for on-site runoff	8.5	12	12	12	12	12
Off-site "pass through"	4.25	6	7	8.5	9.5	11
Allowable release rate	12.75	18	19	20.5	21.5	23

4. Detention Volume Methods:

a. Two methods for watershed routing are allowed. The modified rational method may be used for areas up to 5 acres. For larger areas, the Storage Indication or modified Puls method should be utilized. This is the method utilized by WinTR-55 and other hydrology software. These methods are described in the following sections.

The use of other technically proven methods for similar drainage areas needs approval by the Jurisdictional Engineer. For larger drainage areas, the Project Engineer should understand the details of a computerized hydrology program before selection of the program.

b. The Project Engineer will submit the stormwater detention proposal according to the drainage report as described in Section 2A-4. Also required is certification by a licensed Professional Engineer that the stormwater detention facilities design and calculations were performed by the engineer, or under the engineer's supervision, and that the facilities and design meet the criteria of this section.

C. Limitation of Stormwater Runoff

1. No development should cause downstream property owners, water courses, channels, or conduits to receive stormwater runoff from the proposed development site at a higher peak flow rate, or at higher velocities than would have resulted from the same storm event occurring over the site of the proposed development with the land in its natural, pre-developed condition.

2. The Project Engineer can submit to the Jurisdictional Engineer the following factors for consideration in changing storm detention requirements as a condition for approval of development:

   a. Specific elements of the drainage report as outlined in Section 2A-4 and items listed in Section 2G-1, A, 1.

   b. Historical or potential localized drainage or flood problems adjacent to the site.

   c. Historical or potential area wide drainage or flooding problems in the watershed.

   d. Location of the site relative to existing drainageways and/or stormwater conveyances.

   e. Extent of proposed site increase in impervious surface area.

   f. Anticipated future development of the drainage basin.

   g. Existing site features which may facilitate or impede detention design and/or construction.

3. Multiple and contiguous tracts of land of which only part will be initially developed but are contained in the same basin are described below under two conditions:

   a. One Owner: The basin will be considered for stormwater detention for the entire tract. The results of the study, including staged construction of stormwater facilities, will be contained in the drainage report as outlined in Section 2A-4. As a minimum, the developed tract will require detention.

   b. Multiple Owners: Many times, upstream undeveloped discharges occur through the proposed developed property, which cannot be avoided. Possible options for stormwater detention design in a basin with tracts having multiple owners are:

      1) Isolation Detention:

         a) Isolate the proposed development portion from the rest of the basin. Construct a detention control structure on the downstream side of a developed area and outside of a mainline channel where there is no pass-through from upstream undeveloped property. This allows the detention basin to serve only the developed area.

         b) Isolate the stormwater to be bypassed from the developed area by a split-flow structure upstream of the proposed detention basin.

2) **Main Channel Detention:** Care should be exercised in not placing a control structure in a mainline channel unless it is designed for development to occur in a progressive manner. The designer needs to simulate the detention and corresponding release rate for only the developed area. A control structure that handles both flows (to be detained and pass through) has to be designed to retain the difference between the pre-developed and post-developed runoff rate from the developed area only and bypass the remaining upstream discharge. This can result in a complicated outlet control structure and routing system that has to split the flows within the detention basin.
3) **Regional Detention:** Develop a regional detention system within the watershed that handles logical segments of the watershed or the entire watershed.

D. Detention Basin Design Methods

A detention basin is to be designed to reduce the peak inflow by temporarily storing the excess stormwater and then releasing the water volume at allowable rates over an extended period. The main objective of this section is to outline the design procedure in order to determine the detention basin storage volume required. The design of a stormwater detention basin requires both hydrologic and hydraulic information. The basic hydrologic data includes the inflow hydrograph and the allowable release. In order to determine the volume required, the inflow hydrograph needs to be developed first. The hydraulic information of a basin requires prior knowledge of the basin geometry and outlet structures. Two common methods for determining the detention basin size are the Modified Rational Method and the TR-55 Method.

1. Modified Rational Method:

a. Theory: The simplest but least accurate detention routing method is the Modified Rational Method. The Modified Rational method uses the peak flow calculating capability of the Rational method, paired with assumptions about the inflow and outflow hydrographs to compute and approximation of storage volumes for simple detention calculations.

To find the required volume, the Modified Rational Method uses a trial method to find the critical storage for a given drainage area. The basic approach assumes the stormwater runoff hydrograph (detention basin inflow hydrograph) for the design storm is trapezoidal in shape. The peak runoff rate is calculated using the Rational formula:

$$q_{pi} = CiA$$

Equation 2G-1.01

where:

q_{pi} = peak runoff from site (peak inflow into detention basin)
C = runoff coefficient
i = rainfall intensity, in/hr
A = drainage area, ac

Note: Refer to Section 2B-4 for additional information on the use of the Rational method.

It is assumed the peak of the outflow hydrograph falls on the recession limb of the inflow hydrograph and the rising limb of the outflow hydrograph can be approximated by a straight line. The storage volume is determined by the critical (inflow) duration, and using a constant outfall release rate. With these assumptions:

$$S_d = q_{pi}t_d - \frac{Q_a(t_d + T_c)}{2}$$ **Equation 2G-1.02**

where:

S_d = detention volume required, ft^3
Q_a = allowable peak outflow rate, cfs
t_d = design storm duration, sec
T_c = time of concentration for the watershed, sec

The design storm duration is the duration that maximizes the detention storage volume, S_d, for a given return period. The storm duration can be found by trial and error using rainfall data from Section 2B-2. This is normally an iterative process done by hand or with a spreadsheet. Downstream analysis is not possible with this method, as only approximate graphical routing takes place.

b. Limitations: Use of the Modified Rational method has limitations. This method makes several assumptions including a constant rainfall over the watershed and a maximum release rate that is constant over the storm duration. Because of these assumptions the Modified Rational method does not produce a true inflow or outflow hydrograph, merely approximations of such. In addition, the Modified Rational method cannot easily account for off-site pass through from upstream drainage areas. For these reasons, the use of the Modified Rational method is limited to sites of 5 acres or less with no off-site pass through.

c. Design Example: Development of a 4.0 acre undeveloped site into an industrial complex is proposed. A detention basin will be used to limit the post-development peak discharge to the Q_5 pre-developed rate. The inflow hydrographs are developed using varying durations multiplied by the discharges for each Q_{100}. The outflow hydrograph for each duration, multiplied by the constant Q_5, is subtracted from the inflow hydrograph. The highest remaining storage volume is selected as the final basin volume.

There are three steps in the Modified Rational Method as follows:

1) **Step 1:** The first step is to collect the physical data for the drainage area. This is the drainage area, the time of concentration, the runoff coefficient, pre-developed peak discharge, etc.
   - Existing 4.0 acre undeveloped site
   - Soil Group D
   - C = 0.22 for Q_5 pre-developed condition
   - C = 0.9 for post-developed (industrial)
   - T_c = 15 min.
   - Q_a = 10.0 cfs (pre-developed Q_5 = 0.22x3.8x4.0 = 3.3 cfs)
2) **Step 2:** The second step is to establish the peak runoff rate from the developed site for various intensity-duration relationships at the design frequency (Q_{100}), beginning with the time of concentration and continuing with other increased storm durations.

Table 2G-1.01: Peak Basin Inflow for Various Durations

Duration (hour)	**C_{100}**	**Intensity (inches/hour)**	**Area (acres)**	**Inflow, q_{pi} (cfs)**
0.25	0.9	7.48	4.0	26.9
0.50	0.9	5.12	4.0	18.4
1.00	0.9	3.25	4.0	11.7
2.00	0.9	2.01	4.0	7.2
3.00	0.9	1.48	4.0	5.3
6.00	0.9	0.87	4.0	3.1

3) **Step 3:** The third step is to calculate the release volume and required storage until the maximum or critical storage is found. The allowable release rate for this detention basin needs to remain below 10 cfs as determined in Step 1 above. Table 2G-1.02 below outlines the process of calculating the required storage for each storm duration.

Table 2G-1.02: Storage Duration Values

(1) **Duration (hour)**	(2) **Q_{100} Intensity (inches/hour)**	(3) **Q_{100} Inflow (cfs)**	(4) **Q_{100} Volume (cubic feet)**	(5) **Release Vol. Q_5 (cubic feet)**	(6) **Storage (cubic feet)**
0.25	7.48	26.9	24,200	3,000	21,200
0.50	5.12	18.4	33,100	5,900	27,200
1.00	3.25	11.7	42,100	11,900	30,200
2.00	2.01	7.2	51,800	23,800	28,000
3.00	1.48	5.3	57,200	35,600	21,600
6.00	0.87	3.1	67,000	71,300	0

Column (3) Peak Flow = Q = CIA (take from Table 2G-1.01 above)
Example: 0.9 x 7.48 x 4.0 = 26.9 cfs
Column (4) Runoff Volume = Q (Col 3) x Duration of Storm (Col 1) x 3600
Example: 26.9 cfs x 0.25 hrs x 3600 s/hr = 24,200 cu. ft.
Column (5) Release Volume = 3.3 cfs x Duration of Storm (Col 1) x 3600
Example: 3.3 x 0.25 x 3600 s/hr = 3,000 cu. ft.
Column (6) Required Storage = Runoff Volume (Col 4) – Release Volume (Col 5)
Example: 24,200 – 3,000 = 21,200 cu. ft.

As Table 2G-1.02 shows, the critical duration is one hour, since it produces the largest detention volume of 30,200 cubic feet. Therefore, the detention basin needs to be designed to accommodate the 30,200 cubic feet of storage with at least a 1 foot freeboard for the detention dike. The basin emergency spillway release rate should be determined based on the onsite discharge greater than the 100 year post-developed peak discharge.

A second analysis must still be completed for the 2 year pre/post developed condition. When storage volumes are known for the 2 year and 100 year storms, a suitable outlet control structure can be designed.

2. **Flood Routing:** The most commonly used method for calculating detention basin volume is to route an inflow hydrograph through a detention pond utilizing the Storage Indication or modified Puls method. This method compares the difference in the average values of two closely spaced inflows and outflows, yielding the change in storage over a given time period. By continuing this process for the duration of the storm and beyond, the total required storage for the basin can be determined.

This is the methodology utilized by WinTR-55 and other hydrology software and can also be completed through the use of a spreadsheet. A detailed description of the manual process for routing a storm through a detention basin is presented in Chapter 8 of FHWA's HEC-22.

E. Estimating Storage Volume

TR-55 indicates that the method presented should not be used for final design. The final design should be verified by routing the inflow hydrograph and determining if the proposed volume is adequate

The volume of the basin is determined by developing a hydrograph and routing the design storm through the basin. If the design storm can be routed through the basin without overtopping or exceeding the freeboard requirements, the basin volume is adequate. If the routing procedure indicates the storage elevation of the basin exceeds the freeboard requirements or overtops the basin, additional volume in the basin is required.

The final design of a detention facility requires three items:

- an inflow hydrograph
- a stage vs. storage curve
- a stage vs. discharge curve

1. To check the capacity of a basin with a known volume, use the methods described in the previous sections.

    a. Develop an inflow hydrograph for the storm in question.

    b. Develop the stage-storage and stage-discharge curves for the basin.

    c. Route the storm through the basin to determine the outflow hydrograph. Check the peak of the outflow hydrograph to ensure that it does not exceed the allowable value. Also, check the peak storage volume to ensure that it does not exceed the capacity of the basin.

2. Analyzing a known basin utilizing the methods developed in the previous sections is relatively straightforward. However, determining the required size of a proposed basin is an iterative process, and can be quite time consuming without a method to develop a preliminary volume estimate. Fortunately, TR-55 provides a method for determining quick estimates of detention basin volumes.

    a. Figure 2G-1.01 relates two ratios: peak outflow to peak inflow (q_o/q_i) and storage volume to runoff volume (V_s/V_r). The value for q_i is determined by the peak of the inflow hydrograph. The value for q_o is normally dictated by the allowable release rate. The volume of runoff can be calculated by the Rational method or tabular hydrograph method.

       The relationships in Figure 2G-1.01 were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data.

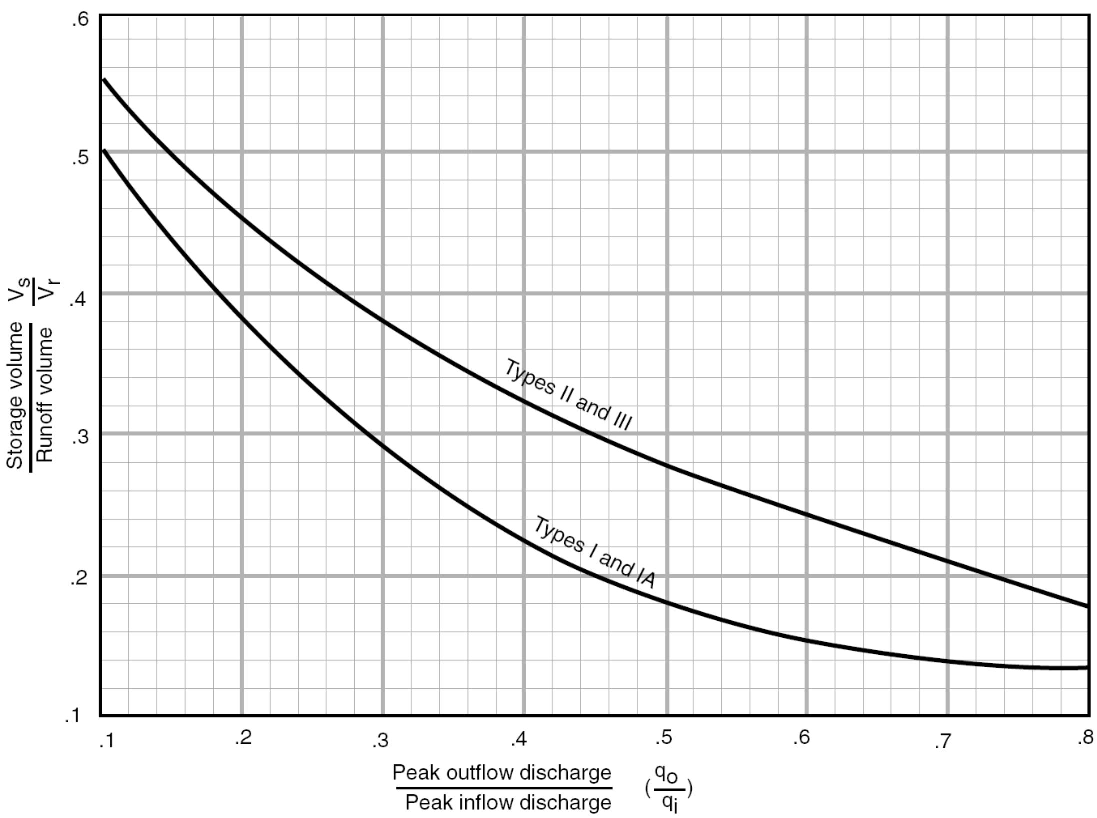
    b. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that:
       1) Each stage requires a design storm and a computation of the storage required for it.
       2) The discharge of the upper stage(s) includes the discharge of the lower stage(s).

c. The brevity of the procedure allows the designer to examine many combinations of detention basins. When combined with the Tabular Hydrograph Method, the procedure's usefulness is increased. Its principal use is to develop preliminary indications of storage adequacy.

This estimating technique becomes less accurate as the q_o/q_I ratio approaches the limits shown in Figure 2G-1.01. The curves in Figure 2G-1.01 depend on the relationship among available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When the storage volume (V_s) required is small, the shape of the outflow hydrograph is sensitive to the rate of the inflow hydrograph. Conversely, when V_s is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge (q_o) approaches the peak inflow (q_i), the parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant.

The procedure should not be used to perform final design if an error in storage of 25% cannot be tolerated. Figure 2G-1.01 is biased to prevent undersizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

Figure 2G-1.01: Approximate Detention Basin Routing for All Rainfall Types



Source: FHWA, HEC-22

d. The purpose of Figure 2G-1.01 is to provide a starting point for the size of the basin. The process may have to be repeated several times to achieve a basin that has sufficient volume and meets specific inlet and outlet controls.

F. Detention Facilities Requirements

1. Earthen Detention:

a. Slopes on embankments should be at least 4:1 or flatter and should have appropriate temporary and permanent erosion control stabilization.

b. Detention bottom cross-slopes to the main detention swale or channel will be 2% minimum. Concrete paved swale or channel bottom (cunette) and subsurface drains is required for slopes less than 1.5%. The Jurisdictional Engineer may require a pilot channel in the detention basin bottom.

c. The embankment top should be at least 6 feet wide.

d. Freeboard should be a minimum of 1 foot above the controlled emergency spillway discharge. If there is not room for an emergency spillway, the minimum freeboard above the 100 year surface elevation of the structure should be increased to 2 feet.

e. The embankment should be protected from catastrophic failure due to overtopping following Iowa DNR requirements where applicable. Overtopping can occur when the pond outlets become obstructed or when a larger than 100 year storm occurs. Failure protection for the embankment may be provided in the form of a buried, heavy rip rap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum developed inflow rate for the 100 year storm. The spillway is also needed to control the release point of the overflows. Structures should not be permitted in the path of the emergency spillway or overflow, and easements should be considered. The flowline of the emergency spillway should be set equal to or above the 100 year water surface elevation. Stormwater easements need to be considered downstream of the emergency spillway.

2. Parking Lot Storage:

a. Paved parking lots may be designed to provide temporary detention storage of stormwater on a portion of their surfaces not to exceed 25%.

b. Outlets should be designed to empty the stored waters slowly, and depths of storage must be limited to 9 inches so as to prevent damage to parked vehicles. The minimum pipe size for the outlet is 12 inches in diameter where a drop inlet is used to discharge to a storm sewer or drainageway.

   Where a weir and a small diameter outlet through a curb are used, the size and shape are dependent on the discharge/storage requirements. A minimum pipe size of 6 inches in diameter is recommended.

c. To assure that the detention facility performs as designed, maintenance access should be provided. The outlet should be designed to minimize unauthorized modifications that affect function. Any repaving of the parking lot will be evaluated for impact on volume and release rates and are subject to approval.

d. Storage areas should be posted with warning signs.

3. **Multipurpose Basins:** Dry bottom basins may be designed to serve secondary purposes for recreation, open space, or other types of use which will not be adversely affected by occasional or intermittent flooding.

4. **Maintenance:** The owner of the detention basin may be the developer, homeowner, homeowner's association, or Jurisdiction. The method of ownership and maintenance responsibility of the detention basin including easements, should be defined in the Jurisdiction's ordinance or in a developer's agreement with the Jurisdiction.

   Maintenance of the detention area must be performed on a regular basis to ensure the basin will operate as designed when needed. Maintenance should include:
   - Mowing to control trees and weeds. No trees should be permitted in the impoundment dam.
   - Checking for the integrity of the dam, including repair of varmint holes, and low places in the dam other than the emergency spillway.
   - Ensuring the emergency spillway is operating properly and at the proper elevation.
   - Ensuring all valves and gates are exercised regularly and in operating order.
   - Inspecting outlet orifices to ensure proper operation, including the proper operation of any orifice plates.
   - Ensuring the inlet to the basin allows proper flow to the detention area.
   - Ensuring inlet, outlet, and emergency spillways are free from obstructions.
   - Inspecting any related signs are in place and legible.
   - Inspect fence, if any, for continuity.
   - Inspect erosion control to ensure it is adequate.

G. References

Flood Plain Development. Iowa Administrative Code. Title V. Chapter 70.

U.S. Department of Transportation. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22. Third Ed. 2009.

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CHAPTER 3
Sanitary Sewers

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General Information

A. Concept

Sanitary sewer systems are essential to the public health and welfare in areas of concentrated population and development. Every community produces water-borne wastes of domestic, commercial, and industrial origin. The sanitary sewer performs the needed function of collecting these wastes and conveying them to points of approved discharge or disposal. The use of uniform and adequate sanitary sewer design criteria is essential for public safety and proper wastewater treatment, maintenance and control.

It is important to collect the needed information for the proper design of the sanitary sewer system. Generally, this information includes:

1. Topography, surface and subsurface conditions, above and below ground utilities, soil characteristics, water table elevations, and traffic control needs.

2. Locations of streets, alleys, and easements.

3. Capacity, condition, and elevations of the existing sanitary sewer to which the proposed pipe will connect. Determination of backups or unusual maintenance problems on the connecting sewer.

4. Information relative to any proposed expansion of the proposed project by annexation or service agreement.

5. Locations of historical and archaeological sites and any environmental sensitive areas within the project area.

6. Access needs for construction and operation of the sanitary sewer.

7. Quantity of flow in the pipe to be extended.

The Iowa DNR requires underground storage tank (UST) owners to meet specific design requirements for USTs installed within 1,000 feet of a community water system due to the potential for volatile organic compounds to be released from the UST and impact water main pipe. The same potential exists for sanitary sewer pipe, especially force mains that are installed at a shallower depth. The Project Engineer should determine if there is an UST in the area of the force main project. If so, the Designer should determine the need to design the force main to prevent future permeation of any volatile organic compounds into the sanitary sewer system. There are various elements to consider, some of which include soil types, groundwater table depth, size of the UST, age of the UST, etc.

B. Conditions

1. **Design:** The design for sanitary facilities should be in conformance with the following:

   a. "Iowa Standards for Sewer System, Chapter 12," Iowa Department of Natural Resources.

   b. "Recommended Standards for Sewage Works Great Lakes-Upper Mississippi River Board of State Sanitary Engineers." (Ten State Standards).

   c. Jurisdiction's Plumbing Code.

   d. In case of a conflict between the above design standards, the most restrictive requirement applies.

2. **Construction Standards:** Construction standards should be the most recent edition of the SUDAS Standard Specifications. All details, materials, and sewer appurtenances should conform to these standards.

3. **Project Submittals:** An application for a permit to construct should follow the Department of Natural Resources Rules and Regulations. A construction permit issued by the Iowa Department of Natural Resources (Iowa DNR) is required for the construction, installation or modification of any disposal system or part thereof or any extension or addition thereto. A permit to construct sewer extensions may be obtained from a local public works department when the department's permitting authority has been delegated to the local public works department under section 455B.183 of the Code of Iowa.

   DNR construction permits are normally not required for the following sewers:

   a. Storm sewers that transport only surface water runoff.

   b. Any new disposal system or extension or addition to any existing disposal system that receives only domestic or sanitary sewage from a building or housing occupied by fifteen persons or less.

   c. Replacement of previously approved construction where the replacement is done with the exact same methods, materials, capacities, and design considerations. However, if there is any change, the proposed construction will require a construction permit.

   d. Sanitary sewer service connections, defined as any connection from a single property unit to an existing sanitary sewer.

   Engineering services to obtain a construction permit and complete the approved construction should be performed in three stages:

   a. Engineering report or facilities plan (not required for minor sewer extensions).

   b. Preparation and submittal of construction plans, specifications, and contractual documents.

   c. Preparation and submittal of permit forms, including a sewage treatment agreement from the agency providing treatment.

   d. Construction inspection, administration, compliance, and acceptance.

All reports, plans, and specifications should be prepared in conformance with Chapter 542B of the Code of Iowa.

Engineering reports, permit forms, or facilities plans should be submitted to the Iowa DNR at least 120 days prior to the date for starting construction, upon which action by the Department is desired, or according to the Iowa Operation Permit or other schedules. If the project meets the requirements of Iowa Code Section 455 B. 183 for a minor sewer extension, and the county or city public works department has been approved to issue permits, the information should be submitted to the local officials for processing.

The final plans and specifications should not be prepared until the engineering report has been approved. This enables the Department to review the concept and design basis, make appropriate comments, and indicate to the applicant the general acceptability of the proposal before additional expenses are incurred for developing final plans and specifications. After the engineering report has been approved, the final plans and specifications should be submitted in accordance with 400--24.2(455B) of the Iowa Administrative Code or in accordance with the Iowa Operation Permit or other schedules. These plans and specifications should be prepared in accordance with the approved engineering report or facilities plan. Any changes from the approved report must receive prior approval from the Iowa DNR before incorporation into the plans and specifications.

Flow Determination

A. Sanitary Sewers Design Period

The length of time used in forecasting flows and setting capacities of the sanitary sewer is called the design period. The design period is related to the planning horizon for development of the project area and the expected life of the sanitary sewer pipe. In some cases, no specific planning horizon is identified. Instead the build-out population or land use is used. This is the maximum population and/or commercial and industrial development that could occur within the project area and beyond, if appropriate. The flows are determined based on that population or land use development without regard to time frames.

For residential development, the flows can be predicted using the following densities:

1. **Discharge (Q) Average Daily Flow (minimum):**

   Area x Area Density x Flow Rate = Average Daily Flow **Equation 3B-1.01**

   Number of Units x Unit Density x Flow Rate = Average Daily Flow **Equation 3B-1.02**

2. **Discharge (Q) Peak Sewer Flow (minimum):** Average daily flow times ratio of peak to average daily flow (See Figure 3B-1.01 for ratio). NOTE: Population values shown in Figure 3B-1.01 are based on the area that discharges into the sewer.

3. **Design Density and Rate:** See Table 3B-1.01.

B. Footing Drain Inflow

If a proposed sewer is to serve an older developed area with existing footing drain inflow, special design information should be obtained from the Jurisdiction. Additional extraneous flow allowances may be warranted where high groundwater levels, significant inflow sources, or higher than average per capita wastewater flow rates are expected to occur over the design life of the sewer.

C. Area

Gross area should be used in determining design flows and include streets, alleys, school grounds, parks, and similar dedicated open space.

D. Density Table

Table 3B-1.01: Minimum Values

Land Use	Area Density	Unit Density	Rate
Low Density (Single Family) Residential	10 people / AC	3 people / unit	100 gpcd*
Medium Density (Multi-Family) Residential	15 people / AC 6.0 people / duplex	3 people / unit	100 gpcd*
High Density (Multi-Family) Residential	30 people / AC	2.5 people / unit	100 gpcd*
Office and Institutional	5,000 gpd / AC (IDNR)	Special Design Density	N/A
Commercial and Light Industrial	5,000 gpd/AC (IDNR)	Special Design Density	N/A
Industrial	10,000 gpd/AC (IDNR)	Special Design Density	N/A

* Iowa Department of Natural Resources (DNR) - Dry Weather Flow - One hundred gallons per capita per day (gpcd) should be used in design calculations as the minimum average dry weather flow. This 100 gpcd value may, with adequate justification, include maximum allowable infiltration for proposed sewer lines.

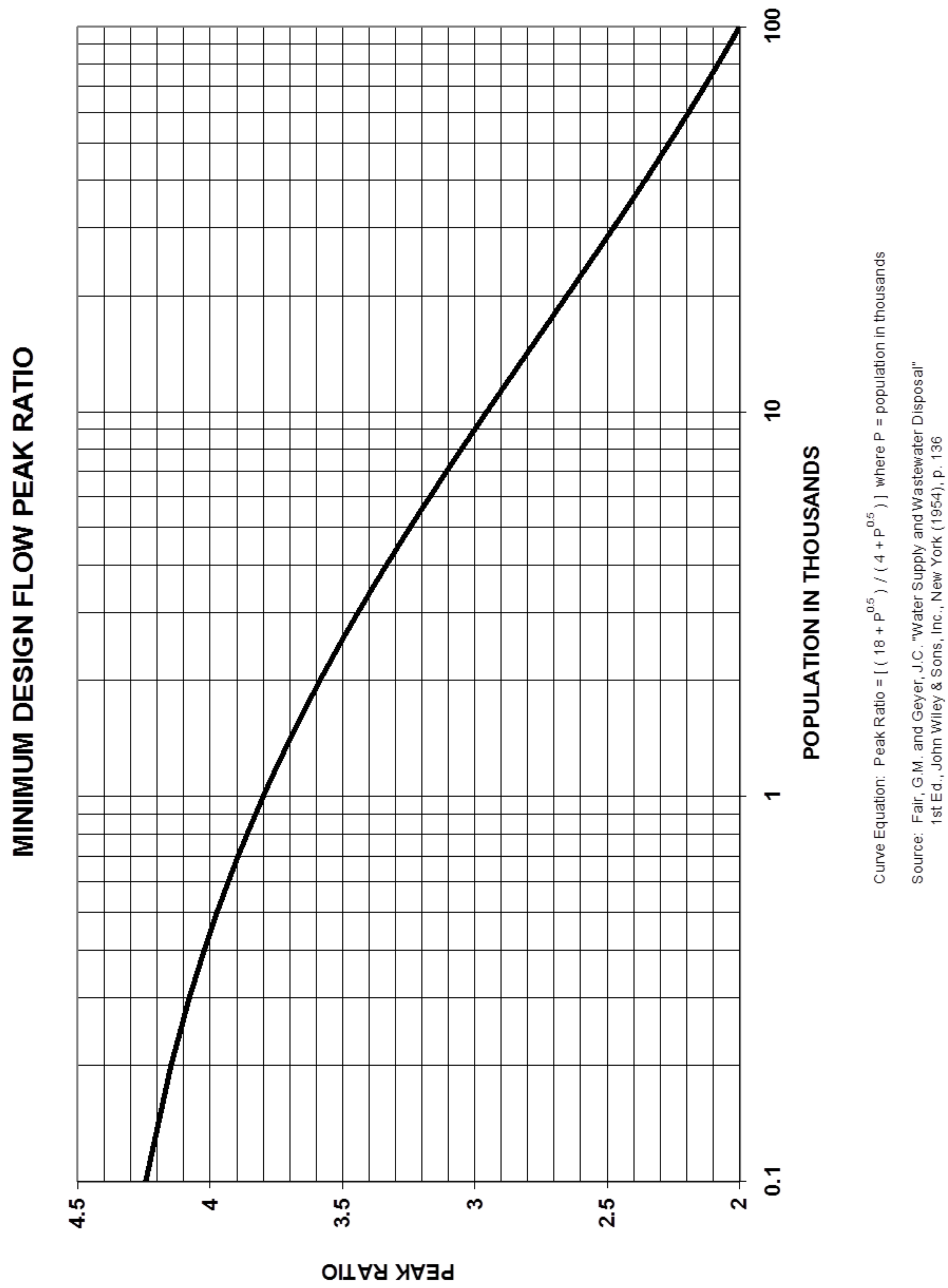
The area densities listed include the peaking factor.

Note: If the Project Engineer uses values different from the above table, approval by the Jurisdictional Engineer is required.

E. Special Design Densities

Special design densities should be based on specific flow measurements or known flow rates and are subject to approval by the Jurisdiction Engineer based on methodology provided by the Project Engineer prior to submittal to the Iowa DNR.

Figure 3B-1.01: Ratio of Peak to Average Daily Sewage Flow



Facility Design

A. Capacity of Pipe

Pipe sizes 15 inches and smaller should carry the peak flow at a depth of no more than 0.67 of the pipe diameter. Pipe sizes greater than 15 inches should carry the peak flow at a depth of no more than 0.75 of the pipe diameter. See Figure 3C-1.01 to determine full flow values. To calculate 0.67 full and 0.75 full, multiply the full flow values from Figure 3C-1.01 by 0.79 and 0.91 respectively. Iowa DNR uses 0.75 of the pipe diameter for pipes 8 inches to 15 inches with no mention of larger pipes.

B. Flow Within the Pipe

The accepted approach to achieving adequate capacity and self-cleansing design is to assume one-dimensional, incompressible, steady, uniform flow. Since only atmospheric pressure generally exists at the surface of the flow, it can be considered open channel flow. Manning's equation is the most widely used and is one of the best open channel flow equations. Figure 3C-1.01 provides solutions to Manning's equation.

The minimum self-cleaning velocity is 2 feet per second with the pipe flowing full. Recent research indicates that in smaller diameter pipes (less than 18 inches) flowing less than 20% full, the self-cleansing velocities are not achieved. The same is true in larger diameters flowing less than 30% full. Specific care should be taken on any portions of the pipe network that will experience these low flows for significant lengths of time due to dead ends or slow development activity.

The maximum velocity flowing full is 15 feet per second. Special design mechanisms may be necessary to address displacement of solids and impact of flow downstream.

C. Pipe Material

Contact the Jurisdictional Engineer for materials allowed by each jurisdiction.

D. Manning's Roughness Coefficient

The roughness coefficient to be used is $n = 0.013$. This coefficient is for all types of approved pipe materials.

E. Minimum Grade

See Table 3C-1.01 below for the minimum slopes for each pipe diameter. Minimum grade on sanitary sewer service stubs should be 1/8 inch per foot.

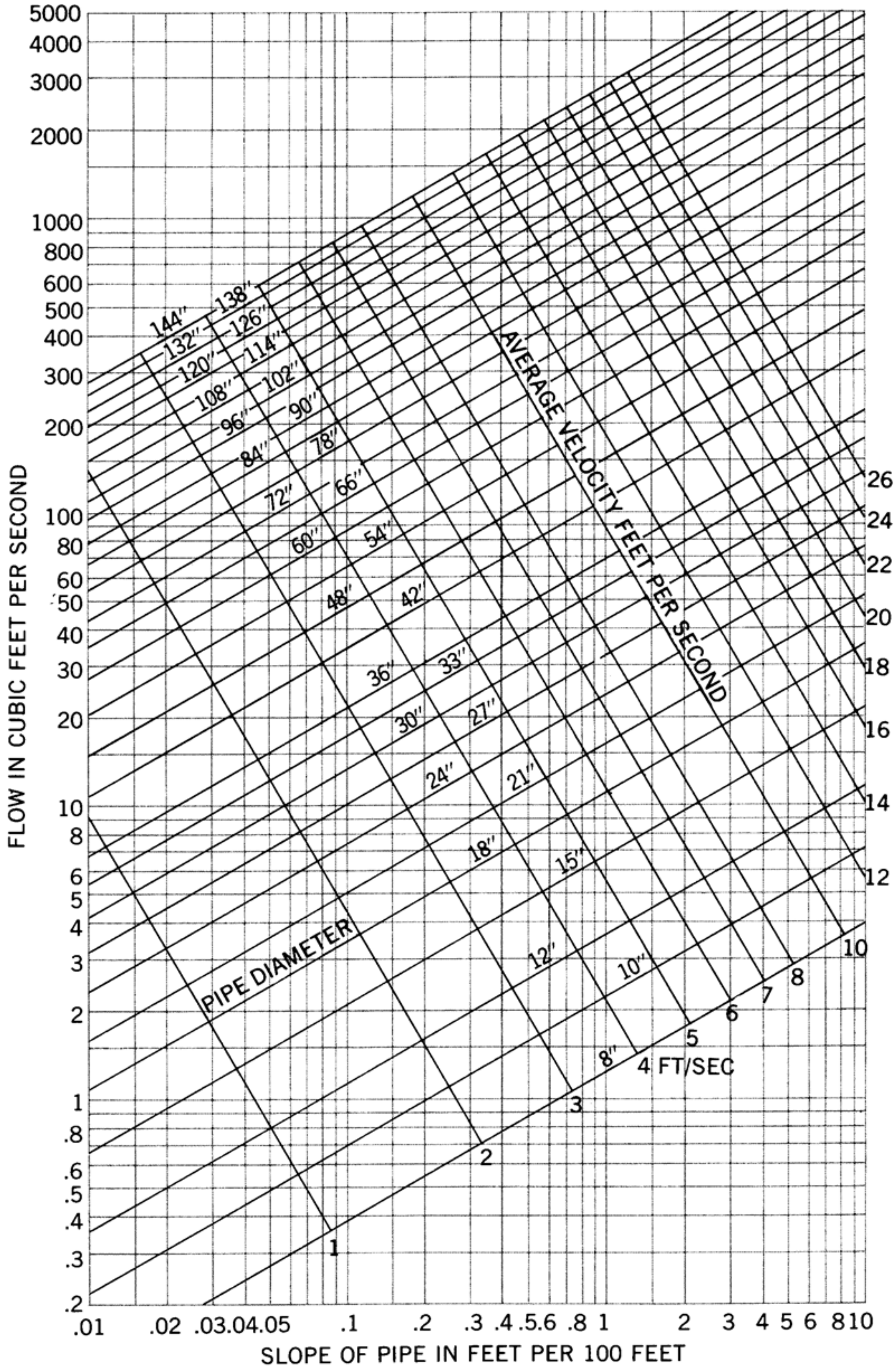
Table 3C-1.01: Minimum Slope

Pipe Size (inches)	Minimum Slope (ft/100 ft)
8	0.40
10	0.28
12	0.22
15	0.15
18	0.12
21	0.10
24	0.08
27	0.067
30	0.058
36	0.046

F. Size of Sewer Pipe

Gravity public sanitary sewers should not be less than 8 inches in diameter. Minimum size of building sanitary sewer stub should be 4 inches in diameter for residential and 6 inches in diameter for commercial. The size will increase based on the proposed number of fixtures that the sewer stub serves.

Figure 3C-1.01: Flow for Circular Pipe Flowing Full (Based on Manning's Equation n=0.013)



G. Crossings and Clearances

1. **Storm Sewers:** Sanitary sewer crossings of storm sewers should have no less than 6 inches of clearance. Special structural support will be required if there is less than 18 inches clearance. The minimum horizontal clearance should be 5 feet. Clearance refers to the distance from the outside of the sanitary sewer pipe to the outside of the storm sewer pipe.

2. **Protection of Water Supplies:** (from Iowa DNR's Iowa Wastewater Facilities Design Standards, Chapter 12, Section 12.5.8)

   a. **Wells:** Sewers constructed of standard sewer materials shall not be laid within 75 feet of a public well or 50 feet of a private well. Sewers constructed of water main materials may be laid within 75 feet of a public well and within 50 feet of a private well but no closer than 25 feet to either.

   b. **Horizontal Separation of Gravity Sewers from Water Mains:** Gravity sewer mains shall be separated from water mains by a horizontal distance of at least 10 feet unless:
      1) the top of a sewer main is at least 18 inches below the bottom of the water main, and
      2) the sewer is placed in a separate trench or in the same trench on a bench of undisturbed earth at a minimum horizontal separation of 3 feet from the water main.

      When it is impossible to obtain the required horizontal clearance of three feet and a vertical clearance of 18 inches between sewers and water mains, the sewers must be constructed of water main materials meeting both a minimum pressure rating of 150 psi and the requirements of Sections 8.2 and 8.4 of the "Iowa Standards for Water Supply Distribution Systems" (SUDAS Specifications Section 5010, 2.01). However, a linear separation of at least 2 feet shall be provided.

   c. **Separation of Sewer Force Mains from Water Mains:** Sewer force mains and water mains shall be separated by a horizontal distance of at least 10 feet unless:
      1) the force main is constructed of water main materials meeting a minimum pressure rating of 150 psi and the requirements of Section 8.2 and 8.4 of the "Iowa Standards for Water Supply Distribution Systems" (SUDAS Specifications Section 5010, 2.01) and
      2) the sewer force main is laid at least 4 linear feet from the water main.

   d. **Separation of Sewer and Water Main Crossovers:** Vertical separation of sanitary sewers crossing under any water main should be at least 18 inches when measured from the top of the sewer to the bottom of the water main. If physical conditions prohibit the separation, the sewer may be placed not closer than 6 inches below a water main or 18 inches above a water main. The separation distance shall be the maximum feasible in all cases.

      When the sewer crosses over or is less than 18 inches below a water main one full length of sewer pipe of water main material shall be located so both joints are as far as possible from the water main. The sewer and water pipes must be adequately supported and have watertight joints. A low permeability soil shall be used for backfill material within 10 feet of the point of crossing.

   e. **Exceptions:** Should physical conditions exist such that exceptions to b through d above are necessary, the design engineer must detail how the sewer and water main are to be engineered to provide protection equal to that required by these sections.

3. **Sewer Crossing Under a Waterway:** (from Iowa DNR's Iowa Wastewater Facilities Design Standards, Chapter 12, Section 12.5.11)

   The top of all sewers entering or crossing streams shall be at a depth below the natural bottom of the stream bed sufficient to protect the line. One foot of cover over the top of the line is required where the sewer is located in rock or cased and three feet of cover is required in other material. In major streams, more than the three feet of cover may be required.

   In paved channels, the top of the sewer line should be placed below the bottom of the channel pavement. Sewer outfalls, headwalls, manholes, gate boxes, or other structures shall be so located that they do not interfere with the free discharge of flood flows of the stream. Sewers located along streams shall be located outside of the stream bed.

   Sewers entering or crossing streams shall be constructed of cast or ductile pipe with mechanical joints or shall be so otherwise constructed that they will remain water tight and free from changes in alignment or grade. Sewer systems shall be designed to minimize the number of stream crossings. The stream crossings shall be designed to cross the stream as nearly perpendicular to the stream flow as possible. Construction methods that will minimize siltation shall be employed. Material used to backfill the trench shall be stone, coarse aggregate, washed gravel, or other materials which will not cause siltation. Upon completion of construction, the stream shall be returned as near as possible to its original condition. The stream banks shall be seeded and planted, or other methods employed to prevent erosion. The design engineer shall include in the project specifications the method or methods to be employed in the construction of sewers in or near streams to provide adequate control of siltation.

4. **Aerial Crossings:** (from Iowa DNR's Iowa Wastewater Facilities Design Standards, Chapter 12, Section 12.5.12)

   Support shall be provided at all joints in pipes utilized for aerial crossings. The supports shall be designed to prevent overturning and settlement.

   Precautions against freezing, such as insulation and increased slope, shall be provided. Expansion jointing shall be provided between above-ground and below-ground sewers.

   For aerial stream crossings the impact of flood waters and debris shall be considered. The bottom of the pipe should be placed no lower than the elevation of the 50-year flood.

5. **Drainage Courses:** Consideration should be given to providing additional depth below the streambed or erosion protection in the case of potentially erodable drainage courses.

H. Depth of Sewer

Gravity sewers should be deep enough to serve basements, assuming a 2% grade plus adequate allowance for pipe fittings on house sewers (absolute minimum of 1%). They should have a minimum depth to the top of pipe of 8 feet unless the sewer can serve existing basements at a lesser depth. For those structures with no basements or when a high ground water table is encountered, depths less than 8 feet may be allowed. In either case, the sewer should be well below the frost line at all points and lower than any water lines placed in the same street. Insulation should be provided for sewers that cannot be placed at a depth sufficient to prevent freezing. For sewers greater than 12 feet deep as measured at the building line, provide risers on service stubs. Maximum depth of sewer should not exceed depth recommended by the pipe manufacturer.

I. Location of Sanitary Sewers

1. Sanitary Sewers in Street Right-of-way:

a. Sanitary sewers parallel to the right of way may be placed in the center of the street or behind the back of curb. Contact Jurisdiction for allowable location.

b. Sanitary sewers perpendicular to the street should follow Iowa DNR clearance requirements between storm sewer, water mains, and other utilities.

2. Sanitary Sewers Outside of Street Right-of-way:

a. Sanitary sewers will be placed in a sanitary sewer public easement. Public sanitary sewer easements should have a minimum total width of 20 feet or two times the depth of the sewer, whichever is greater, with the sanitary sewer centered in the easement. Additional width may be required by the Jurisdictional Engineer to insure proper access for maintenance equipment.

b. Provisions must be made to provide public access to the sanitary sewer easements from public streets.

J. Alignment of Sewers

Sewers less than 24 inches in diameter should be straight between manholes. Curvilinear alignment may be allowed in sewers 24 inches and greater but must start and end at manholes. Minimum grades must be increased to provide average full flow velocities equivalent to sewers with straight alignment. Submittal of the curvilinear alignment design to the Iowa DNR is required.

K. Sewer Linings for Ductile Iron Pipe

If ductile iron pipe is used for sanitary sewer pipe material, the pipe must be lined for sulfate protection. Allowable linings include calcium aluminate cement, polyethylene, ceramic epoxy, or coal tar epoxy.

L. Manholes

1. **Access to Manholes:** Manholes in street right of way must be located in areas which allow direct access by maintenance vehicles. Areas outside the street right of way should be subject to the approval of the Jurisdictional Engineer.

2. **Standard Manhole:** The minimum size for a manhole is 48 inches in diameter. Most Jurisdictions require eccentric manholes with the manhole opening over the centerline of the pipe or on an offset not to exceed 12 inches. The remaining Jurisdictions allow for concentric manholes. Check with Jurisdictional Engineer regarding use of eccentric and concentric manholes and built-in steps.

3. **Special Manholes:** For square or rectangular manholes, the manhole openings should be over the centerline of the pipes or on an offset not to exceed 12 inches. The distance from the centerline of the manhole opening to the face of the inside manhole wall should not exceed 30 inches to better facilitate video inspection and maintenance equipment. This may require more than one manhole opening.

4. Manhole Locations:

a. Manholes should be installed:
   1) at the end of each sewer line
   2) at all changes in pipe size, grade or alignment, and at bends
   3) at all sewer pipe intersections
   4) at intervals not exceeding 400 feet for sewers 24 inches or less or at intervals not exceeding 500 feet when adequate cleaning equipment is available. Spacing of manholes over 500 feet may be permitted in sewers larger than 24 inches if the owner has adequate cleaning equipment.

b. Cleanouts may be substituted, with Jurisdictional approval, for mains shorter than 150 feet.

5. Minimum Manhole Drop:

a. Change in alignment - 0 to 45 degrees - none.

b. Change in alignment across manhole - greater than 45 degrees - 0.10 feet (minimum), 0.25 feet (preferred).

6. Dissimilar Pipe Sizes: Change in pipe size - match eight-tenths full points.

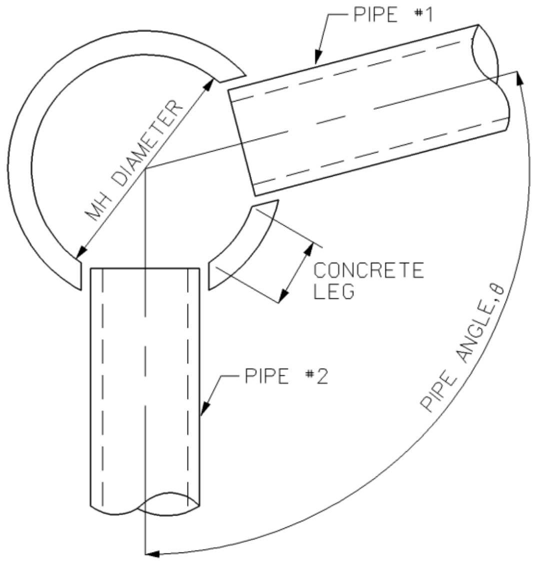
7. Maximum Manhole Drop: A drop connection is required when the invert to invert drop is greater than 2 feet, except when the eight-tenths points match exceeds 2 feet.

8. Manhole Frames and Covers: Bolt-down covers are required on manholes subject to inundation such as in flood plains, detention areas, and storm water easement areas subject to "major storms." Minimum access diameter of 27 inches is required.

9. Manhole Coatings: Exterior waterproof coating (bituminous) is not required unless specified by the Jurisdiction. Interior coatings should be required if sulfate protection is necessary. Drop sections should be coated along with the manhole to protect against sulfate.

10. Manhole Sizes: When utilizing circular precast manholes, it is necessary to determine the diameter required to maintain the structural integrity of the manhole. As a general rule, a minimum concrete leg of 6 inches should remain between the manhole blockouts for adjacent pipes. Determining the required manhole diameter to provide this minimum distance may be done as follows:

a. Determine the diameters of, and the angle between, the two pipes in question. If more than two pipes connect at the manhole, the adjacent pipes with the critical configuration (i.e. smallest angle and largest pipes) should be selected. If the critical configuration is not apparent, calculations may be required for all adjacent pipes.



b. Determine the blockout diameter. The blockout is the opening provided in the manhole for the pipe. Blockout dimensions are based on the outside diameter of the pipe, plus an additional distance to accommodate the integrally cast gasket for sanitary sewer pipe. For storm sewer, a circular or doghouse type opening is provided with additional clearance to allow for the insertion of the pipe and sufficient space to accommodate placement of concrete grout in the opening. Typical blockout dimensions for various pipe sizes and materials are given in Table 3C-1.02.

Table 3C-1.02: Manhole Blockout Sizes

Pipe Diameter (inches)	**Manhole Blockout (inches)**			
	RCP		***PVC***	***DIP***
	Sanitary (gasketed)	***Storm (non-gasketed)***		
8	N/A	N/A	12	12
10	N/A	N/A	14	14
12	24	21	16	16
14	N/A	N/A	16	18
15	26	24	19	N/A
16	N/A	N/A	N/A	20
18	30	28	22	23
20	N/A	N/A	N/A	24
21	35	31	25	N/A
24	38	35	28	29
27	42	38	31	N/A
30	44	42	35	36
33	47	47	N/A	N/A
36	52	48	42	41
42	59	57	N/A	N/A
48	66	64	N/A	N/A
54	72	71	N/A	N/A
60	79	78	N/A	N/A

c. Determine the diameter of the manhole required to provide the minimum concrete leg dimension. This diameter may be calculated with the following equation:

$$MH_d = \frac{BO_1 + BO_2 + 2CL}{\theta \times \left(\pi / 180\right)}$$ **Equation 3C-1.01**

Where:

MH_d = Manhole Diameter, inches
BO = Blockout Diameter, inches
CL = Minimum Concrete Leg Length, inches (6 inches)
θ = Angle between pipe centerlines, degrees

d. Round the minimum manhole diameter calculated, up to the next standard manhole size (48 inches, 60 inches, 72 inches, 84 inches, 96 inches, 108 inches, or 120 inches).

e. Verify that the manhole diameter calculated is sufficient for the largest pipe diameter (see Table 3C-1.03).

Table 3C-1.03: Minimum Manhole Diameter Required for Pipe Size

Pipe Diameter (inches)	**Minimum Manhole Diameter Required for Pipe (inches)**		
	RCP	***PVC***	***DIP***
8	N/A	48	48
10	N/A	48	48
12	48	48	48
14	N/A	N/A	48
15	48	48	N/A
16	N/A	N/A	48
18	48	48	48
20	N/A	N/A	48
21	48	48	N/A
24	48	48	48
27	60	48	N/A
30	60	60	60
33	60	N/A	N/A
36	60	60	60
42	72		
48	84		
54	96		
60	96		

M. Sewer Services

1. Each structure or complex under one ownership should be served by a separate service line connected to a public or private sanitary sewer. The service should be perpendicular to the sewer line where possible, with tee or wye connections to the public sewer.

2. Sewer services must meet all the Jurisdiction's requirements.

3. Unless individual onsite treatment systems are allowed, all platted lots of a proposed subdivision are to have separate sewer services for each owner and be adjacent to a public sanitary sewer main without crossing any adjacent properties. Additional sewer services will be required for each additional principal structure on a given lot.

4. Sewer services across one property to provide service to an adjacent property should be avoided. If a condition exists that requires crossing an adjacent property, the following should be met:

    a. A private utility easement is provided that is 10 feet wide (minimum) or two times the depth, whichever is greater.

    b. The Jurisdictional Engineer determines that a sewer main extension will not be necessary and in all likelihood no future development of abutting properties will benefit from a main extension.

5. Connect sewer services to sewer mains. Connections directly to manholes will require Jurisdiction’s approval. Individual single family residential services will not be connected to a manhole unless at terminal manholes which cannot possibly be extended in the future. The services may not enter the manhole at greater than 2 feet above the invert of the outlet. Sewer flow channels in the manhole bottom must be provided for all services. Commercial and multi-family sewer services can be connected, with Jurisdictional approval, to a manhole on the collector sewer if flows are large enough to keep the manhole clean.

N. Force Mains

1. **Minimum Velocity:** 2 fps at minimum pumping condition.

2. **Air Release Valves:** Should be located at high points to control the excess accumulation of sewage gases.

O. Siphons

In general, sanitary sewer siphons should be avoided and will only be accepted where no feasible alternative exists and where there will be sufficient flow in the sewer so that maintenance will be held to a minimum. All siphons should have a minimum of two barrels with a minimum pipe size of 6 inches diameter. Design provisions should be made for diversion of normal flow to either barrel for maintenance. Sufficient head should be provided to insure velocities of at least 3 feet/second for average flow.

Pipe and Manhole Materials

Table 3D-1.01: Sanitary Sewer Pipe Materials

Typical Application	Pipe Material	Size Range	Standard	Thickness Class (min.)	Pipe Stiffness (min.)	Joints
Gravity Flow	Solid Wall PVC	8” to 15”	ASTM D 3034	SDR 26	115 psi	Bell and Spigot
Gravity Flow	Solid Wall PVC	8” to 15”	ASTM D 3034	SDR 35	46 psi	Bell and Spigot
Gravity Flow	Solid Wall PVC	18” to 27”	ASTM F 679	N/A	46 psi	Bell and Spigot
Gravity Flow	Corrugated PVC	8” to 10”	ASTM F 949	N/A	115 psi	Bell and Spigot
Gravity Flow	Corrugated PVC	12” to 36”	ASTM F 949	N/A	46 psi	Bell and Spigot
Gravity Flow	Closed Profile PVC	21” to 36”	ASTM F 1803	N/A	46 psi	Bell and Spigot
Gravity Flow	Truss Type PVC	8” to 15”	ASTM D 2680	N/A	200 psi	Bell and Spigot
Gravity Flow	RCP	18” to 144”	ASTM C 76	Class IV Wall B	4,000 psi	Tongue and Groove
Gravity Flow	Ductile Iron	8” to 54”	AWWA C151	Class 52	300 psi	MJ or Push on
Gravity Flow	VCP	8” to 42”	ASTM C 700	N/A	N/A	Bell and Spigot
Gravity Flow	Double Walled Polypropylene	12” to 30”	ASTM F 2736	N/A	46 psi	Bell and Spigot
Gravity Flow	Triple Walled Polypropylene	30” to 36”	ASTM F 2764	N/A	46 psi	Bell and Spigot
Force Main	Ductile Iron	4” to 64”	AWWA C151	Class 52	300 psi	MJ or Push on
Force Main	PVC	4” to 12”	AWWA C 900	DR 18	150 psi	Bell and Spigot
Force Main	PVC	14” to 30”	AWWA C 905	DR 18	150 psi	Bell and Spigot

Gravity mains greater than 42 inches in diameter will be lined reinforced concrete pipe or ductile iron.
Force mains greater than 30 inches in diameter will be ductile iron.

Table 3D-1.02: Manhole Types

Figure No.[1]	Type	Description	Depth Restrictions
6010.301	SW-301	Circular Sanitary Sewer Manhole	N/A
6010.302	SW-302	Rectangular Sanitary Sewer Manhole	12’ max.
6010.303	SW-303	Sanitary Sewer Manhole Over Existing Sewer	N/A
6010.304	SW-304	Rectangular Base/Circular Top Sanitary Sewer Manhole	12’ min. to 22’ max.
6010.305	SW-305	Tee-section Sanitary Sewer Manhole	N/A

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.

Table 3D-1.03: Manhole Casting Types

Figure No.[1]	Casting Type	Number of Pieces	Ring/ Cover	Bolted Frame	Bolted Cover (Floodable)	Gasket
6010.601	SW-601, A	2	Fixed[3]	Yes	No	Yes[2]
6010.601	SW-601, B	3	Adjustable[4]	No	No	Yes[2]
6010.601	SW-601, C	2	Fixed[3]	Yes	Yes	Yes[2]
6010.601	SW-601, D	3	Adjustable[4]	No	Yes	Yes[2]

[1] The figure numbers listed in this table refer to figures from the SUDAS Specifications.
[2] Machine bearing surfaces required.
[3] Typically used with non-paved or flexible surfaces, including HMA, seal coat, gravel, and brick.
[4] Typically used with PCC surfaces, including castings in concrete boxouts.

References

American Concrete Pipe Association. *Design Manual Concrete Pipe*. 2011.

American Water Works Association. *Water Distribution*.

Ductile Iron Pipe Research Association. *Handbook of Ductile Iron Pipe.*

Environmental Protection Agency Guidelines

Fair, Geyer, and Okun. *Water Supply and Wastewater Removal*. Vol. 1. 2010.

Great Lakes Standard Sewage Works. *10 State Standards*. 2004.

Iowa Department of Natural Resources Design Standards

WEF Manual of Practice No. FD-5. *ASCE Manual of Practice No. 60: Gravity Sanitary Sewer Design and Construction.*

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CHAPTER 4
Water Mains

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General Information

A. Concept

The important design requirements of water main systems are to supply each user with sufficient volume of water for a particular designated use plus required fire flows at adequate pressure, and to maintain the quality of the potable water delivered by the treatment plant. It is important that maintenance considerations are constantly addressed in the design of water main systems. The performance of a water main system for health and fire-flow purposes depends on the Jurisdiction's ability to maintain the system at an affordable cost.

Certain planning considerations related to a new system development or system expansion requires the designer to consider factors such as future growth, cost, and system layout. For system layout, all major demand areas should be serviced by an arterial-loop system. High demand areas are served by distribution mains tied to an arterial-loop system to form a grid without dead-end mains. Areas where adequate water supply must be maintained at all times for health and fire control purposes should be tied to two arterial mains where possible. Minor distribution lines or mains that make up the secondary grid system are a major portion of the grid since they supply the fire hydrants and domestic and commercial consumers.

B. Conditions

1. **General:** Numerous agencies, besides local Jurisdictions, may stipulate conformance to water main requirements. These agencies consist of water boards, benefited water districts, rural water associations, and the Iowa Department of Natural Resources (Iowa DNR). For the purpose of uniformity, the Project Engineer should contact the Jurisdictional Engineer if there are questions on where to submit reports, plans, and specifications for conformance to specific requirements and approvals. The Project Engineer should also contact the Jurisdictional Engineer to identify local requirements. It is necessary all water main projects meet the requirements of Iowa DNR and the evidence of approval be provided to the Jurisdiction in charge. In case of conflict between the above design standards, the most restrictive requirement applies.

2. **Plans:** The plans for water mains and appurtenances should show all appropriate physical features adjacent to the proposed water mains along with horizontal and vertical controls and hydrant coverages. Other utilities, such as sanitary and storm sewers, manholes, etc., should be shown on the plans with horizontal and vertical separation distances. Design details for other utilities that do not affect the water main may not be shown on water main plans. Traffic control criteria should also be included with the plans and should follow the latest edition of the Manual on Uniform Traffic Control Devices (MUTCD).

3. **Iowa DNR Project Submittals:** This section complies with the current edition of the Recommended Standards for Water Works (the Iowa Water Supply Design Standards by reference) as adopted by the Iowa DNR. The Project Engineer is responsible for obtaining any revisions, memorandums, and interpretations to the Iowa DNR rules and regulations.

a. General: All reports, final plans, specifications, and design criteria should be submitted at least 60 days prior to the date on which action by the reviewing authority is desired. Environmental assessments and permits for construction, to take water, for waste discharges, for stream crossings, etc., may be required from other federal, state, or local agencies. Preliminary plans and the engineer's report should be submitted for review prior to the preparation of final plans. No approval for construction can be issued until final, complete, detailed plans and specifications and the appropriate permit forms have been submitted to the reviewing authority and found satisfactory. Documents submitted for formal approval include, but are not limited to:

1) Engineer's report, where pertinent
2) Summary of the design criteria, including an up-to-date hydraulic analysis
3) Operation requirements, where applicable
4) General layout
5) Detailed plans
6) Specifications
7) Cost estimates
8) Water purchase contracts between water supplies, where applicable
9) Other information as required by reviewing authority

Where the design/build construction concept is to be utilized, special consideration must be given to: designation of a project coordinator; close coordination of design concepts and submission of plans and necessary supporting information to the reviewing authority; allowance for project changes that may be required by the reviewing authority; and reasonable time for project review by the reviewing authority.

b. Plans: Plans for water distribution system improvements should, where pertinent, provide the following:

1) General Layout, Including:
- a) Suitable title
- b) Name of municipality or other entity or person responsible for the water supply
- c) Area or institution to be served
- d) Scale
- e) North point
- f) Datum used
- g) Boundaries of the municipality or area to be served
- h) Date, name, and address of the designing engineer
- i) Conformance with engineering registration requirements of the state
- j) Legible prints suitable for reproduction
- k) Location and size of existing water mains
- l) Location and nature of existing water works structures and appurtenances affecting the proposed improvements, noted on one sheet

2) Detailed Plans, Including:
- a) Stream crossings, providing profiles with elevations of the stream bed and the normal and extreme high and low water levels
- b) Profiles having a horizontal scale of no more than 100 feet to the inch and a vertical scale of no more than 10 feet to the inch, with both scales clearly indicated
- c) Location of all existing and potential sources of pollution that may affect the water source or underground treated water storage facilities
- d) Size, length, and materials of proposed water mains
- e) Location of existing or proposed streets; water sources, ponds, lakes, and drains; storm, sanitary, combined, and house sewers; septic tanks, disposal fields, and cesspools

f) All appurtenances, specific structures, equipment, water treatment plant waste disposal units, and points of discharge having any relationship to the plans for water mains and/or water works structures
g) Locations of all sampling taps
h) Adequate description of any features not otherwise covered by the specifications

c. Specifications: Complete, detailed, technical specifications should be supplied for the proposed project, including:

1) A program for keeping existing water works facilities in operation during construction of additional facilities to minimize interruption of service
2) Procedures for flushing, disinfection, and testing, as needed, prior to placing the project in service
3) Materials or other facilities including any necessary backflow or back-siphon protection

See the Iowa DNR rules and regulations for more detail on submittal of reports, plans, and specifications.

d. Local Project Submittals: Some Jurisdictions or water boards have been delegated by Iowa DNR to issue permits for minor water main extensions. Permits for all other projects must be submitted to the Iowa DNR, and evidence of approval is to be provided to the Jurisdiction in charge. Include the Treatment Agreement form if appropriate.

Size Determination

A. General

Domestic usage requirements for a service area can be determined either from past records or from general usage information shown in Table 4B-1.01. This data should then be adjusted for commercial, industrial, and projected growth factors to ensure the system's design capacity should meet future demand.

A factor in sizing main facilities is the need for fire protection. Fire flow requirements are set by the Insurance Services Office (ISO). This group determines the minimum flow the system must be able to maintain for a specified period of time in order to achieve a certain fire protection rating. Fire insurance rates are then based, in part, on this classification.

B. Network Analysis

Pipe carrying capacity depends on pipe size, pressure, flow velocity, and head loss resulting from friction. Friction factors include roughness of pipe, flow velocity, and pipe diameter. The required pipe size can be calculated when the other requirements and characteristics are known.

When the distribution system or system expansion is extensive, it may be necessary to analyze the system and balance the flow among all areas in relation to demand. This analysis requires a plot of pressures and flows at points throughout the system.

C. Velocity Requirements

Velocity of flow is also a factor in determining the capacity of pipes and, therefore, the required pipe size. Velocities should normally be 5 fps or less, due to high friction losses that occur at greater velocities. This may be difficult to obtain under normal operating conditions, and velocities can significantly exceed this guideline under fire-flow conditions.

D. Minimum Criteria

1. **Minimum Design Period Requirements:** Water mains should have a minimum size based on a hydraulic analysis utilizing 20 year design for a specified water demand. Consideration should be given to projected land uses and demand based on full development of the service area. The specified water demand depends on the area to be serviced and the type of water main (feeder, arterial, or distribution).

2. **Minimum Size Requirements:**

   a. **Water Service Stub:** The water service stub must meet the Jurisdiction's standards and provide adequate design flows.

   b. **Distribution Mains:** All water mains should be sized large enough to provide existing and future residential, commercial, and industrial water demands and fire protection flows to the area to be served. The minimum water main size is 8 inches in diameter, unless otherwise approved by the Jurisdictional Engineer. The Jurisdiction reserves the right to increase the size of the mains to meet future water demands.

   c. **Arterial or Feeder Mains:** Arterial or feeder mains, typically 12 inches and larger, should conform to an existing grid pattern or as directed by the Jurisdiction to meet long range plans of the Jurisdiction.

3. **Pressure Requirements:** The recommended minimum operating pressure of the distribution system should be no less than 35 psi. The residual pressure required under fire flow conditions should not drop below 20 psi at any hydrant or any point in the system. When operating pressure exceeds 100 psi, individual or system pressure reducing devices may be required.

E. Flow Considerations

1. **Design Flows:** The water main system must be able to meet the following flow requirements:

   a. Peak day demands plus fire flow demands.

   b. Instantaneous peak demands for water mains from source, treatment, and/or storage facilities.

2. **Peak Day Demands:**

   a. **General:** The peak day demand is the average rate of consumption on the maximum day. The maximum day is the 24 hour period during which the highest consumption total is recorded in the latest 3 year period. High consumption that will not occur again due to changes in the system, or that was caused by unusual operations, should not be considered.

   When no actual figure for maximum daily consumption is available, it should be estimated on the basis of consumption in other cities of similar character. Such estimates should be at least 2.0 times greater than the average daily water demand for cities having more than 500 people and 2.5 times greater than the average daily water demand for cities having 500 people or less.

   b. **Average Day Demand (minimum):**

   Area x Area Density x Rate = Average Daily Demand **Equation 4B-1.01**

   Number of Units x Unit Density x Rate = Average Daily Demand **Equation 4B-1.02**

Table 4B-1.01: Density

Land Use	Area Density	Unit Density	Rate
Low Density (Single Family) Residential	10 people/AC	3.0 people/unit	100 gpcd
Medium Density (Multi-Family) Residential	15 people/AC	3.0 people/unit 6.0 people/duplex	100 gpcd
High Density (Multi-Family) Residential	30 people/AC	2.5 people/unit	100 gpcd
Office and Institutional	Special Design Density[1]		
Commercial	Special Design Density[1]		
Industrial	Special Design Density[1]		

[1] Special design densities should be subject to approval by the Jurisdictional Engineer based on methodology provided by the Project Engineer.

Note: If the Project Engineer uses values different than the above table, approval by the Jurisdictional Engineer and Iowa DNR is required.

3. **Instantaneous Peak Demands:** Where existing data is not available to accurately predict the instantaneous peak demand for the design year, the following criteria may be used as a minimum for estimating the instantaneous peak demand:

   a. 220 people or less = Average day demand (gpm) x 9.0.

   b. More than 220 people = Average day demand (gpm) x $7/P^{0.167}$
      P = design year population in thousands.

   If major water users exist in the system, the peak may be greater than those listed above.

4. **Fire Flows:** The following general information is taken from the *Fire Suppression Rating Schedule* (Edition 05-2008) of the Insurance Services Office (ISO). The latest ISO requirements must be checked to verify fire flow criteria. Insurance requirements for fire protection may vary with each Jurisdiction and must be confirmed by the Project Engineer.

   a. For one- and two- family dwellings not exceeding two stories in height, the following needed fire flows should be used.

Distance Between Buildings	Needed Fire Flow
Over 100’	500 gpm
31’ to 100’	750 gpm
11’ to 30’	1,000 gpm
10’ or less	1,500 gpm

   For wood shingle roof coverings on the building or on exposed buildings add 500 gpm to the needed fire flows.

   b. Multi-family, commercial, and industrial areas are considered high risk areas. The fire flows available in these areas require special consideration. The distribution and arterial mains in the high risk areas are to accommodate required fire flows in those areas.

Facility Design

A. General

Water mains and appurtenances, including hydrants and valves, should be provided along all streets including connections to and extensions from existing water systems.

The location and spacing of water mains and their appurtenances is not only important for service and fire protection, but also maintenance requirements. Figures 4C-1.02 through 4C-1.04 show guidelines for the location of these facilities.

B. Water Mains

1. Water main pipe will typically be either polyvinyl chloride (PVC) pipe or ductile iron pipe (DIP); and meet AWWA Standards. For larger mains (24 inch and greater), prestressed concrete cylinder pipe meeting AWWA Standards can be used.

   Where distribution systems and service connections are installed in areas of known groundwater contaminated by volatile organic compounds (LUST), pipe and joint materials (non-PVC pipe) that do not allow permeation of the volatile organic compounds must be used.

   The Iowa DNR requires underground storage tank (UST) owners to meet specific design requirements for USTs installed within 1,000 feet of a community water system. The Project Engineer should determine if there is an UST within 1,000 feet of the project area. If so, the Designer should determine the need to design the water mains to prevent future permeation of any volatile organic compounds into the water system. There are various elements to consider, some of which include soil types, groundwater table depth, size of the UST, age of the UST, etc.

   Consult with manufacturers concerning permeation of the pipe walls, jointing materials, valve seats, etc.

2. Water mains should be extended to the plat or property boundaries, to the next street, or as directed by the Jurisdiction.

3. New main installation should be located in the parking area (between the curb and the property line) of the right-of-way and minimum of 4 feet behind the curb. Where possible, water mains should be located along the south and east sides of the street.

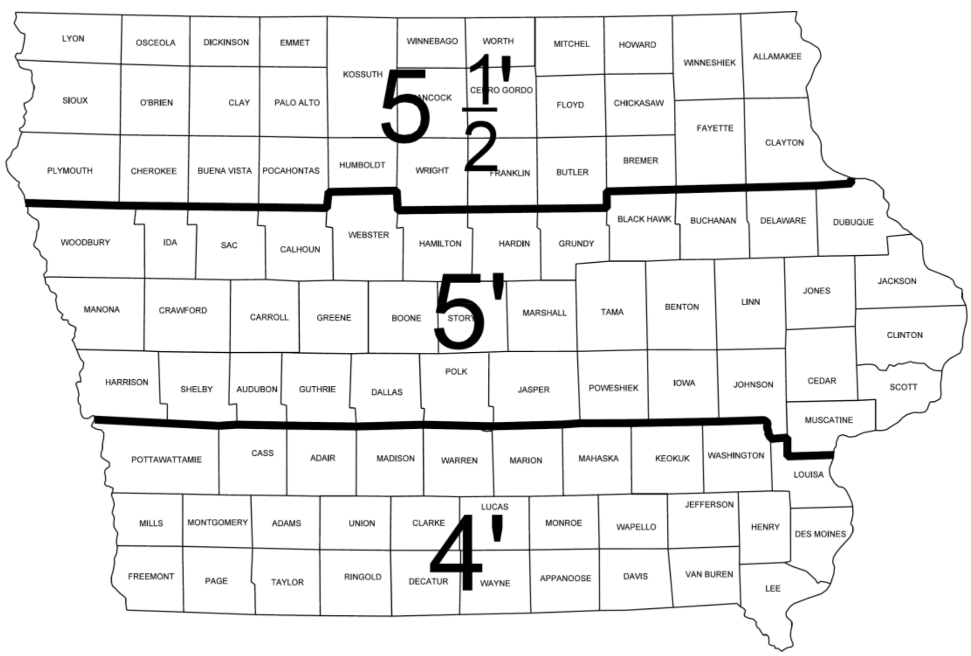
4. Dead-ends should be minimized by looping mains whenever possible. Dead-ends should terminate with an approved flushing device (blowoff, hydrant, flushing hydrant). They may terminate with an approved fire hydrant when adequate pressure is available at required flows. For maintenance considerations and when adequate fire flows are not available, flushing hydrants may be allowed by the Jurisdiction with the hydrant outlet sized and arranged to prevent the attachment of fire hoses. Unless required by a Jurisdiction, permanent inline shut-off valves should not be placed at the end of dead-end mains. A valve may be placed one or two pipe lengths back from the end of the project. No services should be placed past the valve. These

pipes will provide sufficient support for the valve and allow a future extension to be made without impacting current water customers.

5. Water mains and extensions should be designed with a minimum cover as indicated on Figure 4C-1.01, unless more or less cover has been approved by the Jurisdictional Engineer. Greater depths of cover, surface loading conditions, or unusual trench conditions may require a stronger class of pipe according to the AWWA Standard regarding the type of pipe being installed. Where a dip must be placed in a main in order to pass under another utility, the length of the deeper main should be kept to a minimum, and bends should be considered to affect the desired offset.

6. Water mains should be adequately protected from corrosive soil environments. Comply with AWWA C105. Complete soil testing or check with the Jurisdictional Engineer to determine if corrosive soils are present within the project area. If so, include polyethylene encasement for ductile iron pipe, valves, and fittings or use of other nonmetallic pipe materials. If nonmetallic materials are used, be sure to provide polyethylene encasement for fittings and valves. In severe instances, cathodic protection may be required.

Figure 4C-1.01: Minimum Depth of Cover for Water Main Installation



C. Blowoffs

A blowoff or approved flushing device is required on all dead-end mains where a hydrant is not installed. The minimum riser assembly size should be no less than 2 diameter sizes smaller than the diameter of the water main. The flushing device should be sized to provide flows that will give a velocity of at least 2.5 feet per second in the main being flushed. When the water main is extended, the blowoff should be removed. A new valve should be placed between the existing and extended main.

D. Valves

1. As a minimum, valves should be located at intersections, such that only one unvalved pipe exists at the intersection. Valves should be equally spaced, if possible, with spacing no more than 800 feet in residential areas and no more than 400 feet in high density residential, commercial, and industrial areas. (See Figures 4C-1.02 through 4C-1.04 for valve locations at intersections).

2. Valves should not be located in the sidewalk line or in driveways.

3. All valves should be installed with valve boxes.

4. No valves (except blowoff valves) should be placed at the end of a dead-end main unless required by a Jurisdiction. A valve should be installed between the existing main and new main when the main is extended. Intermediate valve locations between the end of a dead-end main and last valved street intersection may be required by the Jurisdiction to provide required valve spacing.

5. A tapping sleeve and valve should be used when making a perpendicular connection to an existing main.

6. If the project area has high water pressure, usually exceeding 100 psi, it may be appropriate to install system pressure relief valves as opposed to individual building controls. The potential for using a system pressure reducing valve is limited by the interconnected nature of a distribution system. Check with the Jurisdiction to determine the potential need for use of pressure reducing valves.

E. Fire Hydrants

1. Hydrants should comply with AWWA C502. The connecting pipe between the supply main and the hydrants should be a minimum of 6 inches in diameter and be independently valved. Fire hydrants should not be installed on water mains that do not provide a minimum pressure.

2. Hydrant drains should not be connected to or located within 10 feet of sanitary sewers.

3. Locations of fire hydrants are governed by the rules and regulations of the Iowa DNR and the local Jurisdiction and by the following principles. Satisfy each principle in the order they are listed. See Figures 4C-1.02 through 4C-1.04 for typical hydrant locations.

    a. Locate fire hydrants within 25 feet of each street intersection, measured from an end of a street paving return.

       Locate fire hydrants outside street paving returns. Avoid conflicts with storm sewers, intakes, and sidewalks. Whenever possible, locate fire hydrants at the high point of the intersection.

    b. Locate fire hydrants between street intersections to provide spacings of no more than 450 feet in single family residential districts and no more than 300 feet in all other districts. Coverage radii for structures as noted below should be checked when determining hydrant placement.

       Vary spacings slightly to place fire hydrants on extensions of property lines. When hydrants are required between intersections, they should be located at the high point of the main for air release or at a significant low point for flushing on the downhill side of an in-line valve.

When street curvature or grid patterns places a proposed protected structure at an unusual distance from the fire hydrant, the coverage radius should not exceed 300 feet in single family residential districts and 150 feet in all other districts. The Jurisdiction's fire marshall may have additional private fire protection requirements.

c. On cul-de-sac streets, hydrants should be located at the intersection of the cul-de-sac street and cross-street and the end of the cul-de-sac.

   1) For cul-de-sacs between 300 feet and 500 feet in length, an additional hydrant should be located at the mid-block.

   2) For cul-de-sacs greater than 500 feet in length, hydrants should be placed at near equal spacings, but not exceeding the spacings described above.

d. Hydrants must be located to provide the required fire flows. ISO evaluates fire hydrant locations within 1,000 feet of the test location, measured along the streets as fire hose can be laid, to evaluate the availability of water for fire protection. Hydrant capacity is credited as shown in the following table:

Hydrant Location	Credited Capacity
Within 300’ of location	1,000 gpm
Within 301’ to 600’ of location	670 gpm
Within 601’ to 1,000’ of location	250 gpm

F. Water Service Stubs

Water service stubs for each building or platted lot should be provided, including corporation stop, service line, and curb stop (shut-off) with box. Check with the Jurisdiction to determine appropriate placement location. In no case should the shut-off be in the sidewalk. Avoid locations where driveway approaches are likely to be constructed in the future.

G. Separation of Water Mains from Sewer Mains

The following comply with the Iowa Department of Natural Resources separation requirements.

1. **Horizontal Separation of Gravity Sewers from Water Mains:** Separate gravity sewer mains from water mains by a horizontal distance of at least 10 feet unless:
   - the top of a sewer main is at least 18 inches below the bottom of the water main, and
   - the sewer is placed in a separate trench or in the same trench on a bench of undisturbed earth at a minimum horizontal separation of 3 feet from the water main.

   When it is impossible to obtain the required horizontal clearance of 3 feet and a vertical clearance of 18 inches between sewers and water mains, the sewers must be constructed of water main materials meeting the requirements of SUDAS Specifications Section 5010, 2.01. However, provide a linear separation of at least 2 feet.

2. **Separation of Sewer Force Mains from Water Mains:** Separate sewer force mains and water mains by a horizontal distance of at least 10 feet unless:
   - the force main is constructed of water main materials meeting a minimum pressure rating of 150 psi and the requirements of SUDAS Specifications Section 5010, 2.01, and
   - the sewer force main is laid at least 4 linear feet from the water main.

3. **Separation of Sewer and Water Main Crossovers:** Vertical separation of sanitary and storm sewers crossing under any water main should be at least 18 inches when measured from the top of the sewer to the bottom of the water main. If physical conditions prohibit the separation, the sewer may be placed not closer than 6 inches below a water main or 18 inches above a water main. Maintain the maximum feasible separation distance in all cases. The sewer and water pipes must be adequately supported and have watertight joints. Use a low permeability soil for backfill material within 10 feet of the point of crossing.

   Where the sanitary sewer crosses over or less than 18 inches below a water main, locate one full length of sewer pipe of water main material so both joints are as far as possible from the water main.

   Where the storm sewer crosses over or less than 18 inches below a water main, locate one full length of sewer pipe of water main material or reinforced concrete pipe (RCP) with flexible O-ring gasket joints so both joints are as far as possible from the water main.

H. Surface Water Crossings

Comply with the Recommended Standards for Water Works, 2007 Edition. Surface water crossings, whether over or under water, present special problems. The reviewing authority should be consulted before final plans are prepared.

1. **Above-water Crossings:** Ensure the pipe is adequately supported and anchored; protected from vandalism, damage, and freezing; and accessible for repair or replacement.

2. **Underwater Crossings:** Provide a minimum cover of 5 feet over the pipe unless otherwise specified in the contract documents. When crossing water courses that are greater than 15 feet in width, provide the following.

   a. pipe with flexible, restrained, or welded watertight joints,

   b. valves at both ends of water crossings so the section can be isolated for testing or repair; ensure the valves are easily accessible and not subject to flooding, and

   c. permanent taps or other provisions to allow insertion of a small meter to determine leakage and obtain water samples on each side of the valve closest to the supply source.

I. Air Relief Facilities

1. **Air Relief Valves:** At high points in water mains where air can accumulate, provisions should be made to remove the air by means of air relief valves. Automatic air relief valves should not be used in situations where flooding of the manhole or chamber may occur.

2. **Air Relief Valve Piping:**

   a. Use of manual air relief valves is recommended wherever possible.

   b. The open end of an air relief pipe from a manually operated valve should be extended to the top of the pit and provided with a screened, downward-facing elbow if drainage is provided for the manhole.

for the manhole.

c. The open end of an air relief pipe from automatic valves should be extended to at least 1 foot above grade and provided with a screened, downward-facing elbow.

d. Discharge piping from air relief valves should not connect directly to any storm drain, storm sewer, or sanitary sewer.

J. Valve, Meter, and Blowoff Chambers

Wherever possible, chambers, pits, or manholes containing valves, blowoffs, meters, or other such appurtenances to a distribution system should not be located in areas subject to flooding or in areas of high groundwater. Such chambers or pits should drain to the ground surface or to absorption pits underground. The chambers, pits, and manholes should not connect directly to any storm drain or sanitary sewer. Blowoffs should not connect directly to any storm drain or sanitary sewer.

K. Thrust Blocks and Restrained Joints

Concrete thrust blocks and restrained joints are used to counteract joint movement at points where piping changes directions or at dead-ends.

1. **Thrust Blocks:** Concrete thrust blocks are typically used on pipes 16 inches in diameter or smaller. Thrust blocks may be used on other pipes independently or in combination with restrained joints. Table 4C-1.01 assumes a bearing area of thrust blocks based on 1,000 psf soil pressure and 150 psi water pressure. Where water pressures are higher and/or soil conditions are poor, the designer should design the correct block size using the equation below Table 4C-1.01. No bolts should come into contact with the concrete thrust blocks. If necessary, polyethylene wrap should be wrapped around the pipe, including the bolt circle, before the concrete is placed. Concrete should have a minimum compressive strength of 4,000 psi at 28 days.

Table 4C-1.01: Thrust Block Minimum Bearing Surface (SF)

Pipe Size (inches)	Bends				Tee or Dead-end
	11.25°	***22.5°***	***45°***	***90°***	
4	1.0	1.0	2.0	4.0	3.0
6	1.0	2.0	4.0	8.0	6.0
8	2.0	4.0	7.0	14.0	10.0
10	3.0	6.0	11.0	21.0	15.0
12	4.0	8.0	16.0	29.0	21.0
14	5.0	11.0	21.0	39.0	28.0
16	7.0	14.0	27.0	50.0	36.0
18	9.0	17.0	34.0	63.0	45.0
20	11.0	21.0	42.0	78.0	55.0
24	15.0	31.0	60.0	111.0	78.0
30	24.0	47.0	92.0	171.0	121.0
36	34.0	67.0	132.0	244.0	173.0

Note: Areas based upon water pressure of 150 psi and allowable soil pressure of 1,000 psf.

Required Area, ft^2 = (2) (water pressure, psi)(cross-sectional area of pipe outside diameter, in^2)
(sin(angle of bend / 2))/(allowable soil pressure, psf)

2. **Restrained Joints:**

   a. **For Pipe Diameters 8 inch through 12 inch:** Provide a minimum of 40 feet of restrained pipe in all directions along the pipe from the fitting for pipe diameters 8 inch through 12 inch, depths of bury of at least 5 feet, and a maximum test pressure of 150 psi.

   b. **For Pipe Diameters Greater than 14 inch:** Restrained joints are typically used on pipes larger than 14 inches in diameter. They may be used on other pipe sizes independently or in combination with concrete thrust blocks. See pipe manufacturer's recommendations for determining restrained lengths of pipe required.

L. Crossings

1. **Railroad Crossings:** The regulations of the railroad company involved will govern when a water main is installed under or over any railroad tracks.

2. **Roadway Crossings:** The jurisdiction responsible for the roadway should have regulations for crossing a roadway. For primary and interstate highways, the Iowa DOT is the responsible jurisdiction. For non-primary, federal-aid roadways use the most recent version of the "Policy for Accommodating Utilities on the County and City Non-Primary Federal-Aid System." For all other roadways, contact the responsible jurisdiction.

M. Flushing, Disinfection, and Pressure Tests

Before going into service, all new mains should be adequately flushed, pressure tested, and disinfected according to the rules and regulations of the local Jurisdiction and Iowa DNR. The procedures, once approved by the Jurisdiction, should be conducted under the supervision of the Jurisdiction or designated representative.

1. **Disinfection:** Disinfect the water main according to AWWA C651. Verify requirements and acceptable methods with the Engineer. Three methods of disinfecting new water mains are available. They include the tablet method, the continuous feed method, and the slug method. The tablet method is the most convenient, but the least effective. SUDAS Specifications Section 5030 indicates that the tablet method is not to be used unless approved by the Engineer. The continuous feed method is acceptable for general application. The goal for disinfection is to obtain a concentration in the new main of 25 mg/L free chlorine. The chlorine is to be retained in the pipe for a minimum of 24 hours, but no more than 48 hours.

2. **Flushing:** Once the main has passed the chlorination tests, it is to be flushed according to the requirements of AWWA C651 until the water in the new main is at the same chlorine level as the other sections of the distribution system. The velocity in the main should be at least 2.5 feet per second for adequate flushing. If there is any potential threat the highly chlorinated water will damage the environment, a neutralizing chemical should be added to the water to render it acceptable.

3. **Hydrostatic Pressure Testing:** Pressure test according to AWWA C600. All air must be expelled from the new main. The test pressure should be 1.5 times the working pressure of the system or 150 psi, whichever is greater. The test should continue for a minimum of 2 hours. If the pressure falls by 5 psi or more, additional makeup water must be added to return the pipe to the test pressure. The amount of makeup water used must meet the requirements of SUDAS Specifications Section 5030.

Figure 4C-1.02: Standard Water Main Location at Cul-de-sac

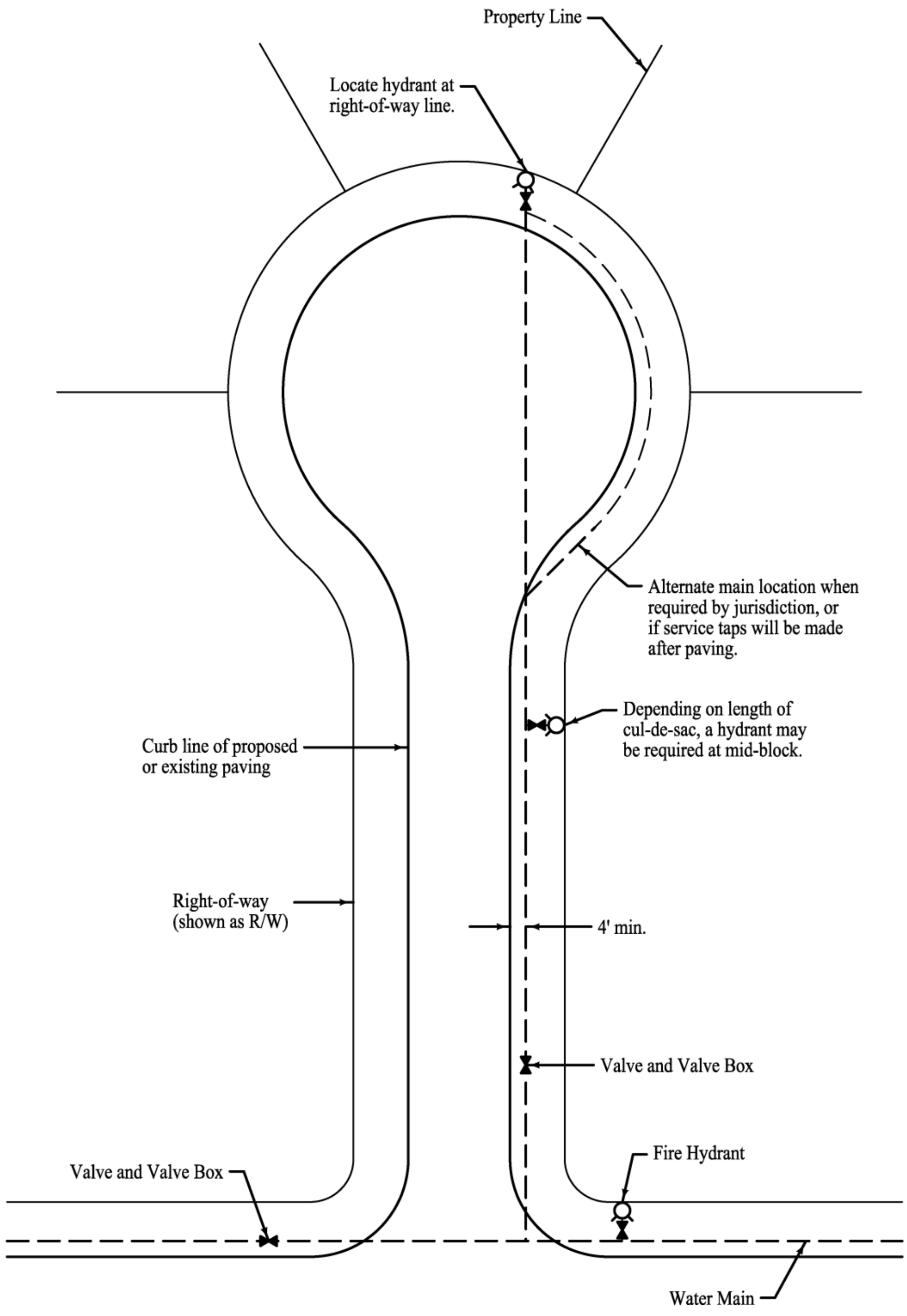
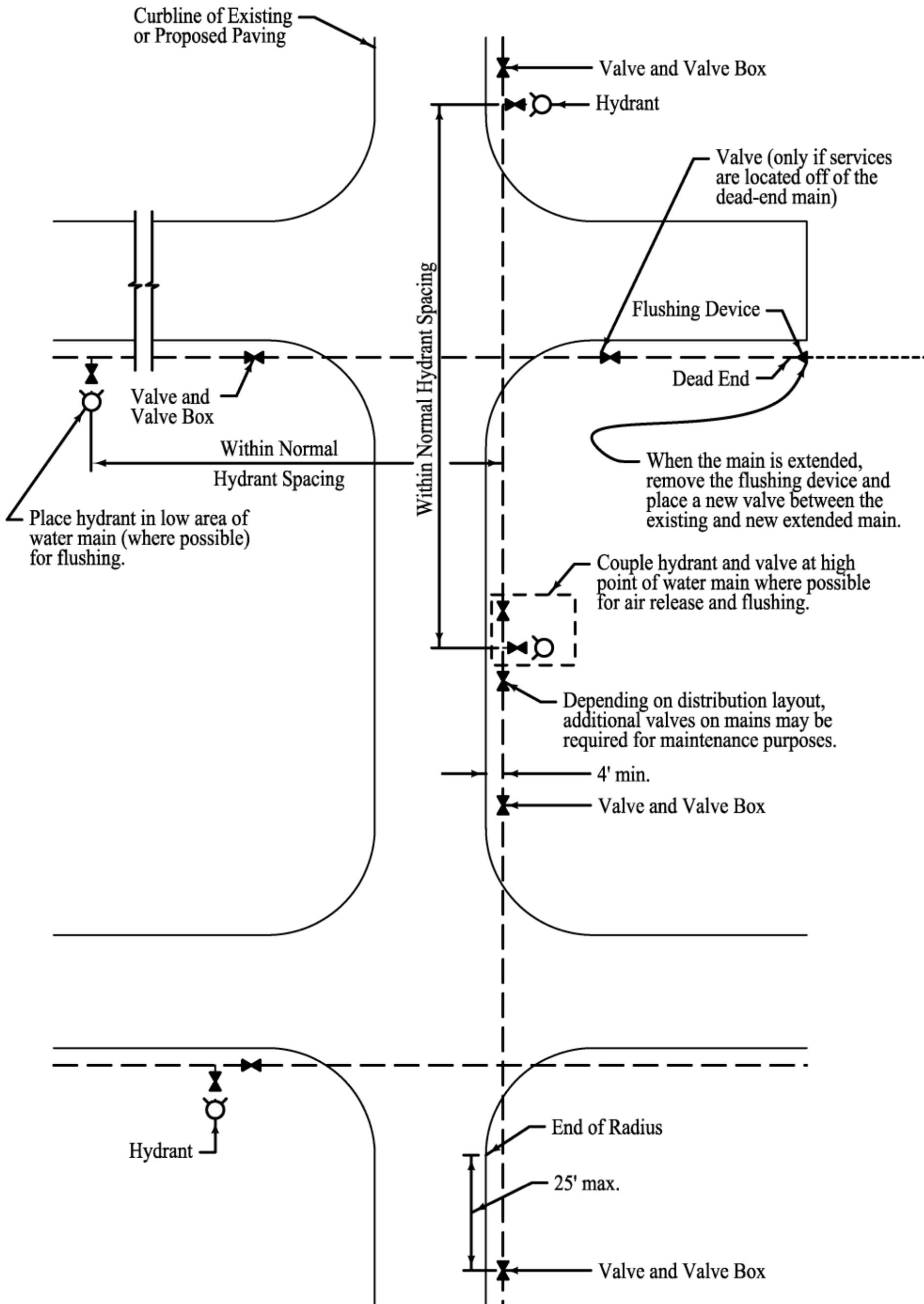


Figure 4C-1.03: Standard Water Main Location



Notes:

1. Install three valves and one fire hydrant at each intersection, except at T-intersections, which will have two valves.
2. Where possible, locate fire hydrant near high point.
3. Locate fire hydrants within 25 feet of intersection return radius, but outside of radius to avoid conflicts with storm sewers and intakes.
4. Where possible, locate fire hydrants on the downhill side of an in-line valve for air release and flusing purposes.

References

American Society of Civil Engineer Books and Manuals

American Water Works Association Standards

Great Lakes-Upper Mississippi River Board. *10 State Standards*. 2004.

Insurance Service Office (ISO). *Fire Suppression Rating Schedule*.

Iowa Administrative Rules

Iowa Department of Natural Resources Design Standards

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CHAPTER 5
Roadway Design

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General Information

A. Concept

The primary consideration of this chapter is that all new roadways and major reconstruction of existing corridors provide for safe, efficient, and economic transportation throughout the design life of the roadway. The values contained herein, specifically under design criteria, are to be considered basic design guidelines that will serve as framework for satisfactory design of new street and highway facilities. The Project Engineer is encouraged to develop the design based on this framework and tailored to particular situations that are consistent with the general purpose and intent of the design criteria through the exercise of sound engineering judgment.

The design criteria provided herein are divided into two classifications: preferred and acceptable. Designers should strive to provide a design that meets or exceeds the preferred criteria. Situations do arise that require special considerations; therefore, to eliminate hardships or problems, the Engineer may allow an exception to the preferred design criteria upon submittal of justification for such variances by the Project Engineer.

Cost effective design is encouraged along with the joint use of the transportation corridor and the consideration of the environment. The values contained herein are not intended as criteria for resurfacing, restoration, or rehabilitation projects.

B. References

The design for roadway facilities should comply with the current edition of the following references, unless a specific edition is cited:

Jurisdiction Supplemental Design Standards.

The American Association of State Highway and Transportation Officials (AASHTO). *A Policy on Geometric Design of Highways and Streets* (“Green Book”).

The American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide*.

The U.S. Department of Transportation - Federal Highway Administration. *Manual on Uniform Traffic Control Devices (MUTCD)* and *Traffic Control Device Handbook.*

Transportation Research Board. *Highway Capacity Manual*.

The Institute of Transportation Engineers. *Transportation and Traffic Engineering Handbook.*

Street Classifications

A. General

The classifying of streets and highways is necessary for communication among engineers, administrators, and the general public. Streets can be classified based upon major geometric features (e.g. freeways, streets, and highways), route numbering (e.g. U.S., State, and County), or Administrative classification (e.g. National Highway System or Non-National Highway System). However, functional classification, the grouping of streets and highways by the character of service they provide, was developed specifically for transportation planning purposes and is the predominant method of classifying streets for design purposes. For urban areas, the functional classification hierarchy consists of major arterials, minor arterials, collectors, and local streets.

The information contained in this section is based on AASHTO criteria. The Project Engineer should use the various AASHTO publications and particularly the current edition of AASHTO's "Green Book" to verify the application of values provided herein when complex design conditions or unusual situations occur.

B. Arterial Streets

1. **Major (Principal) Arterial:** The major arterial (referred to as a principal arterial by AASHTO) serves the major center of activities of urbanized areas, the highest traffic volume corridors, the longest trip, and carries a high proportion of a total urban travel on a minimum of mileage. The system should be integrated both internally and between major rural connections.

   The major arterial system carries most of the trips entering and leaving the area as well as most of the through movements bypassing the central city. In addition, significant intra-area travel such as between central business districts and outlining residential areas, between major inner-city communities, and between major suburban centers, is served by major arterials. Frequently, the major arterial carries important intra-urban as well as inter-city bus routes. Finally, in urbanized areas, this system provides continuity for all rural arterials that intercept the urban boundary.

   Access to private property from the major arterial is specifically limited in order to provide maximum capacity and through movement mobility. Although, no firm spacing rule applies in all or even in most circumstances, the spacing between major arterials may vary from less than 1 mile in highly developed central areas to 5 miles or more in developed urban fringes.

2. **Minor Arterial:** The minor arterial inter-connects with and augments the major arterial system. It accommodates trips of moderate length at a somewhat lower level of travel mobility than major arterials. This system places more emphasis on land access but still has specific limits on access points. A minor arterial may carry local bus routes and provide intra-community continuity but ideally does not penetrate identifiable neighborhoods. This system includes urban connections to rural collector roads where such connections have not been classified as urban major arterials.

   The spacing of minor arterials may vary from 1/8 to 1/2 mile in highly developed areas to 2 to 3 miles in suburban fringes but is not normally more than 1 mile in fully developed areas.

C. Collector Streets

The collector street system provides both land access and traffic circulation within residential neighborhoods and commercial and industrial areas. It differs from the arterial system in that facilities on the collector system may penetrate residential neighborhoods, distributing trips from the arterials through the area to their ultimate destinations. Conversely, the collector street also collects traffic from local streets in residential neighborhoods and channels it into the arterial system. In the central business district, and in other areas of similar development and traffic density, the collector system may include the entire street grid.

1. **Major Collector:** This type of street provides for movement of traffic between arterial routes and minor collectors and may collect traffic, at moderately lower speeds, from local streets and residential and commercial areas. A major collector has control of access to abutting properties with a majority of access at local street connections. Normally, a slightly higher emphasis is placed on through movements than direct land access.

2. **Minor Collector:** This type of street provides movement of traffic between major collector routes and residential and commercial local streets as well as providing access to abutting property at moderate low speeds. Consideration for through movements and direct land access is normally equal.

D. Local Streets

Local streets allow direct access to abutting land and connections to the higher order street systems. They offer the lowest level of mobility and deliberately discourage major through traffic movements.

E. Private Streets

Certain Jurisdictions allow private streets in specific situations. Private streets are similar to the local streets but generally are located on dead-end roads less than 250 feet in length, short loop streets less than 600 feet in length, or frontage roads parallel to public streets. Design criteria for local private streets are not included in this manual. The Jurisdiction should be contacted to determine if they are allowed.

Geometric Design Tables

A. General

The following sections present two sets of design criteria tables - Preferred Roadway Elements (Table 5C-1.01) and Acceptable Roadway Elements (Table 5C-1.02). In general, the "Preferred" table summarizes design values taken from the AASHTO's "Green Book" that may be considered "preferred" while the "Acceptable" table represents AASHTO minimums or practical minimums not covered in AASHTO.

Designers should strive to provide a design that meets or exceeds the criteria established in the "Preferred" table. For designs where this is not practical, values between the "Preferred" and "Acceptable" tables may be utilized, with approval of the Engineer.

The Federal Highway Administration has modified some of the controlling geometric design criteria for projects on the National Highway System (NHS). These changes were based on an analysis of the 13 controlling criteria reported in NCHRP Report 783 and are incorporated in 23 CFR 625. The changes include reducing the number of criteria to 10 by eliminating bridge width, vertical alignment, and horizontal clearance since those elements were covered under another criteria or they were found not to have significant operational or safety impacts. For lower speed facilities with a design speed of less than 50 mph, the controlling criteria only includes design speed and structural capacity.

However, since all projects on the NHS, regardless of funding source, must meet the design guidelines in the Iowa DOT Design Manual, which includes the FHWA criteria, SUDAS has not modified the geometric design criteria contained herein that is used for locally funded and non-NHS Federal-Aid projects.

B. Design Controls and Criteria

The selection of various values for roadway design elements is dependent upon three general design criteria: functional classification, design speed, and adjacent land use.

1. **Functional Classification:** The first step in establishing design criteria for a roadway is to define the function that the roadway will serve (refer to Section 5B-1 for street classifications). The functional classification of the roadway is the basis for the cross-sectional design criteria shown in Tables 5C-1.01 and 5C-1.02. It also serves as the basis for the ultimate selection of design speed and geometric criteria.

   Under a functional classification system, design criteria and level of service vary according to the intended function of the roadway system. Arterials are expected to provide a high level of mobility for longer trip length; therefore, they should provide a higher design speed and level of service. Since access to abutting property is not their main function, some degree of access control is desirable to enhance mobility. Collectors serve the dual function of accommodating shorter trips and providing access to abutting property. Thus, an intermediate design speed and level of service is important. Local streets serve relatively short trip lengths and function primarily for property access; therefore, there is little need for mobility or high operating speeds. This function is reflected by use of lower design speeds and an intermediate level of service.

2. **Design Speed:** Design speed is the selected speed used to determine various geometric features of the roadway, including horizontal and vertical alignment. The design speed selected should be as high as practical to attain the desired degree of safety, mobility, and efficiency. It is preferred to select a design speed that is at least 5 mph greater than the anticipated posted speed limit of the roadway. Selecting a design speed equal to the posted speed limit may also be acceptable and should be evaluated on a project by project basis, subject to approval of the Engineer. Once the design speed is selected, all pertinent roadway features should be related to it to obtain a balanced design.

   In some situations, it may be impractical to conform with the desired design speed for all elements of the roadway (e.g. horizontal radius or clear zone). In these situations, warning signs or additional safety treatments may be required (e.g. warning signs or guard rail).

3. **Adjacent Land Use:** In addition to functional classification and design speed, the surrounding land use can impact the design elements of the roadway corridor as well. Land use can be categorized into three groups: residential, commercial, and industrial.

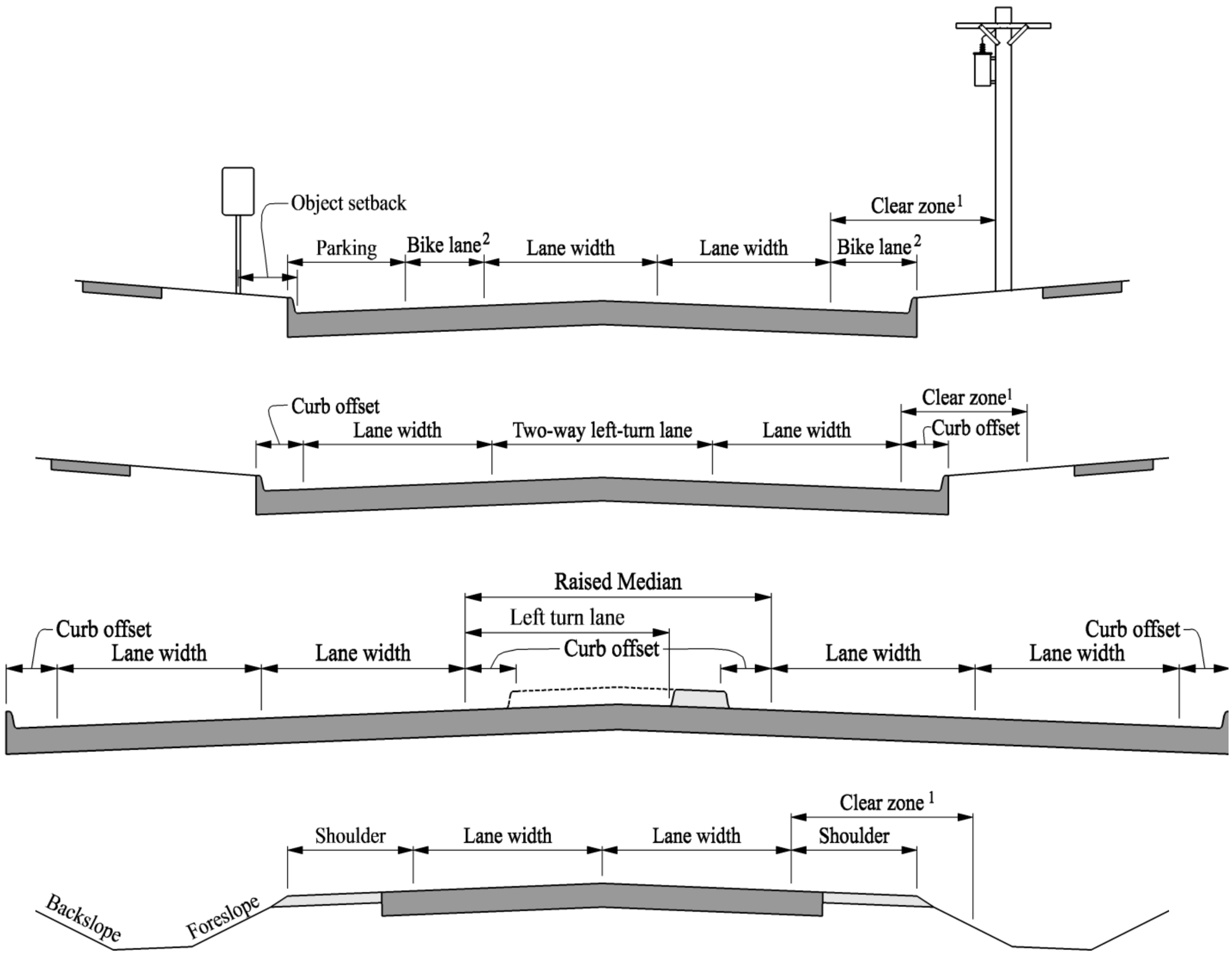
   a. Residential areas are regions defined by residential or multi-family zoning districts where single-family houses, apartment buildings, condominium complexes and townhome developments are located. Because these facilities typically have lower overall traffic volumes, low truck volumes, and are utilized primarily by drivers who are familiar with the roadway, some design values can be set at a lower level than for commercial or industrial areas.

   b. Commercial and industrial areas are highly developed regions generally defined by commercial and industrial zoning districts where factories, office buildings, strip malls, and shopping centers are or will be located. The areas typically require higher level design values due to increased traffic volumes, increased truck volumes, and decreased driver familiarity.

C. Roadway Design Tables

The following figures illustrate the location of various design elements of the roadway cross-section as specified in Tables 5C-1.01 and 5C-1.02.

Figure 5C-1.01: Roadway Design Elements



[1] Clear zone is measured from the edge of the traveled way.
[2] See Chapter 12 for bike lane requirements.

Table 5C-1.01: Preferred Roadway Elements

Elements Related to Functional Classification

Design Element	Local		Collector		Arterial	
	Res.	***C/I***	***Res.***	***C/I***	***Res.***	***C/I***
General						
Design level of service[1]	D	D	C/D	C/D	C/D	C/D
Lane width (single lane) (ft)[2]	10.5	12	12	12	12	12
Two-way left-turn lanes (TWLTL) (ft)	N/A	N/A	14	14	14	14
Width of new bridges (ft)[3]	See Footnote 3					
Width of bridges to remain in place (ft)[4]	----------	----------	----------	----------	----------	----------
Vertical clearance (ft)[5]	14.5	14.5	14.5	14.5	16.5	16.5
Object setback (ft)[6]	3	3	3	3	3	3
Clear zone (ft)	Refer to Tables 5C-1.03, 5C-1.04, and 5C-1.05					
Urban						
Curb offset (ft)[7]	2	2	2	3	3	3
Parking lane width (ft)	8	8	8	10	N/A	N/A
Roadway width with parking on one side[8]	26/31[9]	34	34	37	N/A	N/A
Roadway width without parking[10]	26	31	31	31	31	31
Raised median with left-turn lane (ft)[11]	N/A	N/A	19.5	20.5	20.5	20.5
Cul-de-sac radius (ft)	45	45	N/A	N/A	N/A	N/A
Rural Sections in Urban Areas						
Shoulder width (ft)						
ADT: under 400	4	4	6	6	10	10
ADT: 400 to 1,500	6	6	6	6	10	10
ADT: 1,500 to 2000	8	8	8	8	10	10
ADT: above 2,000	8	8	8	8	10	10
Foreslope (H:V)	4:1	4:1	4:1	4:1	6:1	6:1
Backslope (H:V)	4:1	4:1	4:1	4:1	4:1	4:1

Res. = Residential, C/I = Commercial/Industrial

Elements Related to Design Speed

Design Element	Design Speed, mph[12]							
	25	***30***	***35***	***40***	***45***	***50***	***55***	***60***
Stopping sight distance (ft)	155	200	250	305	360	425	495	570
Passing sight distance (ft)	900	1090	1,280	1,470	1,625	1,835	1,985	2,135
Min. horizontal curve radius (ft)[13]	198	333	510	762	1,039	926	1,190	1,500
Min. vertical curve length (ft)	50	75	105	120	135	150	165	180
Min. rate of vertical curvature, Crest (K)[14]	18	30	47	71	98	136	185	245
Min. rate of vertical curvature, Sag (K)	26	37	49	64	79	96	115	136
Minimum gradient (percent)	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Maximum gradient (percent)	5	5	5	5	5	5	5	5

Note: For federal-aid projects, documentation must be provided to explain why the preferred values are not being met. For non-federal aid projects, the designer must contact the Jurisdiction to determine what level of documentation, if any, is required prior to utilizing design values between the “Preferred” and “Acceptable” tables.

Table 5C-1.01 Footnotes:

[1] Number of traffic lanes, turn lanes, intersection configuration, etc. should be designed to provide the overall specified LOS at the design year ADT. Two LOS values are shown for collectors and arterials. The first indicates the minimum overall LOS for the roadway as a whole; the second is the minimum LOS for individual movements at intersections.

[2] Width shown is for through lanes and turn lanes.

[3] Bridge width is measured as the clear width between curbs or railings. Minimum bridge width is based upon the width of the traveled way (lane widths) plus 4 feet clearance on each side; but no less than the curb-face to curb-face width of the approaching roadway. Minimum bridge widths do not include medians, turn lanes, parking, or sidewalks. At least one sidewalk should be extended across the bridge.

[4] See Table 5C-1.02, for acceptable values for width of bridges to remain in place.

[5] Vertical clearance includes a 0.5 foot allowance for future resurfacing.

[6] Object setback does not apply to mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports.

[7] Values shown are measured from the edge of the traveled way to the back of curb. Curb offset is not required for turn lanes. On roadways with an anticipated posted speed of 45 mph or greater, mountable curbs are required. For pavements with gutterline jointing, the curb offset should be equal to or greater than the distance between the back of curb and longitudinal gutterline joint.

[8] Parking is allowed along one side of local or collector streets unless restricted by the Jurisdiction. Some jurisdictions allow parking on both sides of the street. When this occurs, each jurisdiction will set their own standards to allow for proper clearances, including passage of large emergency vehicles. Parking is normally not allowed along arterial roadways.

[9] For local, low volume residential streets, two free flowing lanes are not required and a 26 foot or 31 foot (back to back) roadway may be used where parking is allowed on one side or both sides respectively. For higher volume residential streets, which require two continuously free flowing traffic lanes, a 31 foot or 37 foot roadway should be used for one sided or two sided parking respectively.

[10] Some minimum roadway widths have been increased to match standard roadway widths. Unless approved by the Jurisdiction, all two lane roadways must comply with standard widths of 26, 31, 34, or 37 feet.

[11] Median width is measured between the edges of the traveled way of the inside lanes and includes the curb offset on each side of the median. Values include a left turn lane with a 6 foot raised median as required to accommodate a pedestrian access route (refer to Chapter 12) through the median (crosswalk cut through). At locations where a crosswalk does not cut through the median, the widths shown can be reduced by 2 feet to provide a 4 foot raised median.

[12] It is preferred to select a design speed that is at least 5 mph greater than the anticipated posted speed limit of the roadway. Selecting a design speed equal to the posted speed limit may also be acceptable and should be evaluated on a project by project basis, subject to approval of the Engineer.

[13] Values for low design speed (<50 mph) assume no removal of crown (i.e. negative 2% superelevation on outside of curve). Radii for design speeds of 50 mph or greater are based upon a superelevation rate of 4%. For radii corresponding to other superelevation rates, refer to the AASHTO’s “Green Book.”

[14] Assumes stopping sight distance with 6 inch object.

Table 5C-1.02: Acceptable Roadway Elements

Elements Related to Functional Classification

Design Element	Local		Collector		Arterial	
	Res.	***C/I***	***Res.***	***C/I***	***Res.***	***C/I***
General						
Design Level-of-Service[1]	D	D	D/E	D/E	D/E	D/E
Lane width (single lane) (ft)[2]	10	11	11	11	11	11
Two-Way Left-Turn Lanes (TWLTL) (ft)	N/A	N/A	12	12	12	12
Width of new bridges, (ft)[3]	See Footnote 3					
Width of bridges to remain in place (ft)[4]	20	22	24	24	26	26
Vertical clearance (ft)[5]	14.5	14.5	14.5	14.5	14.5	14.5
Object setback (ft)[6]	1.5	1.5	1.5	1.5	1.5	1.5
Clear zone (ft)	Refer to Tables 5C-1.03, 5C-1.04, and 5C-1.05					
Urban						
Curb offset (ft)[7]	1.5[8]	1.5[8]	1.5[8]	1.5[8]	2	2
Parking lane width (ft)	7.5	7.5	7.5	9	10	10
Roadway width with parking[9, 11]	26/31[10]	31	31	34[11]	34	34
Roadway width without parking[11]	26[10]	26	26	26	26	26
Raised median with left-turn lane (ft)[12]	N/A	N/A	18	18	18.5	18.5
Cul-de-sac radius (ft)	45	45	N/A	N/A	N/A	N/A
Rural Sections in Urban Areas						
Shoulder width (ft)						
ADT: under 400	2	2	2	2	8	8
ADT: 400 to 1,500	5	5	5	5	8	8
ADT: 1,500 to 2,000	6	6	6	6	8	8
ADT: over 2,000	8	8	8	8	8	8
Foreslope (H:V)[13]	3:1	3:1	3:1	3:1	4:1	4:1
Backslope (H:V)	3:1	3:1	3:1	3:1	3:1	3:1

Res. = Residential, C/I = Commercial/Industrial

Elements Related to Design Speed

Design Element	Design Speed, mph[14]															
	25		***30***		***35***		***40***		***45***		***50***		***55***		***60***	
Stopping sight distance (ft)	155		200		250		305		360		425		495		570	
Passing sight distance (ft)	900		1,090		1,280		1,470		1,625		1,835		1,985		2,135	
Min. horizontal curve radius (ft)[15]	198		333		510		762		1,039		833		1,060		1,330	
Min. vertical curve length (ft)	50		75		105		120		135		150		165		180	
Min. rate of vert. curve, Crest (K)[16]	12		19		29		44		61		84		114		151	
Min. rate of vert. curve, Sag (K)	26		37		49		64		79		96		115		136	
Min. rate of vert. curve, Sag (K) based on driver comfort/overhead lighting[17]	14		20		27		35		44		54		66		78	
Minimum gradient (percent)[18]	0.5		0.5		0.5		0.5		0.5		0.5		0.5		0.5	
Maximum gradient (percent)[19]	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I
Local	12	10	12	9	11	9	11	9	10	8	9	8	N/A	N/A	N/A	N/A
Collector	12	9	11	9	10	9	10	9	9	8	8	7	N/A	N/A	N/A	N/A
Arterial	N/A	N/A	9	9	8	8	8	8	N/A	7	N/A	7	N/A	6	N/A	6

R = Residential, C/I = Commercial/Industrial

Note: For federal-aid projects, proposed design values that do not meet the "Acceptable" table may require design exceptions. Design exceptions will be considered on a project-by-project basis and must have concurrence of the Iowa DOT when applicable. For non-federal aid projects, the designer should contact the Jurisdiction to determine what level of documentation, if any, is required prior to utilizing design values that do not meet the "Acceptable" table.

Table 5C-1.02 Footnotes:

[1] Number of traffic lanes, turn lanes, intersection configuration, etc. should be designed to provide the specified LOS at the design year ADT.

[2] Width shown is for through lanes and turn lanes.

[3] Bridge width is measured as the clear width between curbs or railings. Minimum bridge width is based upon the width of the traveled way (lane widths) plus 3 feet clearance on each side; but no less than the curb-face to curb-face width of the approaching roadway. Minimum bridge widths do not include medians, turn lanes, parking, or sidewalks. At least one sidewalk should be extended across the bridge.

[4] The values shown are the clear width across the bridge between curbs or railings. Values are based upon the width of the traveled way (lane width) and include a 1 foot and 2 foot offset on each side for collectors and arterials respectively. Values do not include medians, turn lanes, parking, or sidewalks. In no case should the minimum clear width across the bridge be less than the width of the traveled way of the approach road.

[5] Vertical clearance includes a 0.5 foot allowance for future resurfacing. Vertical clearance of 14.5 feet on arterials is allowed only if an alternate route with 16 feet of clearance is available.

[6] Object setback does not apply to mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports.

[7] Values shown are measured from the edge of the traveled way to the back of curb. Curb offset is not required for turn lanes. On roadways with an anticipated posted speed of 45 mph or greater, mountable curbs are required. For pavements with gutterline jointing, the curb offset should be equal to or greater than the distance between the back of curb and longitudinal gutterline joint.

[8] At locations where a 1.5 foot curb offset is used, an alternative intake boxout, with the intake set back a minimum of 6 inches from the curb line, must be used to prevent intake grates from encroaching into the traveled way.

[9] Some jurisdictions allow parking on both sides of the street. When this occurs, each jurisdiction will set their own standards to allow for proper clearances, including passage of large emergency vehicles.

[10] For low volume residential streets, two free flowing lanes are not required and a 26 foot roadway may be used where parking is allowed on one side only. For higher volume residential streets, which require two continuously free flowing traffic lanes, a 31foot roadway should be used.

[11] Some minimum roadway widths have been increased to match standard roadway widths. Unless approved by Jurisdiction, all two lane roadways must comply with standard widths of 26, 31, 34, or 37 feet.

[12] Median width is measured between the edges of the traveled way of the inside lanes and includes the curb offset on each side of the median. Values include a left turn lane with a 6 foot raised median as required to accommodate a pedestrian access route (refer to Chapter 12) through the median (crosswalk cut through). At locations where a crosswalk does not cut through the median, the widths shown can be reduced by 2 feet to provide a 4 foot raised median.

[13] The use of 3:1 foreslopes is allowed, as shown, but may require a wider clear zone as slopes steeper than 4:1 are not considered recoverable by errant vehicles.

[14] It is preferred to select a design speed that is at least 5 mph greater than the anticipated posted speed limit of the roadway. Selecting a design speed equal to the posted speed limit may also be acceptable and should be evaluated on a project by project basis, subject to approval of the Engineer

[15] Values for low design speed (<50 mph) assume no removal of crown (i.e. negative 2% superelevation on outside of curve). Radii for design speeds of 50 mph or greater are based upon a superelevation rate of 6%. For radii corresponding to other superelevation rates, refer to the AASHTO's "Green Book."

[16] Assumes stopping sight distance with 2 foot high object.

[17] Use only if roadway has continuous overhead lighting.

[18] A typical minimum grade is 0.5%, but a grade of 0.4% may be used in isolated areas where the pavement is accurately crowned and supported on firm subgrade.

[19] Maximum gradient may be steepened by 2% for short distances and for one way downgrades.

Table 5C-1.03: Preferred Clear Zone Distances for Rural and Urban Roadways

Design Speed mph	Design Traffic ADT	Foreslope			Backslope or Parking		
		6:1 or flatter	*5:1 to 4:1*	*3:1*	*6:1 or flatter*	*5:1 to 4:1*	*3:1*
		In feet from edge of traveled way					
Urban 40 or less	All	For low-speed urban roadways, refer to Table 5C-1.05.					
Rural 40 or less	Under 750	10	10	*	10	10	10
	750 to 1,500	12	14	*	12	12	12
	1,500 to 6,000	14	16	*	14	14	14
	Over 6,000	16	18	*	16	16	16
Rural and Urban 45 to 50	Under 750	12	14	*	12	10	10
	750 to 1,500	16	20	*	16	14	12
	1,500 to 6,000	18	26	*	18	16	14
	Over 6,000	22	28	*	22	20	16
Rural and Urban 55	Under 750	14	18	*	12	12	10
	750 to 1,500	18	24	*	18	16	12
	1,500 to 6,000	22	30	*	22	18	16
	Over 6,000	24	32	*	24	22	18
Rural and Urban 60	Under 750	18	24	*	16	14	12
	750 to 1,500	24	32	*	22	18	14
	1,500 to 6,000	30	40	*	26	22	18
	Over 6,000	32	44	*	28	26	22

Source: Adapted from the *Roadside Design Guide*, 2006

Table 5C-1.04: Acceptable Clear Zone Distances for Rural and Urban Roadways

Design Speed mph	Design Traffic ADT	Foreslope			Backslope or Parking		
		6:1 or flatter	*5:1 to 4:1*	*3:1*	*6:1 or flatter*	*5:1 to 4:1*	*3:1*
		In feet from edge of traveled way					
Urban 40 or less	All	For low-speed urban roadways, refer to Table 5C-1.05.					
Rural 40 or less	Under 750	7	7	*	7	7	7
	750 to 1,500	10	12	*	10	10	10
	1,500 to 6,000	12	14	*	12	12	12
	Over 6,000	14	16	*	14	14	14
Rural and Urban 45 to 50	Under 750	10	12	*	10	8	8
	750 to 1,500	14	16	*	14	12	10
	1,500 to 6,000	16	20	*	16	14	12
	Over 6,000	20	24	*	20	18	14
Rural and Urban 55	Under 750	12	14	*	10	10	8
	750 to 1,500	16	20	*	16	14	10
	1,500 to 6,000	20	24	*	20	16	14
	Over 6,000	22	26	*	22	20	16
Rural and Urban 60	Under 750	16	20	*	14	12	10
	750 to 1,500	20	26	*	20	16	12
	1,500 to 6,000	26	32	*	24	18	14
	Over 6,000	30	36	*	26	24	20

Source: Adapted from the *Roadside Design Guide*, 2006

* Foreslopes steeper than 4:1 are considered traversable, but not recoverable. An errant vehicle can safely travel across a 3:1 slope, but it is unlikely the driver would recover control of the vehicle before reaching the bottom of the slope; therefore, fixed objects should not be present on these slopes or at the toe of these slopes.

Table 5C-1.05: Clear Zone for Low-speed (40 mph or less Design Speed) Urban Roadways

Roadway Classification	**Distance from the Edge of the Traveled Way, feet**[1]	
	Preferred	***Acceptable***
Arterial	10	7
Collector	8	5.5
Local	8	5.5

[1] Values in the table are measured from the edge of the traveled way. Parking lane, bike lane, and curb offset widths may be included as part of the clear zone; however, a minimum clear zone behind the back of curb of 6 feet (preferred) or 4 feet (acceptable) should be provided regardless of roadway classification. Clear zone requirements also apply along medians of divided roadways.

Source: Maze et al, 2008

D. References

American Association of State Highway and Transportation Officials (AASHTO) *Roadside Design Guide*. 3rd ed. Washington, DC. 2006.

Maze T. Hawkins N. et al. Clear Zone **-** A Synthesis of Practice and an Evaluation of the Benefits of Meeting the 10ft Clear Zone Goal on Urban Streets. Center For Transportation Research and Education. Iowa State University. 2008.

Geometric Design Elements

A. Level of Service

Level of service (LOS) is a measure of the operating conditions of a roadway facility. LOS is based upon traffic performance related to speed, travel time, freedom to maneuver, traffic interruptions, and comfort and convenience. The LOS ranges from A (least congested) to F (most congested). Refer to the Highway Capacity Manual for a more thorough discussion of the LOS concept.

Based upon the traffic capacity analysis, the number of lanes, turn lanes, and intersection controls should be selected to provide a design with the desired LOS for the design year traffic. Design year traffic is based upon a 20 year traffic projection. The current Highway Capacity Manual and the current AASHTO "Green Book" should be used for traffic projections and to determine the number of lanes and intersection configuration at the desired LOS.

The LOS for the roadway overall is based upon Average Daily Traffic (ADT), while the LOS at signalized intersections is based upon the peak hourly volume (PHV).

As a planning tool, the following tables are provided to indicate approximate capacities for two lane and four lane streets and highways and intersection capacity for four way stop and signalized intersections. These tables do not consider site specific details and should not be utilized for final design purposes.

Table 5C-2.01: Maximum ADT vs. LOS and Type of Terrain for Two Lane Highways

Terrain	LOS		
	B	***C***	***D***
Level	3,200 - 4,800	5,300 - 7,900	9,000 - 13,500
Rolling	1,800 - 2,800	3,500 - 5,200	5,300 - 8,000
Hilly	900 - 1,300	1,600 - 2,400	2,500 - 3,700

Table 5C-2.02: Reduced Capacity of Narrow Lanes with Restricted Lateral Clearance

Usable Shoulder Width or Clearance to Obstruction (feet)	**Two Lane Roadway** (percent of capacity of 12 feet lane)		
	12 feet lanes	***11 feet lanes***	***10 feet lanes***
6	100	93	84
4	92	85	77
2	81	75	68
0	70	65	58

Table 5C-2.03: Planning Capacity at LOS C[1], D, and E[2]
Two Way Arterial Streets (Non-intersection)

Number of Lanes	Turn Lanes	Capacity, VPD at LOS D			
		Minimal Side Friction	***Light (Residential) Side Friction***	***Moderate (Mixed Zoning) Side Friction***	***Heavy Side Friction***
Two Lanes Undivided	Without turn lanes	12,100	11,600	11,200	10,400
	With turn lanes	16,000	15,300	14,000	13,900
Four Lanes Undivided	Without turn lanes	24,300	23,400	23,400	21,900
	With left turn lanes or 5 lane with center TWLTL	32,100	30,900	30,900	29,100
Four Lanes Divided	Without turn lanes	27,100	26,200	26,100	23,300
	With left turn lanes	35,400	34,200	34,100	32,500
	With left and right turn lanes	37,500	36,200	34,400	34,400

LOS - Level of Service
TWLTL - Two-Way Left-Turn Lane
VPD - Vehicles per Day
[1] Capacity at LOS C may be determined by multiplying LOS D values above by 0.8.
[2] Capacity at LOS E may be determined by multiplying LOS D values above by 1.2.

Source: Adapted from "2000 Des Moines Area Daily Directional Capacities At Level of Service D" - Des Moines Area MPO

Table 5C-2.04: Approximate LOS C Service Volumes (VPH) for
Four Way Stop-controlled Intersections (Sum of all Four Legs)

Demand Split	**Two Lanes on Each Street**	**Street 1: Two Lanes Street 2: Four Lanes**	**Four Lanes on Each Street**
50/50	1,200	1,800	2,200
55/45	1,140	1,720	2,070
60/40	1,080	1,660	1,970
65/35	1,010	1,630	1,880
70/30	960	1,610	1,820

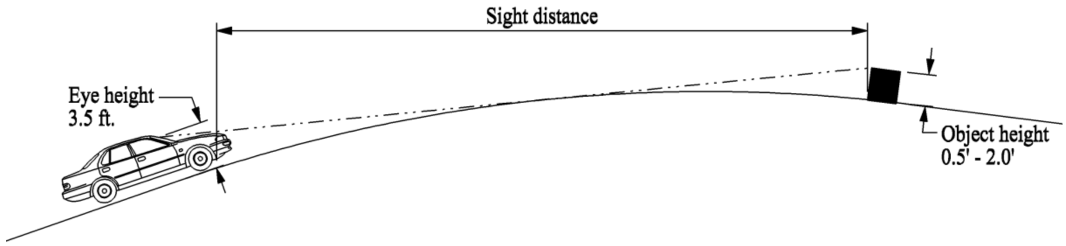
B. Sight Distance

The following information is taken from the 2004 AASHTO "Green Book." The Project Engineer should check the current edition of the AASHTO "Green Book" when specific information is needed to verify values provided.

1. **Stopping Sight Distances:** The minimum stopping sight distance is the distance required by the driver of a vehicle traveling at the design speed to bring the vehicle to a stop after an object on the road becomes visible. This distance directly affects the length and rate of curvature for vertical curves.

   The method for measuring stopping sight distance on vertical curves assumes a height for the driver's eye and a height for an object in the road. For a crest vertical curve, the sight distance is the distance at which an object in the road appears to the driver over the crest of the curve.

Figure 5C-2.01: Vertical Sight Distance Determination



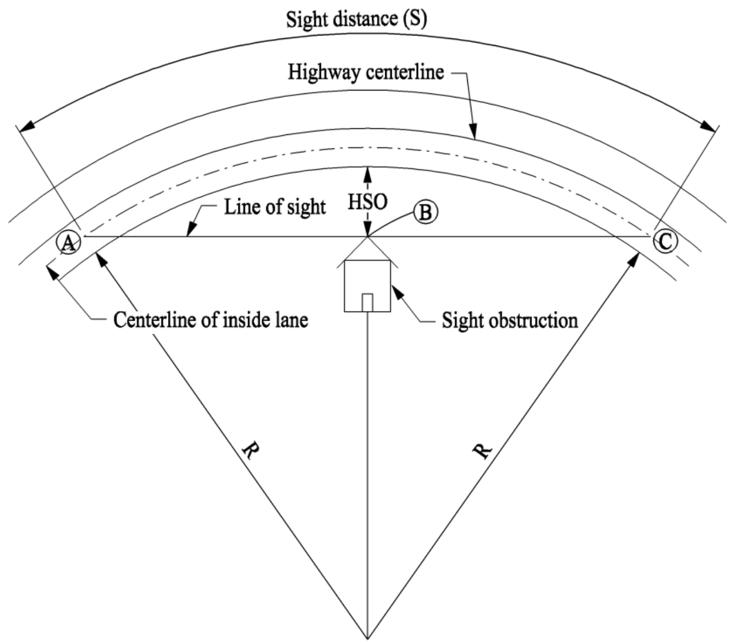
Stopping sight distance is calculated based upon an assumed height of the driver's eye and an assumed height of an object in the roadway. For all sight distance criteria, the height of the driver's eye is assumed to be 3.5 feet above the surface of the road, as recommended by AASHTO. Tables 5C-1.01 and 5C-1.02 in Section 5C-1 assume two different values for the height of the object in the roadway. The "Acceptable" values in Table 5C-1.02 use a 2 foot object height according to the current edition of the AASHTO "Green Book." The "Preferred" values in Table 5C-1.01 assume an object height of only 6 inches. This lower object height was the design value used in previous versions of the AASHTO "Green Book." The results of assuming a smaller object height for the preferred values in Table 5C-1.01 are higher required K values and longer vertical curves.

2. **Sight Distance on Horizontal Curves:** The horizontal alignment must provide at least the minimum stopping distance for the design speed at all points. This includes visibility around curves and roadside encroachments.

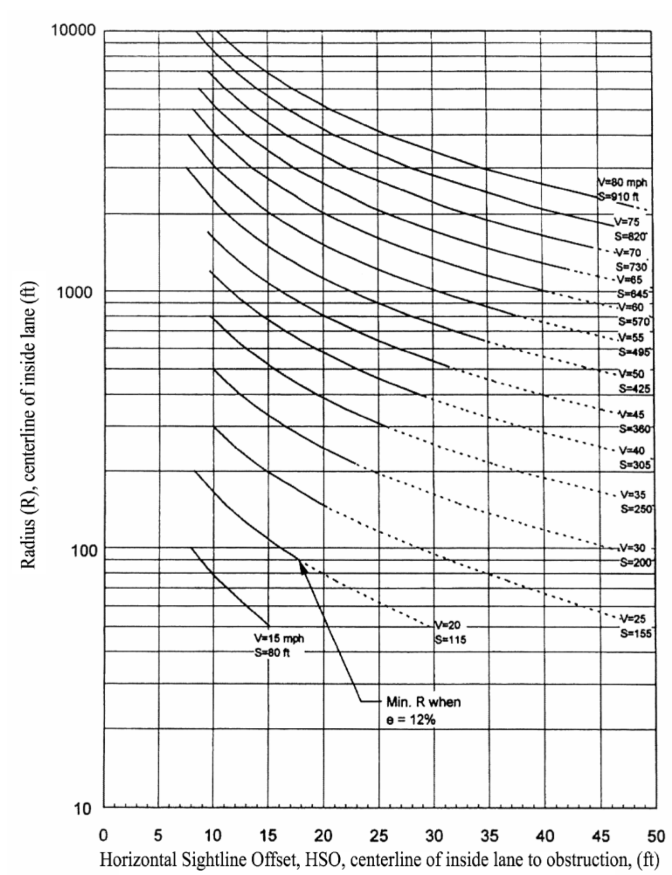
   Where there are sight obstructions such as walls, cut slopes, buildings, fences, bridge structures, or other longitudinal barriers on the inside of curves, an adjustment in the minimum radius of the curve may be necessary. In no case should sight distance be less than the stopping sight distance specified in Tables 5C-1.01 and 5C-1.02 in Section 5C-1. The sight distance design procedure should assume a 6 foot fence (as measured from finished grade) exists along all property lines except in the sight distance triangles required at all intersections.

   Available sight distance around a horizontal curve can be determined graphically using the method shown in Figures 5C-2.02 and 5C-2.03 below. From the center of the inside lane (Point A), a line is projected through the point on the obstruction that is nearest to the curve (Point B). The line is then extended until it intersects the centerline of the inside lane (Point C).

Figure 5C-2.02 and Figure 5C-2.03: Sight Distances for Horizontal Curves



HSO = Horizontal Sightline Offset



Source: Adapted from AASHTO "Green Book," 2004 Edition, Exhibits 3-53 and 3-54

3. **Passing Sight Distance:** Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another safely and comfortably without interfering with oncoming traffic traveling at the design speed. Two lane roads should provide adequate passing zones at regular intervals. Minimum passing sight distances are shown in Tables 5C-1.01 and 5C-1.02 in Section 5C-1.

   Passing sight distance is measured between an eye height of 3.5 feet and an object height of 3.5 feet. On straight sections of roadway, passing sight distance is determined primarily by the vertical curvature of the roadway. On horizontal curves, obstructions adjacent to the roadway on the inside of the curve can limit sight distance. This is most common in a cut section where the adjacent terrain projects above the surface of the roadway. Passing sight distance should be verified using the methods described in the current edition of the AASHTO "Green Book."

4. **Intersection Sight Distance:** In addition to the stopping sight distance provided continuously in the direction of travel on all roadways, adequate sight distance at intersections must be provided to allow drivers to perceive the presence of potentially conflicting vehicles. Sight distance is also required at intersections to allow drivers of stopped vehicles to decide when to enter or cross the intersecting roadway. If the available sight distance for an entering or crossing vehicle is at least equal to the appropriate stopping sight distance for the major road, then drivers have sufficient sight distance to anticipate and avoid collisions. However, in some cases, this may require a major road vehicle to slow or stop to accommodate the maneuver by a minor road vehicle. To enhance traffic operations, intersection sight distances that exceed stopping sight distances are desirable along the major road.

   Each intersection has the potential for several different types of vehicular conflicts. The possibility of these conflicts actually occurring can be greatly reduced by providing proper sight distance and appropriate traffic controls. Each quadrant of an intersection should contain a triangular area free of obstructions that might block an approaching driver's view of potentially conflicting vehicles. This clear area is known as the sight triangle.

   a. **Sight Triangles:** Proper sight distance at intersections is determined through the establishment and enforcement of sight triangles. The required dimensions of the legs of the triangle depend on the design speed of the roadways and the type of traffic control provided at the intersection. Two types of clear sight triangles are considered in intersection design: approach sight triangles and departure sight triangles.
      1) **Approach Sight Triangles:** Approach sight triangles allow the drivers at uncontrolled or yield controlled intersections to see a potentially conflicting vehicle in sufficient time to slow or stop before colliding within the intersection. Although desirable at all intersections, approach sight triangles are not needed for intersections approaches controlled by stop signs or traffic signals.
      2) **Departure Sight Triangles:** A second type of clear sight triangle provides sight distance sufficient for a stopped driver on a minor-road approach to depart from the intersection and enter or cross the major road. Departure sight triangles should be provided in each quadrant of each intersection approach controlled by a stop sign.

      At signalized intersections, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left turning vehicles should have sufficient sight distance to select gaps in oncoming traffic.

      The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection because different types of controls impose different legal constraints on drivers and, therefore, result in different driver behavior. The AASHTO "Green Book" contains the required procedures, equations, and tables for determining the required sight distance under various intersection and traffic control configurations.

b. **Identification of Sight Obstructions within Sight Triangles:** Within a sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered if practical. Such objects may include buildings, parked vehicles, highway structures, roadside hardware, hedges, trees, bushes, unmowed grass, tall crops, walls, fences, and the terrain itself. Particular attention should be given to the evaluation of clear sight triangles at intersection ramp/crossroad intersections where features such as bridge railings, piers, and abutments are potential sight obstructions.

The determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 3.5 feet above the roadway surface and that the approaching vehicle to be seen is 3.5 feet above the surface of the intersecting road.

C. Horizontal Alignment

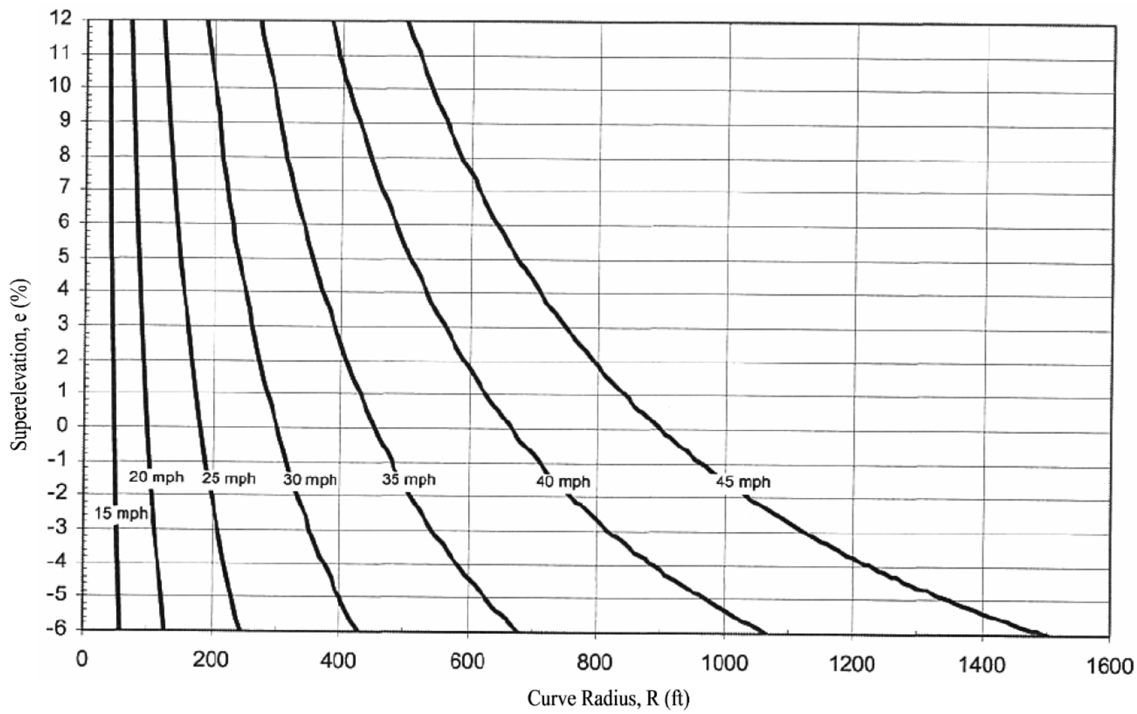
1. **Roadway Curvature and Superelevation:** On urban streets where operating speed is relatively low and variable, the use of superelevation for horizontal curves can be minimized. Although superelevation is advantageous for traffic operation, in urban areas the combination of wide pavements, the need to meet the grade of adjacent properties, the desire to maintain low speed operation, the need to maintain pavement profiles for drainage, and the frequency of cross streets and driveways and other urban features often combine to make the use of superelevation impractical or undesirable. Generally, the absence of superelevation on low speed urban streets is not detrimental to the motorist and superelevation is not typically provided on urban streets with a design speed of 45 mph or less.

The preferred radii shown in Section 5C-1, Table 5C-1.01 assume that a normal crown is maintained around a horizontal curve. With a standard 2% pavement cross-slope, this effectively results in a negative 2% superelevation for the outside lane. For roadways with a cross-slope other than 2%, including four lane and wider sections that utilize a steeper cross-slope for the outside lanes, the required curve radius should be determined from the guidance provided in the current AASHTO "Green Book" or from Figure 5C-2.04 below.

While superelevation on low speed urban roadways is not desirable, it may be necessary in situations where site conditions require a horizontal curve that cannot sustain traffic with the negative superelevation that results from maintaining the normal crown. For these situations, superelevation equal to the normal cross-slope may be provided for the outside lane. Section 5C-1, Table 5C-1.02 assumes the adverse crown in the outside lane of a curve is removed. For a roadway with a normal 2% cross-slope, this results in a superelevation of 2% across the width of the pavement. For roadways with cross-slopes other than 2%, the required radius and the resulting superelevation should be determined from the guidance provided in current AASHTO "Green Book" or from Figure 5C-2.04 below. The maximum superelevation for low speed urban roadways should not exceed the normal cross-slope or a maximum of 3%.

For roadways with design speeds of 50 mph or greater, superelevation of the roadway is acceptable and expected by motorists. The radii provided in Section 5C-1, Tables 5C-1.01 and 5C-1.02 are based upon superelevation rates of 4% and 6% respectively. The maximum superelevation rate in urban areas should not exceed 6%.

Figure 5C-2.04: Superelevation, Radius, and Design Speed for Low Speed (<50mph) Urban Street Design



Source: AASHTO "Green Book," 2004 Edition, Exhibit 3-17

2. **Intersection Alignment:** The centerline of a street approaching another street from the opposite side should not be offset. If the offset cannot be avoided, the offset should be 150 feet or greater for local streets. The centerline of a local street approaching an arterial or collector street from opposite side should not be offset unless such offset is 300 feet or greater.

3. **Adding, Dropping, or Redirecting Lanes:**

   a. **Dropping or Redirecting Through Lanes:** When dropping a lane, the minimum taper ratio to be used should be determined by the following formula, or from Table 5C-2.05:

      L = WS for velocities of 45 mph or more

      $L = \frac{WS^2}{60}$ for velocities of 40 mph or less.

      L = Minimum length of taper.
      S = Numerical value of posted speed limit or 85th percentile speed, whichever is higher.
      W = Width of pavement to be dropped or redirection offset.

   Preferably, taper ratios should be evenly divisible by five. Calculations that result in odd ratios should be rounded to an even increment of five. The table below utilizes the formulas to determine the appropriate taper ratio for dropping a 12 foot wide lane. The ratio remains constant for a given design speed while the length varies with the pavement width.

The procedure for determining minimum taper ratios for redirecting through lanes is the same as for lane drops, except for design speeds over 45 mph the use of reverse curves rather than tapers is recommended.

Table 5C-2.05: Length and Taper Ratio for Dropping 12 Foot Lane

Design Speed (mph)	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**
Taper Ratio	10:1	15:1	20:1	25:1	45:1	50:1	55:1	60:1
Length (feet)	120	180	240	300	540	600	660	720

b. Adding Through or Turn Lanes: For design speeds of 45 mph or greater, a 15:1 lane taper should be used when adding a left or right turn lane. For design speeds less than 45 mph, a 10:1 taper may be used.

For design speeds less than 45 mph, shorter tapers that are squared off or taper at 1:1 may provide better "targets" for approaching drivers and give more positive identification to an added through lane or turn lane. For turn lanes, the total length of taper and deceleration length should be the same as if a standard taper was used. This results in a longer length of full width pavement for the turn lane. This design provides increased storage that may reduce the likelihood turning vehicles will back up into the through lane during peak traffic periods. The use of short taper sections must be approved by the Engineer.

Figure 5C-2.05: Adding or Dropping Lanes

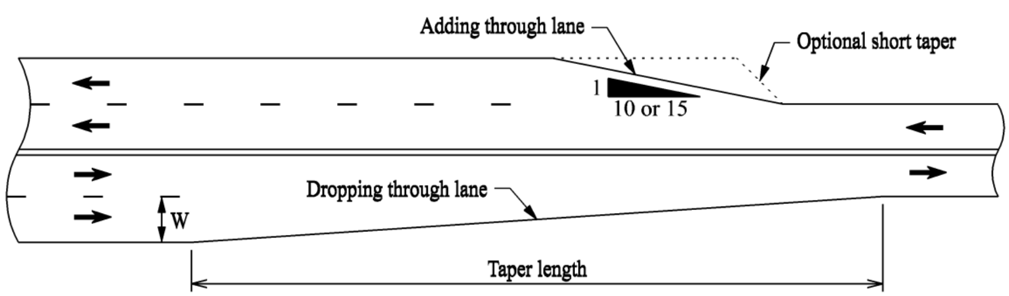
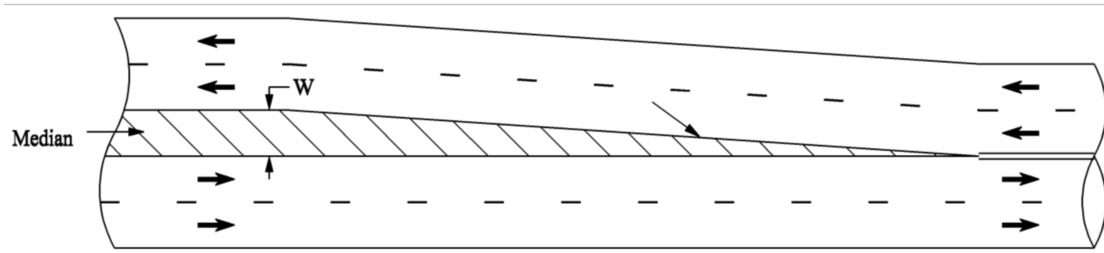


Figure 5C-2.06: Redirecting Through Lanes



D. Vertical Alignment

1. **Minimum Grades:** Flat and level grades on uncurbed pavements are preferred when the pavement is adequately crowned to drain the surface laterally. However, with curbed pavements, longitudinal grades must be provided to facilitate surface drainage. A typical minimum grade is 0.5%, but a grade of 0.4% may be used in isolated areas where the pavement is accurately crowned and supported on firm subgrade. The minimum allowance grade for bubbles and cul-de-sacs is 1%. Particular attention should be given to the design of stormwater inlets and their spacing to keep the spread of water on the traveled way within tolerable limits. Roadside channels and median swales frequently require grades steeper than the roadway profile for adequate drainage.

2. **Maximum Grades:** Grades for urban streets should be as level as practical, consistent with the surrounding terrain. The maximum design grades specified in Section 5C-1, Table 5C-1.02 should be used infrequently; in most cases grades should be less than the maximum design grade.

   Where sidewalks are located adjacent to a roadway, a maximum roadway grade of 5% is desirable. ADA requirements allow sidewalks adjacent to a roadway to match the running grade of the roadway, regardless of the resulting grade. However, sidewalk accessibility is greatly enhanced, especially over long distances, when grades are limited to 5% or less. It is recognized that meeting limitations will not be possible or practical in many situations; however, an attempt should be made to limit roadway grades to this level, especially in areas with high levels of anticipated pedestrian usage.

3. **Maximum Grade Changes:** Except at intersections, the use of grade breaks, in lieu of vertical curves, is not encouraged. However, if a grade break is necessary and the algebraic difference in grade does not exceed 1%, the grade break will be considered by the Engineer.

4. **Vertical Curves:** Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage.

   The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases. Wherever economically and physically feasible, more liberal stopping sight distances should be used. Furthermore additional sight distance should be provided at decision points.

   a. **Crest Vertical Curves:** Minimum lengths of crest vertical curves as determined by sight distance requirements are generally satisfactory from the standpoint of safety, comfort, and appearance. Figure 5C-2.06 shows the required length of crest vertical curve to provide stopping sight distance based upon design speed and change in grade.

   b. **Sag Vertical Curves:** Headlight sight distance is generally used as the criteria for determining the length of sag vertical curves. When a vehicle approaches a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. A headlight height of 2 feet and a 1 degree upward divergence of the light beam from the longitudinal axis of the vehicle is commonly assumed. For safety purposes, the sag vertical curve should be long enough that the light beam distance is the same as the stopping sight distance. Figure 5C-2.07 specifies the required sag curve length to meet the sight distance assumptions made above.

For both sag and crest vertical curves with a low algebraic difference in grade, sight distance restrictions may not control the design of the curve. In these cases, rider comfort and curve appearance are the primary considerations for vertical curve design. Generally, vertical curves with a minimum length (in feet) equal to three times the design speed (in mph) are acceptable.

If a roadway has continuous lighting, the length of sag vertical curve (L) may be based on passenger comfort instead of headlight sight distance. Use the following equation for the curve length:

$$L = \frac{AV^2}{46.5}$$

where A = algebraic difference in grades, %
V = design speed, mph

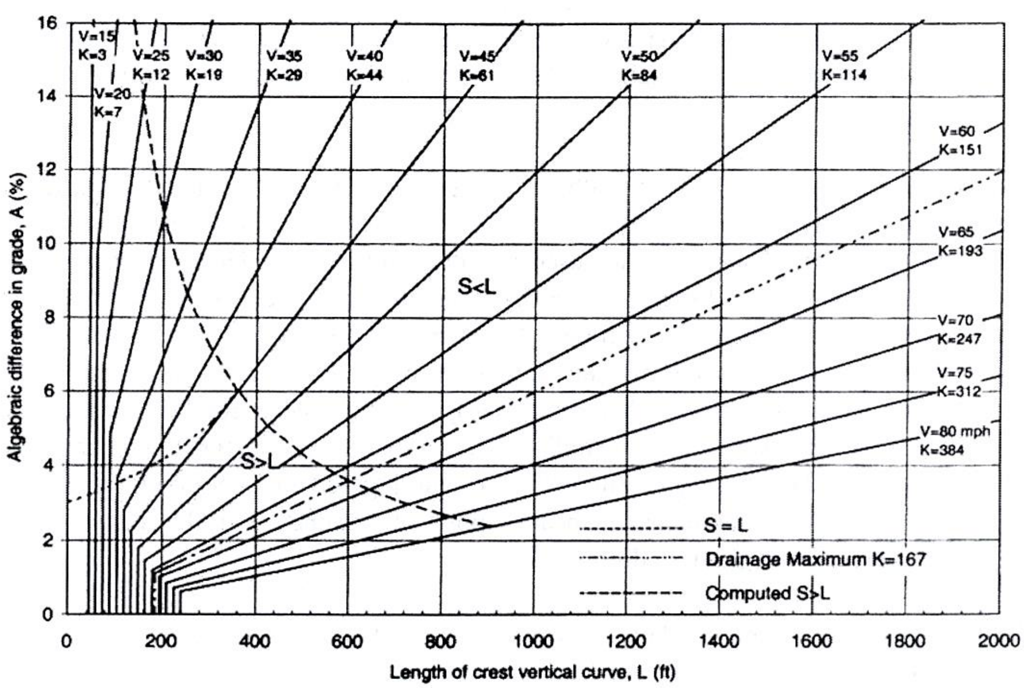
(Equation 3-51 AASHTO Greenbook, 2011)

Drainage considerations also affect the design of vertical curves where curbs are utilized. Both crest and sag vertical curves that have a grade change from positive to negative (or vice versa) contain a level area at some point along the curve. Generally, as long as a grade of 0.30% is provided within 50 feet of the level area, no drainage problems develop. This criterion corresponds to a K value of 167 and is indicated by a dashed line in Figures 5C-2.06 and 5C-2.07 below. K values greater than 167 may be utilized, but additional consideration should be given to drainage in these situations.

$$K = \frac{L(\text{ft})}{(g_2 - g_1)}$$

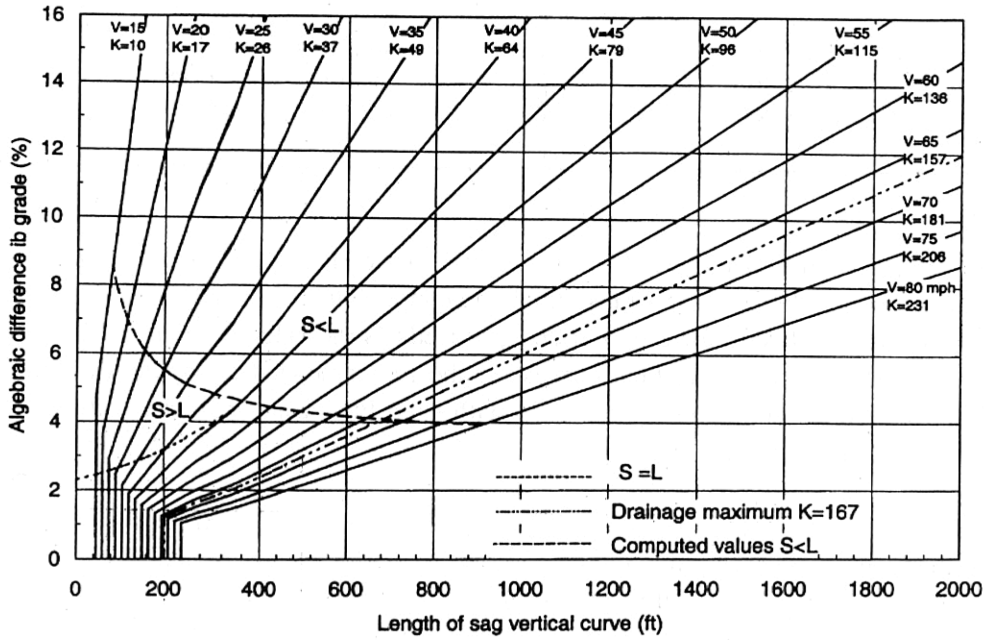
where g_1 and g_2 are in percent

Figure 5C-2.06: Design Controls for Crest Vertical Curves for Stopping Sight Distance and Open Road Conditions



Source: "Green Book," Exhibit 3-71, 2004

Figure 5C-2.07: Design Controls for Sag Vertical Curves, Open Road Conditions



Source: "Green Book," Exhibit 3-78, 2004

5. **Intersection Grades:** The grade of the "through" street should take precedence at intersections. At intersections of roadways with the same classifications, the more important roadway should have this precedence. Side streets are to be warped to match through streets with as short a transition as possible, which provides a smooth ride. Consideration must be given to minimize sheet flow of stormwater across the intersection due to loss of crown on the side street.

   Carrying the crown of the side street into the through street is not allowed. In most cases the pavement cross-slope at the warped intersection should not exceed the grade of the through street.

   The maximum desirable grades of the through street at the intersection and the side street cross-slope should be 2% and should not exceed 3%. The maximum desirable approach grade of the side street should not exceed 4% for a distance of 100 feet from the curb of the through street.

   Establishing intersection spot grades by matching "curb corners" of intersecting streets is not recommended since it may result in an undesirable travel path from the through street to the side street because of the resulting bump on the side street centerline. At sidewalk curb ramps in intersections, the street grades may need to be warped at the curb line to ensure the resulting cross-slope at the bottom of the ramp does not exceed 2%. A detail of the jointing layout with staking elevations should be shown on the plans.

   ADA regulations set specific limits for crosswalk cross-slopes that directly impact street and intersection grades. ADA regulations limit the cross-slope to 2% (measured perpendicular to the direction of pedestrian travel) for crosswalks that cross a roadway with stop control (stop sign) at the intersection. For roadways without stop control (through movement or traffic signal) the cross-slope of the crosswalk is limited to 5%. Effectively, this requirement limits street grades to a maximum of 2% or 5% depending on intersection controls.

   For steep roadways without stop control, construction of a flattened "table" may be necessary to reduce the street grade to 5% or less at the location of the crosswalk. Crosswalk tables at these locations must utilize vertical curves, appropriate for the design speed, to avoid a sudden change in grade at the intersection that could cause vehicles to bottom out or lose control.

For steep roadways with stop control, construction of a flattened “table” may utilize grade breaks or shortened vertical curves to reduce the street grade to 2% or less at the location of the crosswalk. A check should be made to verify that vehicles will not bottom out when traveling over the crosswalk table.

E. Pavement Crowns

The following typical pavement crowns are straight line cross-slope and are desirable sections.

1. **Urban Roadways (Curb and Gutter):** For streets with three or fewer travel lanes, the pavement crown should be 2%.

   For streets with four or more travel lanes, the pavement crown for all inside lanes, including left turn lanes, should be 2%. In order to reduce stormwater spread, the pavement crown for the outside lanes should be 3%.

   For all streets, auxiliary right turn lanes will have varying pavement crowns depending on the desired drainage pathway.

2. **Rural Roadways:** For pavement crowns, a 2% cross-slope is normal with 4% shoulder slope. Iowa DOT Standard Road Plans should be checked for Federal Aid, Farm to Market, and Secondary Roads.

F. Lane Width

The lane width of a roadway greatly influences the safety and comfort of driving. Narrow lanes force drivers to operate their vehicles closer to each other laterally than they would normally desire, resulting in driver discomfort, lower operating speeds, and reduced roadway capacity.

Tables 5C-1.01 and 5C-1.02 in Section 5C-1 indicate minimum lane widths based upon the roadway classification and adjacent land use. In addition to the lane width, a separate offset distance to the curb is required. This curb offset is not included in the lane widths listed.

Auxiliary lanes and turn lanes at intersections should be as wide as the adjacent through lanes. The width for turn lanes is measured to the face of curb. Because motorists are slowing in anticipation of making a turning movement, drivers are comfortable operating their vehicle closer to an adjacent obstacle (curb); therefore, turn lanes do not require a curb offset.

G. Two-way Left-turn Lanes (TWLTL)

Two-way left-turn lanes work well where design speeds are relatively low (25 to 50 mph) and there are no heavy concentrations of left turning traffic. The width of TWLTLs should be limited to a maximum of 14 feet to discourage left-turning motorists from pulling out into the TWLTL and stopping perpendicular to the direction of traffic, while they wait for oncoming traffic to clear.

H. Raised Median Width

A median is defined as the portion of a roadway separating opposing directions of the traveled way. The median width is expressed as the dimension between the edges of the traveled way and includes the left turn lanes, if any are present (refer to Section 5C-1, Figure 5C-1.01). The principal functions of a median are to separate opposing traffic, allow space for speed changes and storage of left turning and U-turning vehicles, minimize headlight glare, and provide width for future lanes. For maximum efficiency, a median should be highly visible both night and day and contrast with the traveled way lanes.

At unsignalized intersections on rural divided highways, the median should generally be as wide as practical. However, in urban areas, narrower medians appear to operate better at unsignalized intersections. If right-of-way is restricted, a wide median may not be justified if provided at the expense of a narrowed border area. A reasonable border width is needed to adequately serve as a buffer between private development along the road and the traveled way. Narrowing the border area may create operational issues similar to those that the median is designed to avoid. In addition, wide medians at signalized intersections result in increased time for vehicles to cross the median. This can lead to inefficient signal operation. Therefore, in urban areas, it is recommended that median width be only as wide as necessary to accommodate left turn lanes. Wider medians should only be used where needed to accommodate turning and crossing maneuvers by larger vehicles.

Medians and boulevards are not normally used on collector streets. However, when allowed, the median or boulevard should conform to the same design standards as set forth for arterial streets.

Median widths are also affected by sidewalk and crosswalk locations. Where a crosswalk cut through is present or proposed, medians (exclusive of any turn lanes) must be a minimum of 6 feet wide to comply with ADA regulations. These regulations require the placement of a 2 foot wide strip of detectable warnings at the curb line on both sides of the median. The detectable warnings must be separated by a minimum 2 foot strip without detectable warnings. Where the median has no curb, the detectable warnings must be placed along the edge of the roadway. At locations where a raised median is stopped short of the crosswalk, the 6 foot raised median and associated detectable warnings are not required, and a standard 4 foot raised median section may be used.

I. Bridges

The bridge widths listed in Section 5C-1, Tables 5C-1.01 and 5C-1.02 represent the clear roadway width (width between barrier rail faces). The widths shown do not account for barrier rail widths, sidewalk, recreational trails, etc.

For existing bridges, a structural analysis should be conducted. The existing bridge should be able to accommodate legal loads. Bridge guardrail should be upgraded if necessary.

J. Clear Zone

The AASHTO Roadside Design Guide (RDG) defines the clear zone as “the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear runout area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry.”

The intent of the clear zone is to provide an errant vehicle that leaves the roadway with an unobstructed recovery area. This area, including medians on divided roadways, should be kept free of all unyielding objects, including utility and light poles, culverts, bridge piers, sign supports, and any other fixed objects that might severely damage an out of control vehicle. Any obstruction that cannot be placed outside of the clear zone should be shielded by traffic barriers or guardrails.

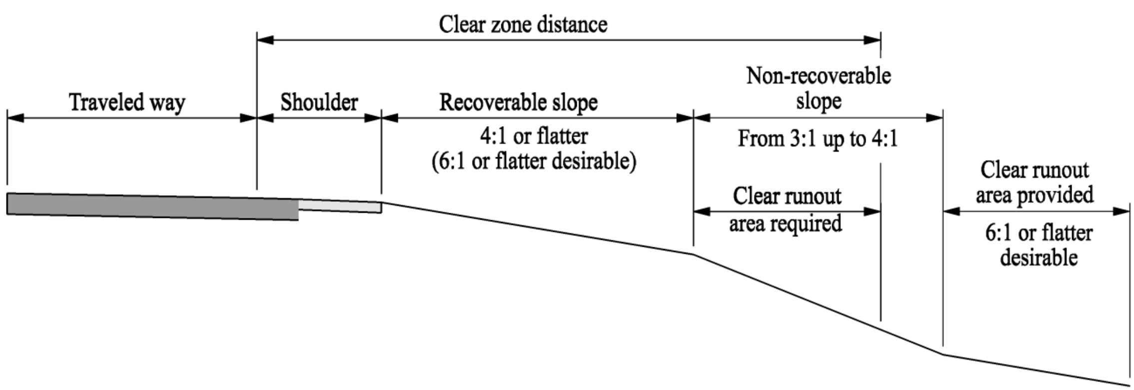
According to the AASHTO RDG, the width of this area varies based upon traffic volumes, design speed, and embankment slope.

Embankment slopes can be classified as recoverable, non-recoverable, or critical. Embankment slopes of 4:1 and flatter are considered recoverable. Drivers who encroach on recoverable slopes can generally stop their vehicles or slow them enough to return to the roadway safely.

A non-recoverable slope is defined as one that is passable, but from which most motorists will be unable to stop or to return to the roadway easily. Vehicles on such slopes are likely to reach the bottom before stopping. Embankments between 3:1 and 4:1 generally fall into this category. Since many vehicles will reach the toe of these slopes, the clear zone distance cannot logically end on a non-recoverable slope, and a clear runout area at the base of the slope is required. Fixed objects should not be present on a non-recoverable slope.

A critical slope is one on which a vehicle is likely to overturn. Slopes steeper than 3:1 generally fall into this category. If a slope steeper than 3:1 begins closer to the traveled way than the suggested clear zone, a barrier might be warranted if the slope cannot be flattened.

Figure 5C-2.08: Clear Zone Components



Source: Adapted from *Roadside Design Guide*, 2006

For horizontal curves, an adjustment factor may be applied to the clear zone width taken from Section 5C-1, Tables 5C-1.03 or 5C-1.04. This adjustment is only required at selected locations. Widening the clear zone should be considered along the outside of curves when crash history suggests the need for additional clear zone width, or whenever the radius of the curve is less than 2,860 feet, the design speed is 55 mph or greater, and the curve occurs on a normally tangent alignment (one where the curve is preceded by a tangent more than a mile in length).

The clear zone along an urban section may contain minor obstructions (traffic signs, mailboxes, etc.). In addition, along lower (<40 mph design speed) urban roadways, larger objects designed to "break-away" when struck by a vehicle may also be located within the clear zone (light poles, cast-iron fire hydrants, etc.). All objects, however, should be kept free from the object setback zone as described in the next section.

K. Object Setback

Like clear zone, object setback is intended to provide an area adjacent to the roadway that is clear of obstructions. However, the purpose of the object setback is to provide an operational clearance to increase driver comfort and avoid a negative impact on traffic flow. It also improves aesthetics, provides an area for snow storage and, in areas with curbside parking, provides a clear area to open car doors.

As discussed in the previous section on clear zone, minor obstructions and larger "breakaway" objects may be located in the clear zone on lower speed roadways (<40 mph design speed), but must be kept free from the object setback. Mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports, may be located within the object setback area.

Additional object setback, as measured from the back of curb, may be required around radii at intersections and driveways in order to provide sufficient clearance to keep the overhang of a truck from striking an object.

L. Border Area

Border area is the area between the roadway and the right-of-way line and is sometimes referred to as the "parking" in urban areas. The grade for the border area is normally 1/2 inch per foot. The border area between the roadway and the right of way line should be wide enough to serve several purposes including provision of a buffer space between pedestrians and vehicular traffic, sidewalk space, and an area for both underground and above ground utilities such as storm sewer, traffic signals, parking meters, and fire hydrants. The border area also provides snow storage and aesthetic features such as grass or other landscaping features. The border width ranges from 14 to 16 feet, including the sidewalk width. Traffic signals, utility poles, fire hydrants, and other utilities should be placed as far back of the curb as practical for safety reasons. Breakaway features should be built when feasible and as an aid to safety considerations.

Table 5C-2.08: Preferred Border Area

Street Classification	Border Area Width (feet)
Major/minor arterial	16
Collector	14.5
Local streets	14

M. Curbs

1. **Curb Offset:** The curb offset is measured from the back of curb to the edge of the lane. The curb offset increases driver comfort and roadway safety. The presence of the curb, and potential vehicle damage and loss of control resulting from striking the curb, causes drivers to move away from the curb, reducing the effective width of the through lane. Due to this driver reaction, and to accommodate the flow of drainage and intake structures, an offset between the curb and the edge of the traveled way is provided.

   The curb offset widths specified in Section 5C-1, Tables 5C-1.01 and 5C-1.02 do not necessarily indicate the width of the curb and gutter or the location of a longitudinal joint; however, the width of the curb and gutter can affect the required width of the curb offset. The presence of a longitudinal joint near the curb (gutterline jointing) can be a limiting factor for usable lane width as some drivers are uncomfortable driving on or near the joint line. This is especially true for HMA roadways with PCC curb and gutter. For pavements with a longitudinal joint line near the gutter, the curb offset should be equal to or greater than the width of the curb and gutter section. In addition, grates and special shaping for curb intakes and depressions for open-throat intakes should be located within the curb offset width and should not encroach into the lane.

2. **Curb and Gutter:** Typically, a curb and gutter cross-section should consists of a 6 inch high, 6 inch wide curb with a concrete gutter section. If the design speed is 40 mph or below, an 8 inch curb may be used for certain arterial and collector streets. For design speeds greater than 40 mph, a 1 foot wide, 6 inch high sloped curb with a minimum 2 foot gutter offset should be used.

N. Parking Lane

Where curbed sections are used, the curb offset width may be included as part of the parking lane.

1. Parking lanes are not allowed on arterial streets.

2. Although on-street parking may impede traffic flow, parallel parking may be allowed by the Jurisdiction on urban collectors where sufficient street width is available to provide parking lanes.

3. Parking lane width determinations should include consideration for the potential use of the lane as a through or turn lane for moving traffic either during peak hours or continuously. If this potential exists, additional parking width should be provided.

O. Cul-de-sacs

A local street open at one end only should have a cul-de-sac constructed at the closed-end. The minimum radius for cul-de-sacs is 45 feet, which may be increased in commercial areas or if significant truck traffic is anticipated. The border area around the cul-de-sac should be the same as the approach street. The transition radius with the approach street will be 50 feet for residential streets and 75 feet for commercial and industrial streets.

P. Shoulder Width

Shoulders accommodate stopped vehicles, emergency use, and provide lateral support of the subbase and pavement. In some cases, the shoulder can accommodate bicyclists. Where no curb and gutter is constructed a soil, granular, or paved shoulder will be provided.

Desirably, a vehicle stopped on the shoulder should clear the pavement edge by 2 feet. This preference has led to the adoption of 10 feet as the desirable shoulder width that should be provided along high volume facilities. In difficult terrain and on low volume highways, usable shoulders of this width may not be practical.

Where roadside barriers, walls, or other vertical elements are used, the graded shoulder should be wide enough that these vertical elements can be offset a minimum of 2 feet from the outer edge of the usable shoulder. It may be necessary to provide a graded shoulder wider than used elsewhere on the curved section of a roadway or to provide lateral support for guardrail posts and/or clear space for lateral dynamic deflection required by the particular barrier in use. On low volume roads, roadside barriers may be placed at the outer edge of the shoulder; however, a minimum of 4 feet should be provided from the traveled way to the barrier.

Q. Intersection Radii

Minimum curb return radii are shown in Table 5C-2.09 below. Where truck traffic is significant, curb return radii should be provided according to the current AASHTO “Green Book;” turning templates are used in this design. The Iowa DOT has an Iowa truck vehicle that can be used to check the proposed radii for truck routes.

Table 5C-2.09: Curb Return Radii Based Upon Roadway Classification

Roadway Classification	Arterial	Collector	Local - Commercial/ Industrial	Local - Residential
Arterial	Special*	Special*	30’	30’
Collector	Special*	30’	30’	25’
Local - Commercial/Industrial	30’	30’	25’	25’
Local - Residential	30’	30’	25’	25’

*Special design required. Use turning templates.

R. Pavement Thickness

Refer to Section 5F-1 for pavement thickness determination and design.

S. References

American Association of State Highway and Transportation Officials (AASHTO). *A Policy on Geometric Design of Highways and Streets* (“Green Book”). Washington, DC. 2004.

American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide*. 3rd ed. Washington, DC. 2006.

Des Moines Area Metropolitan Planning Organization (MPO). *Des Moines Area Daily Directional Capacities At Level of Service D*. Des Moines. 2000.

Asphalt Pavement Mixture Selection

A. Scope

This section is intended for the engineers and technicians who specify asphalt paving material criteria for urban projects, generally ranging from low to medium volume, up to 10M ESALs. Vehicle volumes exceeding 10M $ESAL_{20}$, or projects outside of these design standards, may require more detailed design and/or expert consultation. The section provides a step-by-step process for determining the appropriate mixture criteria and gives the designer additional background information on specific mixture criteria. The section is intended to assist in selecting the mixture criteria that best satisfy the project demands and limitations. Statewide use of this section will improve the standard application of current accepted gyratory mix design technology. In accordance with AASHTO and Iowa DOT Materials I.M. 510, mixture selection involves the use of a 20 year design life whereas pavement thickness design is based on a 50 year design life.

B. Definitions

Equivalent Single Axle Load (ESAL): A standard unit of pavement damage created by a single pass of a vehicle axle.

Car axle = 0.0002 ESAL   18kip truck axle = 1.0 ESAL   24kip truck axle = 3.0 ESAL

$ESAL_{20}$: Estimated cumulative ESALs over a 20 year period.

N: The number of gyratory compaction revolutions at which HMA mixture properties are measured. N_{des} represents 20 years of traffic loading.

Gyratory Mix Design: A laboratory process for achieving desired pavement performance by determining the optimum proportions of aggregates and asphalt binder for hot mix asphalt using a SHRP Superpave gyratory compactor.

Lift Designation (Surface, Intermediate, Base): The terms for the lifts in the hot mix asphalt pavement structure. The surface lift is the top lift, about 1 1/2 inches thick. The intermediate lift(s) is one or more lifts placed under the surface lift, generally 2 to 4 inches thick. The base lift(s) is all mixture placed below the intermediate lift, generally limited to full depth construction.

Modified Asphalt Binders: For design traffic levels greater than 1,000,000 ESALs (High, Very High, and Extremely High), the binders may need to be modified and thus may be more costly.

Nominal Maximum Aggregate Size (NMAS): The mixture size designation used for the combined aggregate gradation. Defined as one sieve size larger than the first sieve to retain more than 10%.

Performance Graded (PG): National asphalt binder grading system, developed by AASHTO, based on high and low pavement operating temperatures (°C). A PG binder is identified using a nomenclature of PG XXYY, followed by an ESAL designation (L, S, H, V, E). The XX is the high pavement temperature in degrees Celsius in which the binder should resist rutting. The YY, in negative Celsius, is the low pavement temperature in which the binder should resist cracking. For example a PG 58-28S should resist rutting to 58 °C and cracking of the pavement to a temperature of -28 °C under standard (0.3 M to 1 M ESALs) traffic loading.

C. Design Checklist

Designers should follow the steps below to ensure that the material criteria selected will best meet the needs of the project and the constraints of the owner agency.

1. **Determine the Level of Traffic Forecasted for the Next 20 Years:** Both current and future traffic levels are needed to determine the appropriate asphalt mixture for the project. Even if the project is not expected to remain in place for 20 years, the material selection levels are based on 20 year values. Common values are average daily traffic (ADT) for the current year, ADT for the 20 year forecast, and percent trucks. In addition to these annualized daily values, the designer should consider potential seasonal high truck volumes, and give particular attention to point sources and future development areas that may generate heavy truck volumes, like quarries, industrial parks, and bus lanes. Seasonal truck volumes may reflect a rate of pavement loading well in excess of the annualized values.

2. **Understand the Pavement Section Design or Rehabilitation Strategy:** In order to make the proper mixture selection, the designer must have knowledge of the proposed pavement construction or rehabilitation and intended pavement performance. The thickness of the pavement will also affect the material and mixture selection. Particular parameters include required structural thickness, existing pavement cross section and condition (dominant distress patterns), traffic patterns and speed, and past maintenance.

3. **Determine the Regional Climate Conditions:** Iowa's 1 day low pavement temperature ranges approximately 5°C from north to south. Adjusted for 98% reliability, the values range from -29 °C to -24 °C. The 7 day high pavement temperature across the state only varies by 3 °C. These values are computed from daily high air temperatures. Adjusted for 98% reliability, the pavement temperature values range from 56 °C to 59 °C. Climate details for a specific location can be obtained from the LTPPB software package available on the FHWA website (http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/pavements/ltpp/dwnload.cfm). See Figures 5D-1.01 and 5D-1.02.

4. **Compute the Anticipated 20 Year Pavement Loading:** The design pavement loading is the starting point for selecting the material and mixture selection criteria. The design pavement loading is measured in ESALs, not ADT. To determine the design ESALs on the project, use the traffic conditions from Step 1 and compute the $ESAL_{20}$. Use the examples outlined in Examples 5D-1.01 and 5D-1.02, for two lane, two way traffic; use Example 5D-1.03 for urban multi-lane situations. Design ESAL levels for asphalt criteria selection are divided into relatively large brackets. While a firm understanding of the traffic and pavement loading is important, good approximations of truck traffic are normally sufficient to determine the design requirements.

5. **Identify Any Special Conditions that Impact the Pavement:** The standard selection process is based on high speed traffic with a broad distribution of vehicle types. There are numerous special conditions that may, through engineering judgement, require changes in the standard pavement materials/mixture selection. These special conditions are outlined below.

   a. **Heavy Trucks:** If the pavement's history has regularly been impacted by heavy trucks, the designer may consider increasing either the binder grade through the designation of a higher design traffic loading, the mix designation (ESAL level), or both. Typical examples of this condition are routes adjacent to quarries, grain elevators, or regional commercial freight distribution centers.

b. **Slow/Stop/Turning:** Urban roadways normally require slower running speeds and often include signed or signaled intersections. The pavement loading condition significantly increases at slower speeds (less than 45 MPH) and stopped vehicles at intersections. The designer may consider increasing the binder grade through the designation of a higher design traffic loading and/or the percent of crushed aggregate to account for this condition. Economics will determine if the higher grade of binder can be applied to the whole project, or just the impacted length of pavement (i.e. intersection and approaches).

c. **Durability:** Many low-volume asphalt pavements are more susceptible to failure due to long term aging than to rutting or fatigue. For pavements with good maintenance histories the designer may want to ensure that the mixture selection will provide adequate durability and, if economically necessary, sacrifice some reliability against rutting or fatigue. This can be accomplished through the selection of a lower compaction level and/or the selection of a softer grade of binder.

d. **Urban vs. Rural:** Separate from the issue of traffic speed, rural projects that pass through urban locations should consider mix sizes (NMAS) that will appeal to the pedestrian traffic. In general, smaller mix sizes will have a better surface appearance than larger mix sizes. The designer can specify smaller mix sizes than those provided in the material selection guide table, but should also consider the availability of the aggregates when making that decision. Similarly, the designer may choose to use a larger mix size on rural sections for the purpose of reducing the asphalt binder content in the mixture.

e. **New Construction vs. Rehabilitation:** The design guide takes into account the major pavement performance factors including rutting, fatigue, and low temperature cracking. When an overlay is placed directly on a slab to be rehabilitated, the existing pavement distress influences the overlay performance and thus the design. If the underlying pavement is PCC or asphalt with thermocracking, the reflective cracking in the overlay will dominate over low temperature cracking so the design parameters related to low temperature cracking for the overlay become less of a factor in the design. If a stress relief layer is included in the overlay design, low temperature cracking should be considered.

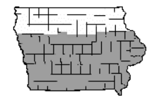
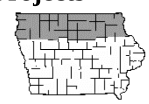
f. **Seasonal Traffic:** Seasonal traffic occurs over a relatively short period of time and may create pavement damage in excess of the normal traffic. For example, grain harvest, Iowa State Fair, festivals, etc. may generate higher volumes (in terms of ESALs) of traffic for a short period of time. This does not only take into account traffic volumes, but also pavement loads.

g. **Mixture Workability:** Smaller mixture sizes are easier to use for hand work.

6. **Select the HMA Mixture Criteria for Each Pavement Layer:** Using the information developed in steps 1 through 5, select the PG binder grade, mixture size, mix design level, and aggregate properties.

a. **PG Asphalt Binder Grade:** The designer should select a binder that nominally satisfies 98% temperature reliability for both the 7 day high pavement temperature and the 1 day low pavement temperature (see 5D-1, C, 3). The designer should select a conventional binder that best satisfies the project conditions. The standard conventional binders in Iowa are PG 58-28S and PG 58-34S. Typically, in urban areas, residential streets will normally use PG 58-28L. For collector and arterial streets, a PG 58-28S or a PG58-28H should be considered. If the designer selects other binder grades to address special considerations, they should recognize the cost impact of specifying non-standard binders.

Table 5D-1.01: Asphalt Binder and Mix

Asphalt Mixture		PG Binder					
Design Traffic (1 x 10^6 ESALs)	**Mix Designation**	**Design Traffic (1 x 10^6 ESALs)**		**Design Speed (MPH)**	**Class I Projects** 		**Class II Projects**
≤ 0.3 M	LT	≤ 0.3 M	and	≤ 45	58-28S	58-34S	58-28S
0.3 M to 1 M	ST	0.3 M to 1 M	and/or	> 45	58-28S	58-34S	58-28S
0.3 M to 1 M	ST	0.3 M to 1 M	and/or	15 to 45	58-28H	58-34H	58-28H
1 to 10 M	HT	1 to 10 M	and/or	15 to 45	58-28H	58-34H	58-28H
> 10 M	VT	> 10 M	or	< 15	58-28V	58-34V	58-28V
> 10 M	ET	> 10 M	and	< 15	58-28E	58-34 E	58-28E

L = Low S = Standard H = High V = Very High E = Extremely High

Class I Projects: Full Depth Hot Mix Asphalt │ HMA + Cold-in-Place Recycling │ HMA + Rubbilization
HMA + Crack and Seat │ HMA Overlay > 4” │ HMA + Full Depth Reclamation (FDR)

Class II Projects: Overlays ≤ 4” │ Parking Lot
Secondary │ Trails

b. **HMA Mixture Size:** Each mixture size (NMAS) is a function of the available aggregates, project conditions, and lift thickness. Minimum lift thickness is a function of density and mixture constructability. The following table shows the minimum lift thickness for the following mix sizes:

Mix Size	Minimum Lift Thickness
3/8”	1”
1/2”	1 1/2”
3/4”	2 1/4”
1”	3”

c. **Mix Design Level:** Based on the projected $ESAL_{20}$ value, seasonal traffic loading and current pavement distress, the designer must select a mix design level. The boundaries of the design levels are not absolute, so the designer should take into consideration the assumptions used to compute the ESAL value.

d. **Aggregate Properties:** The mixture design criteria (Table 5D-1.03) is derived from Iowa DOT Materials I.M. 510. Table 5D-1.03 specifies a 15% increase in percent crushed aggregate for surface and intermediate mixes 1 M ESALs and less to account for slow, stop, and turning conditions. This will be a local decision based on past performance and available aggregates. The actual percent crushed needed to achieve the mix design gyratory compaction volumetrics will vary with the quality of the aggregates used. Both the specified percent crushed and the gyratory compaction volumetrics must be satisfied by the asphalt mixture.

7. **Check for Availability of Materials to Meet the Mix Design Criteria:** Review the mix design criteria selected in step 6 and determine if the binder and aggregates required to meet the mix design criteria are readily available or accessible at a reasonable cost. Contact local producers and/or district materials engineers, if the designer plans to use non-standard criteria. Imported aggregates and modified binders generally cause higher costs. The designer should be ready to justify the mix selection decision.

8. **Place Mix Criteria in the Project Plans and Proposal:** The following information should be placed in the plans and proposal:

   a. **Traffic and $ESAL_{20}$ Projections:** The traffic and $ESAL_{20}$ projections should be listed on the title sheet of the plans. The $ESAL_{20}$ value should coincide with the selected mix design level. If seasonal ESALs are used for design, the title sheet should note that the $ESAL_{20}$ value is based on seasonal loading. The following is an example title sheet.

Traffic	
Current ADT	________
Future ADT	________
Present Trucks	________
$ESAL_{20}$	________

   b. **HMA Mixture:** Each asphalt mixture bid item is defined by the ESAL level, lift designation, and aggregate size. The mixture properties for each mixture level are specified in the specifications and Table 5D-1.03. If the designer specifies a different percent crushed aggregate, this should be identified in the bid item note on the plans. The designer should avoid placing the mix size in additional sections of the plans to minimize errors associated with duplication. The exception to this guide would be a bid item note or tabulation intended to identify locations of different mix sizes for the same lift.

   c. **Asphalt Binder Grade PG XX -YY:** The asphalt binder grade should be specified in the bid item. The designer should avoid placing the binder grade in additional sections of the plans to minimize errors associated with duplication. The exception to this guide would be a bid item note or tabulation intended to identify binder use when multiple binders are specified. The following is an example bid item.

Lift Designation ↓ Mix Size ↓ Binder Grade ↓ Mix Design Level ↓

HMA Standard Traffic (ST) Surface, 3/8", PG 58-28S

D. Material Properties

1. **Typical PG Grades and Their Application:** Two grades (PG 58-28S and PG 58-34S) are common conventional binders used in Iowa. A PG 58-28S asphalt binder will cover the climate range for a majority of projects in Iowa with 98% reliability. The northern tier counties may desire a PG 58-34S for better low temperature protection. Southern counties may consider a PG 58-28H binder for better high temperature properties.

   Some applications utilize specific binder grades. Use PG 58-34E meeting AASHTO T-321 with a minimum of 100,000 cycles to failure for asphalt interlayer applications. Use PG 58-34E meeting AASHTO T-324 with a minimum 90% elastic recovery for high performance thin lift applications.

   When recycled asphalt materials (RAM) are used and they exceed 20% replacement of the total binder, the binder grades may need to be modified. See Iowa DOT Materials I.M. 510.

If warm mix asphalt (WMA) technologies are utilized, the binder grade selection is based on plant mixing temperatures and the level of field compaction. See Iowa DOT Materials I.M. 510 for information on the appropriate binder grade.

2. **Aggregate Source Properties:** Aggregate source properties are defined in Iowa DOT Specifications Section 4127. The mixture criteria listed in Table 5D-1.03 defines the aggregate type for each mixture level specified for the project. Each individual source of aggregate is expected to meet these criteria. The designer may specify a different aggregate type in the bid item note.

3. **Aggregate Consensus Properties:** Aggregate consensus properties are listed in Table 5D-1.03 for each mixture level. These properties include percent crushed aggregate, fine aggregate angularity, clay content (sand equivalent), and flat and elongated particles. These aggregate properties are measured on the combined aggregate, not individual aggregates.

   If the designer specifies a value different from Table 5D-1.03, the value selected should be based on the local practice and desired pavement performance. The asphalt mixture must satisfy both the percent crushed aggregate and laboratory compaction volumetric criteria. The percent crushed aggregate specified is interdependent on the compaction level and the quality of the aggregate.

E. Use of Mixture Selection Guide and Design Criteria Tables

Two tables in Subsection H are provided to assist designers with the selection of asphalt materials for projects. The Asphalt Mixture Selection Guide (Table 5D-1.02) provides the project designer with a set of standard material selections that will satisfy most projects. The Asphalt Mixture Design Criteria (Table 5D-1.03) is derived from Iowa DOT Materials I.M. 510 and provides the mix designer with the detailed mix criteria for each mixture level. The mixture selection guide and mixture design criteria represent the current understanding of accepted asphalt properties for application on urban routes.

The Asphalt Mixture Selection Guide (Table 5D-1.02) represents commonly used mixture parameters, but does not preclude the project designer from deviating from the "recommended" values. The designer should understand the impact of any modification. The first two columns define the standard mixture levels based on traffic loading. The middle columns establish lift thickness and mix size relationships. It should be noted that Table 5D-1.02 does not address required pavement thickness to meet structural needs (Section 5F-1). The Bid Item Designation column ties the mixture levels to the bid items. The final column gives a general statewide guide for the estimated binder content. Local binder content experience may be more appropriate for project estimated quantities. This table does not address the need for special friction aggregate. In general terms, urban routes do not require special friction aggregate.

As mentioned earlier, the Asphalt Mixture Design Criteria (Table 5D-1.03) is derived from Iowa DOT Materials I.M. 510. However, the table differs from I.M. 510. For the surface and intermediate layers of the LT mixes, the amount of crushed aggregate was increased by 15% and for the ST mixes, all layers have an additional 15% crushed aggregate. A different aggregate type and the percent crushed aggregate may be specified by the designer for the project. These values established in the table are prescribed for each mixture and care should be exercised if altered by the project designer. The designer should only change these values when familiar with the material properties and mixture performance for the local area. The bid item plan note must include these values, if it differs from the value in Table 5D-1.03.

F. Example Plans

1. **Title Page:** The traffic and $ESAL_{20}$ projections should be listed on the title sheet of the plans. The $ESAL_{20}$ value should coincide with the selected mix design level. If seasonal ESALs are used for design, the title sheet should note that the $ESAL_{20}$ value is based on seasonal loading.

2. **Typical Section:** Lift thickness should be shown on the typical section. The lift thickness should match or exceed the recommended lift thickness for the mixture size selected, provided compactive requirements are also achieved. The lift should be designated as surface, intermediate, or base. Mixture size or design $ESAL_{20}$ level should not be added to the typical section (it is specified in the bid item).

3. **Bid Items:** Unless otherwise specified, each bid item covers the mixture and binder grade selected. The corresponding bid item note must specify the minimum percent crushed aggregate, if it differs from the value in Table 5D-1.03.

G. Examples for Determination of Traffic ESALs

Similar to pavement thickness design, the asphalt mixture is designed for the frequency and size of the load applied to the pavement. While it is important to have a good understanding of the traffic, it is possible to select the asphalt paving materials based on reasonable approximations. If the designer has actual traffic data, including a distribution of truck types and loads, the current annual ESAL value can be computed from the AASHTO pavement design tables. For most projects however, the designer will determine estimated values based on a general familiarity with the route. The following examples can be used to approximate the design $ESAL_{20}$ for a project.

Example 5D-1.01: Two Lane, Two Way Traffic, Low Volume Street

Step	Task	Values
1	Given: Current AADT Percent Trucks Percent Annual Growth Rate Design Period	1,000 5% 2% 20 years
2	Base Year Design ESALs *[from Section 5F-1, Table 5F-1.08]*	8,000 ESALs
3	Growth Factor *[from Section 5F-1, Table 5F-1.11]*	24.3
4	Compute $ESAL_{20}$ *[8,000 ESALs x 24.3]*	194,400 ESALs
5	Select HMA mixture design level *[from Table 5D-1.02, HMA Mixture Selection Guide]*	**≤ 0.3 M**

Example 5D-1.02: Two Lane, Two Way Traffic, High Volume Street

Step	Task	Values
1	Given: Current AADT Percent Trucks Percent Annual Growth Rate Design Period	10,000 3% 3% 20 years
2	Base Year Design ESALs *[from Section 5F-1, Table 5F-1.08]*	50,000 ESALs
3	Growth Factor *[from Section 5F-1, Table 5F-1.11]*	26.9
4	Compute $ESAL_{20}$ *[50,000 ESALs x 26.9]*	1,345,000 ESALs
5	Select HMA mixture design level *[from Table 5D-1.02, HMA Mixture Selection Guide]*	**1 to 10 M**

Example 5D-1.03: Four Lane Street

Step	Task	Values
1	Given: Current AADT Percent Trucks Percent Annual Growth Rate Design Period	15,000 5% 2% 20 years
2	Base Year Design ESALs *[from Section 5F-1, Table 5F-1.10]*	75,000 ESALs
3	Growth Factor *[from Section 5F-1, Table 5F-1.11]*	24.3
4	Compute $ESAL_{20}$ *[75,000 ESALs x 24.3]*	1,822,500 ESALs
5	Select HMA mixture design level *[from Table 5D-1.02, HMA Mixture Selection Guide]*	**1 to 10 M**

H. Tables and Figures

Table 5D-1.02: Mixture Selection Guide

Design $ESAL_{20}$ (Millions)	Layer Designation	Lift Thickness[3]			Mix Size[1]	Bid Item Designation	Binder Content[2]
		min	*rec*	*max*			
≤ 0.3	Surface	1.5	1.5	2.5	1/2”	Low Traffic (LT)	6.00
	Intermediate	1.5	1.5	3			
	Base	1.5	3	4.5			
0.3 to 1.0	Surface	1.5	1.5	2.5	1/2”	Standard Traffic (ST)	6.00
	Intermediate	1.5	1.5	3			
	Base	1.5	3	4.5			
1.0 to 10.0	Surface	1.5	2	2.5	1/2”	High Traffic (HT)	6.00
	Intermediate	2	2.5	3	3/4”		5.50
	Base	3	4	4.5	1”		5.25

1 The Common mix size is shown. When other mix sizes are used, the minimum lift thickness also changes (see Section 5D-1, C, 6, b).
2 These values are for estimating quantities only. The actual asphalt binder content is established in the approved job mix formula.
3 Some lift thickness values in this guide may conflict with traffic control or allowable compaction criteria.

Table 5D-1.03: Mixture Design Criteria
(derived from Iowa DOT Materials I.M. 510)

Mix	Layer Designation	Gyratory Density		Film Thickness	Aggregate[2]			
		N_{des}	Design % G_{mm} (target)		Quality Type	Crush (min)	FAA (min)	Sand Equivalent (min)
LT	0.3 M S	50	96.0	8.0 - 15.0	A[1]	60[1]	---	40
	0.3 M I							
	0.3 M B		97.0		A[1]	45		
ST	1M S	50	96.0	8.0 - 15.0	A	75[1]	40	40
	1M I				A[1]	60[1]		
	1M B		97.0				---	
HT	10M S	75	96.0	8.0 - 15.0	A	75	43	45
	10M I							
	10M B		96.5		A[1]	60		
For mix design levels exceeding 10M ESALs, see Iowa DOT Materials I.M. 510.								

1 Requirements differing from Iowa DOT Materials I.M. 510; for base mixes, aggregate quality improved from B to A and percent crushed aggregate increased by 15%.

2 Flat & Elongated 10% maximum at a 5:1 ratio

Figure 5D-1.01: High (7 Day) Pavement Temperature at 98% Reliability

Figure 5D-1.02: Low (1 Day) Pavement Temperature at 98% Reliability

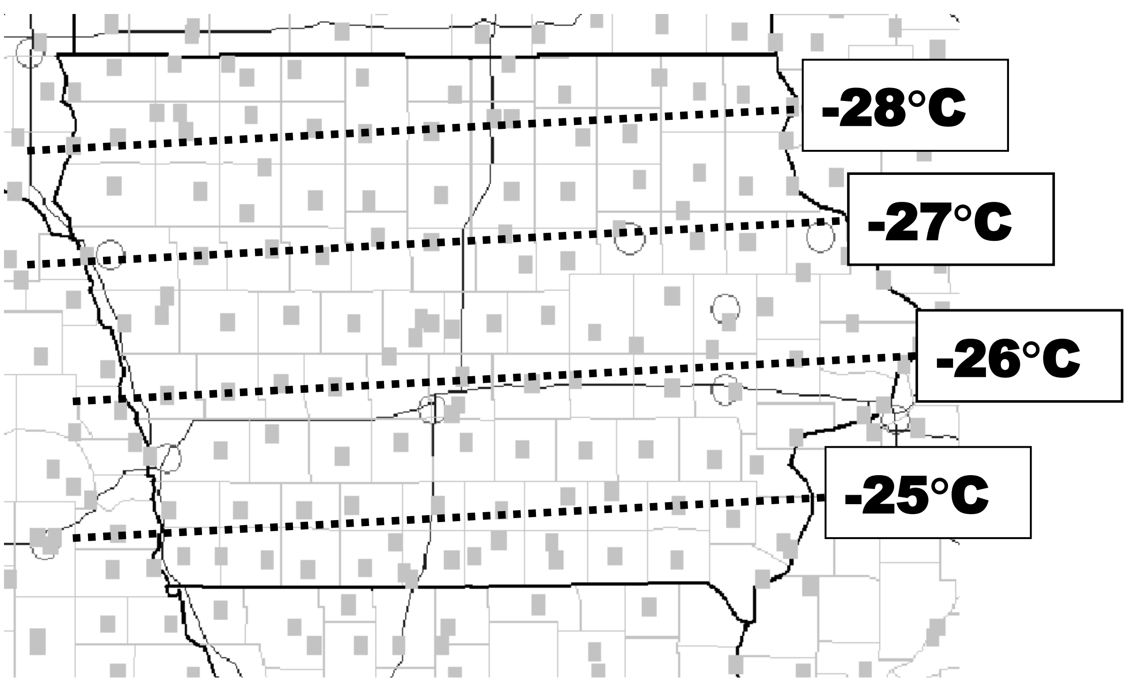
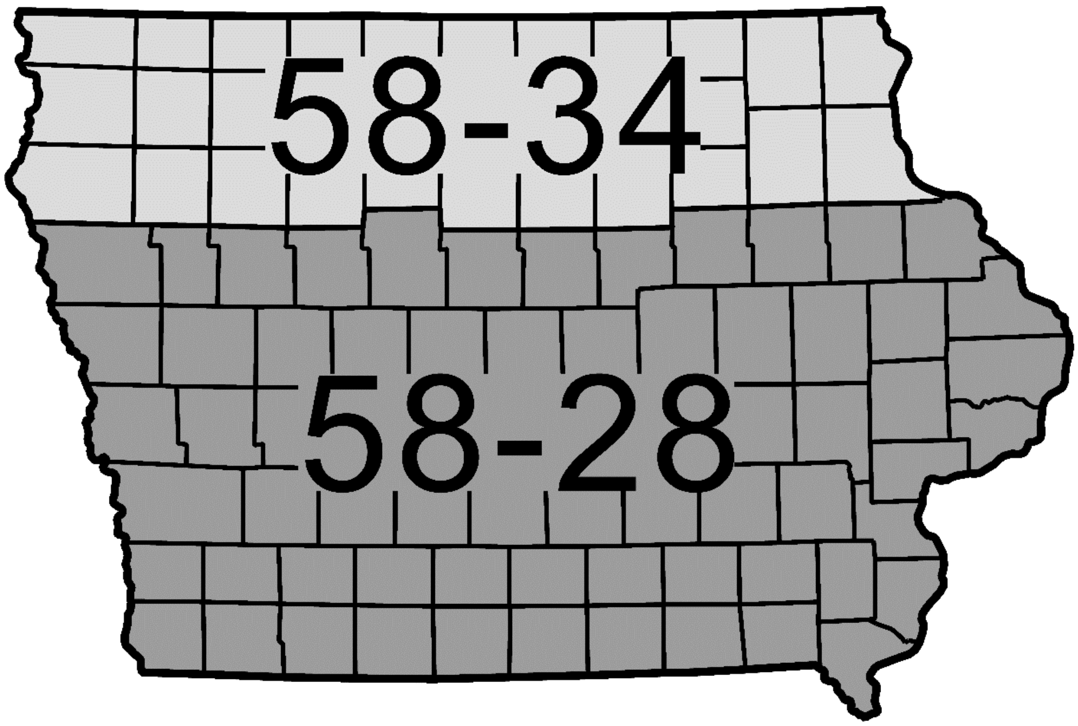


Figure 5D-1.03: Standard Binder Grades

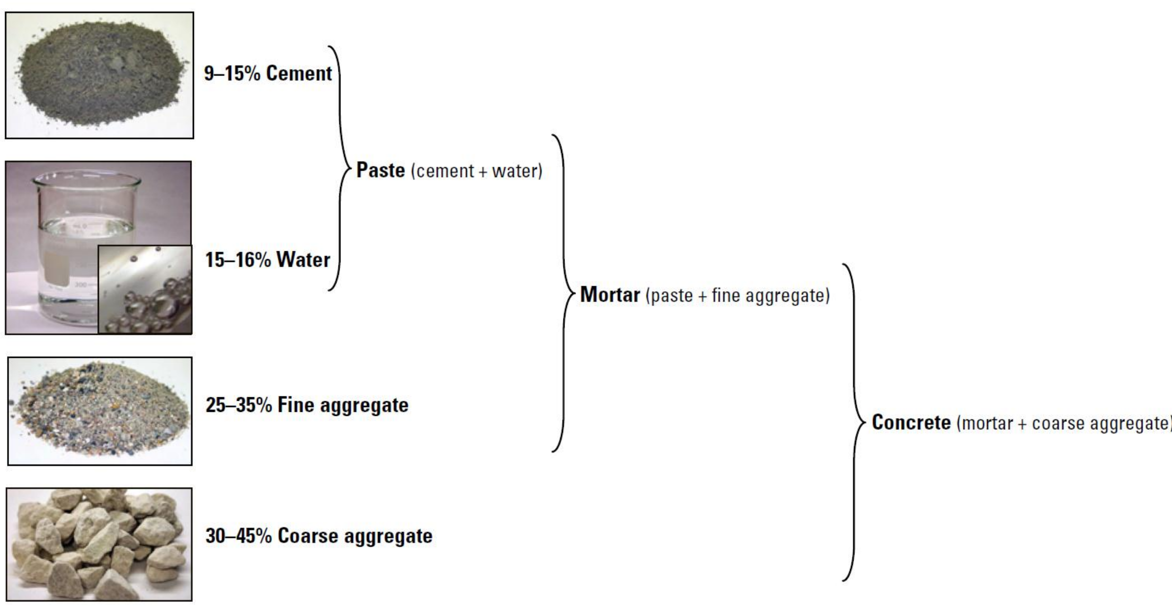


PCC Pavement Mixture Selection

A. General Information

Concrete is basically a mixture of two components, paste and aggregates. Cement and water form the paste, which binds the aggregates, usually sand and gravel or crushed stone, into a solid rocklike mass. The paste hardens due to a chemical reaction of the cement and water, known as hydration. In addition to the basic ingredients, supplementary cementitious materials (SCMs) and chemical admixtures may be included in the paste. This section will introduce the pavement designer to the PCC mixture components and their characteristics and behaviors so the optimum mixture selection for concrete pavements can be determined. It should be noted, the SUDAS Specifications reference the Iowa DOT Specifications for concrete mix materials, design, and proportions.

Figure 5E-1.01: Concrete is Basically a Mixture of Cement, Water/Air, and Aggregates (percentages are by volume)



Sources: Taylor et al, 2006

B. Cementitious Materials

Cementitious materials are classified as either hydraulic cements or pozzolans. The difference between the two is their reaction when mixed with water.

1. **Hydraulic Cements:** Hydraulic cements chemically react with water through a process called hydration. The compounds produced during hydration affect the setting, hardening, and strength gains of hydraulic cement mixtures. The hydration process occurs until the hardening of the concrete is complete and the strength gains have ceased. Portland cement, the most common type of hydraulic cement, contains hydraulic calcium silicates, calcium aluminates, calcium aluminoferrites, and calcium sulfate (gypsum). The reaction of the water being in contact with

the hydraulic cement produces a byproduct of calcium silicate hydrate (C-S-H) and calcium hydroxide (CH). Blended cements, a manufactured blend of Portland cement and one or more supplementary cementitious materials (SCMs), are also hydraulic cements.

a. Types of Hydraulic Cements: As outlined by ASTM C 1157, a performance specification, there are six types of hydraulic cements.

1) **Type GU:** General use
2) **Type HE:** High early strength
3) **Type MS:** Moderate sulfate resistance
4) **Type HS:** High sulfate resistance
5) **Type MH:** Moderate heat of hydration
6) **Type LH:** Low heat of hydration

If an “R” is added after the type name, HE-R, it denotes low reactivity with alkali-reactive aggregates.

b. Types of Portland Cement: There are five different types of Portland cements that are required to meet the specifications of ASTM C 150/AASHTO M 85.

1) **Type I:** Normal
2) **Type II:** Moderate sulfate resistance
3) **Type III:** High early strength
4) **Type IV:** Low heat of hydration
5) **Type V:** High sulfate resistance

c. Types of Blended Cements: Blended hydraulic cements are a combination of two or more types of fine materials. They can be used the same way as Portland cements. Typical materials that are combined are Portland cement and SCMs, including ground granulated blast-furnace slag (GGBF slag), fly ash, silica fume, calcined clay, pozzolans, or hydrated lime. These combinations that make blended hydraulic cements must conform to ASTM C 595 requirements. The two main classes of ASTM C 595 cements are:

1) **Type IS (X):** Portland blast-furnace slag cement
2) **Type IP (X):** Portland-pozzolan cement

The “X” stands for a percentage of the SCM included in the blend. Type IS (40) contains 40% by mass of slag.

d. Selection of Cement: The selection of cement is important when considering which type to use on the job. The main aspect to consider when selecting which cement is right for the job is the availability of cements. Types I and II are available almost everywhere, where Types III, IV, and V are less common in certain areas. Blended cements are available almost everywhere. The Iowa DOT and SUDAS Specifications allow Types I and II cements to be used in pavements, in addition to Types IP and IS blended cements.

2. Pozzolans: Pozzolans do not react when solely mixed with water; they require a source of calcium hydroxide (CH) to hydrate. The most common source of CH comes from the hydration of hydraulic cements, which produces both calcium silicate hydrates (C-S-H) and CH (a less desirable product). When combined with water and CH, pozzolans form additional C-S-H. This additional C-S-H contributes to concrete strength and impermeability of the cement mixtures. Common pozzolans include fly ash, silica fume, and natural pozzolans such as calcined clay, calcined shale, and metakaolin.

Table 5E-1.01: Cementitious Materials

Cementitious Materials		
	Hydraulic cements	**Pozzolans (or materials with pozzolanic characteristics)**
	Portland cement Blended cement	
Supplementary cementitious materials	GGBF slag Class C fly ash	GGBF slag Class C fly ash Class F fly ash Natural pozzolans (calcined clay, calcined shale, metakaolin) Silica fume

Simple Definitions

Cement (hydraulic cement)—material that sets and hardens by a series of nonreversible chemical reactions with water, a process called hydration.

Portland cement—a specific type of hydraulic cement.

Pozzolan—material that reacts with cement and water in ways that improve microstructure.

Cementitious materials—all cements and pozzolans.

Supplementary cementitious materials—cements and pozzolans other than portland cement.

Blended cement—factory mixture of portland cement and one or more SCM.

Source: Taylor et al, 2006

C. Supplementary Cementitious Materials

Supplementary Cementitious Materials (SCMs) are a common addition to the mix in modern concrete mixtures. SCMs can contribute to the concrete through either hydraulic or pozzolanic activity or both. For example, GGBF slags are hydraulic materials, and Class F fly ashes are typically pozzolanic. Class C fly ash has both hydraulic and pozzolanic characteristics. There are four main types of SCMs are fly ash, ground granulated blast furnace slag (GGBF slag), natural pozzolans, and silica fume.

Fly ash is the most commonly used SCM and includes two types, Class C and Class F. Substituting fly ash in concrete mixes can reduce the amount of water required for workability but delays the setting time of the concrete. The addition of fly ash causes a slower but longer reaction rate in the concrete. As a result, the heat of hydration is reduced, and the setting time of the mix is delayed.

1. **Effects of SCMs:** The addition of SCMs to a concrete mixture affects a wide variety of properties for the concrete. Tables 5E-1.02 and 5E-1.03 indicate the effects of SCMs on fresh and hardened concrete properties. The modified properties include the following:

   a. **Fresh Properties:** Fly ash and GGBF slag increase the workability of the concrete, while silica fume can reduce workability at concentrations greater than 5%.

   b. **Durability/Permeability:** SCMs generally increase the durability of the concrete by reducing the permeability of the concrete mix. As a result, the concrete is less susceptible to chloride penetration. The quality of the SCMs and the work practices of the contractor are important to realize these benefits.

c. **Resistance:** Alkali-silica reactivity can be controlled with SCMs. The optimum dosage of fly ash has proven to reduce reactivity. Silica resistance is also improved with the addition of SCMs by reducing the reactive elements that contribute to expansive sulfate reactions. Class F fly ash is more effective than Class C, and GGBF slag is beneficial in sulfate environments. SCM content in concretes subject to freezing should not exceed 50%; however, they are still durable if over 50% is used.

Table 5E-1.02: Effects of SCMs on Fresh Concrete Properties

	Fly ash				Natural pozzolans		
	Class F	Class C	GGBF slag	Silica fume	Calcined shale	Calcined clay	Metakaolin
Water requirements	↓↓	↓↓	↓	↑↑	↔	↔	↑
Workability	↑	↑	↑	↓↓	↑	↑	↓
Bleeding and segregation	↓	↓	↕	↓↓	↔	↔	↓
Air content	↓↓ *	↓ *	↓	↓↓	↔	↔	↓
Heat of hydration	↓	↕	↓	↔	↓	↓	↓
Setting time	↑	↕	↑	↔	↑	↑	↔
Finishability	↑	↑	↑	↕	↑	↑	↑
Pumpability	↑	↑	↑	↑	↑	↑	↑
Plastic shrinkage cracking	↔	↔	↔	↑	↔	↔	↔

Sources: Thomas and Wilson (2002b); Kosmatka, Kerkhoff, and Panarese (2003)

* Effect depends on properties of fly ash, including carbon content, alkali content, fineness, and other chemical properties.

Key:
↓ reduced
↓↓ significantly reduced
↑ increased
↑↑ significantly increased
↔ no significant change
↕ effect varies

Source: Taylor et al, 2006

Table 5E-1.03: Effects of SCMs on Hardened Concrete Properties

	Fly ash				Natural pozzolans		
	Class F	Class C	GGBF slag	Silica fume	Calcined shale	Calcined clay	Metakaolin
Early strength	↓	↔	↓	↑↑	↓	↓	↑↑
Long-term strength	↑	↑	↑	↑↑	↑	↑	↑↑
Permeability	↓	↓	↓	↓↓	↓	↓	↓↓
Chloride ingress	↓	↓	↓	↓↓	↓	↓	↓↓
ASR	↓↓	↕	↓↓	↓	↓	↓	↓
Sulfate resistance	↑↑	↕	↑↑	↑	↑	↑	↑
Freezing and thawing	↔	↔	↔	↔	↔	↔	↔
Abrasion resistance	↔	↔	↔	↔	↔	↔	↔
Drying shrinkage	↔	↔	↔	↔	↔	↔	↔

Sources: Thomas and Wilson (2002b); Kosmatka, Kerkoff, and Panarese (2003)

Key:
↓ reduced
↓↓ significantly reduced
↑ increased
↑↑ significantly increased
↔ no significant change
↕ effect varies

Source: Taylor et al, 2006

2. **Limitations on the Use of SCMs:** Table 5E-1.04, which is adapted from ACI 218, provides recommended maximum amounts of SCMs for concrete exposed to deicing chemicals, such as Iowa concrete pavements. The Iowa DOT and SUDAS Specifications limit the usage of SCMs below the ACI maximum amounts. By those specifications, the maximum allowable fly ash substitution rate is 20%, and the GGBF slag substitution rate is limited to no more than 35% by weight (mass). The total mineral admixture substitution rate cannot exceed 40%. When Type IP or IS cement is used in the concrete mixture, only fly ash substitution is allowed. Substitution of Type I/II cement with both GGBF slag and fly ash is only allowed in ready mix concrete mixtures.

Table 5E-1.04: Cementitious Materials Requirements for Concrete Exposed to Deicing Chemicals

Cementitious Materials*	**Maximum Percent by Total Cementious Materials by Mass****	
	ACI Values	***Iowa Values***
Fly ash and natural pozzolans	25	20
GGBF slag	50	35
Silica fume	10	0
Total of fly ash, GGBF slag, silica fume, and natural pozzolans	50***	40
Total of natural pozzolans and silica fume	35***	20

* Includes portion of supplementary cementitious materials in blended cements.
** Total cementitious materials include the summation of portland cements, blended cements, fly ash, slag, silica fume, and other pozzolans.
*** Silica fume should not constitute more than 10% of total cementitious materials and fly ash or other pozzolans must not constitute more than 25% of cementitious materials.

Source: Taylor et al, 2006

D. Aggregates

Aggregates account for 60% to 75% of concrete by volume and are seldom susceptible to moisture and chemical changes, making them an important ingredient in concrete mixtures. Aggregates influence the concrete's freshly mixed and hardened properties, mixture proportions, and economy. They must be durable and free of any absorbed materials, clay, and materials that effect the interaction with the cement.

1. **Types of Aggregates:**
   a. **Carbonate Rock:** Mainly limestone and dolomite with low porosity and low absorption rate.
   b. **Granite:** Igneous rocks composed mainly of silica and silicates with the highest modulus of elasticity of any rock type available.
   c. **Gravel and Sand:** Typically mixtures of many minerals and rocks. Gravel and sand from shale, siltstone, or unsound material rich rocks tend to be unsound and not recommended. Sand and gravel from higher elevations and that have been smoothed by water are best.
   d. **Manufactured Aggregates:** Produced by crushing rocks into smaller pieces. Least likely to be contaminated, but mixtures with manufactured aggregates tend to be harder to work with and require more water.
   e. **Recycled Aggregates:** Made from crushing concrete pavement and mixed with new aggregates. Typically has a higher absorption rate.

The following aggregate properties are important to consider when mixing concrete: gradation, durability, particle shape, surface texture, absorption, coefficient of thermal expansion, and resistance to freezing and thawing

2. **Gradation:** Gradation is a measure of the size distribution of aggregate particles, determined by passing aggregate through sieves of different sizes (ASTM C 136 / AASHTO T 27). Grading is most commonly shown as the percentage of material passing sieves with designated hole sizes. Aggregates are classified as coarse or fine by ASTM C 33/AASHTO M 6/M 80 as follows:

   a. **Coarse Aggregate:** Coarse aggregate consists of gravel, crushed gravel, crushed stone, or crushed concrete that is retained on the No. 4 sieve. The maximum size of a coarse aggregate is generally 3/8 inch to 1 1/2 inch.
      1) Coarse aggregate requirements allow a wide range in selection.
      2) If the proportion of fine aggregate to total aggregate produces concrete of good workability, the grading for a given maximum size coarse aggregate can be varied moderately without appreciably affecting a mixture's cement and water requirements.
      3) Coarse aggregate size is limited by local availability, the maximum fraction of the minimum concrete thickness or reinforcing spacing, and the ability of the equipment to handle the concrete.

   b. **Fine Aggregate:** Fine aggregate consists of natural sand, manufactured sand, or combinations of the two that pass the No. 4 sieve. Very fine particles (passing the No. 100 sieve) are limited by specifications because they have extremely high surface-to-volume ratios that require more paste.
      1) A relatively wide range in fine aggregate gradation is allowed.
      2) If the water to cementitious materials (w/cm) ratio is kept constant and the ratio of fine to coarse aggregate is chosen correctly, a wide range in grading can be used without a measurable effect on strength.
      3) Generally, increasing amounts of fine material will increase the water demand of concrete.

c. **Well-graded Aggregate:** It is important to maximize the amount of aggregate in concrete mixtures because they are more chemically and dimensionally stable than cement paste. This is done by selecting the best aggregate grading for the job.
   1) Aggregates with a variety of sizes are optimum because smaller particles fill the voids between larger particles, maximizing the aggregate volume.
   2) Aggregate size and grading is important when trying to achieve the preferred water content. Smaller aggregates require more paste because of the high surface to volume ratios, and vice versa.
   3) Mixtures with properly graded aggregates tend to have less permeability and shrinkage, will be easier to handle, and will be the most economical.

d. **Combined Aggregate Grading:** The most important grading in a concrete mixture is the combined aggregate, utilizing the coarse and fine aggregates. Aggregates with a smooth grading curve will generally provide better performance than a gap-graded system.
   1) Research on air-entrained concrete has indicated the w/cm ratio could be reduced by more than 8% using combined aggregate gradation.
   2) If problems develop due to a poor gradation, consider using alternative aggregates, blending aggregates, or conducting a special screening of existing aggregates.

3. **Durability:** Aggregates containing minerals (see Table 3-12 in *Design and Control of Concrete Mixtures*) can react with alkali hydroxides and expand when exposed to moisture, cracking the concrete. Some rock types are potentially susceptible to alkali-silica reactivity (ASR) (shown in Table 3-13 from *Design and Control of Concrete Mixtures*). ASR and alkali carbonate reactivity can be avoided by expansion tests and petrography. Aggregates with coarse surfaces may be susceptible to freeze-thaw damage and cause D-cracking, damaging the concrete. The abrasion resistance of an aggregate indicates the quality; the higher the resistance, the higher the quality. The maximum percent of abrasion allowed for crushed stone is 50% and for gravel is 35% by the Iowa DOT and SUDAS Specifications, as determined according to AASHTO T 96.

   a. **Durability Determination:** Durability of aggregates allowed for use in pavements by the Iowa DOT and SUDAS Specifications is based on service history; geologic correlation; and testing, including abrasion, freeze-thaw, and objectionable materials.

   b. **Durability Classes:** Based on the durability determination, aggregates are designated as follows.
      1) Class 2 Durability: No deterioration of pavements of non-interstate segments of the road system after 15 years and only minimal deterioration in pavements after 20 years of age. Class 2 is the default durability requirement by the SUDAS Specifications for aggregates used in mixes for urban pavements.
      2) Class 3 Durability: No deterioration of pavements of non-interstate segments of the road system after 20 years of age and less than 5% deterioration of the joints after 25 years.
      3) Class 3i Durability: No deterioration of pavements of the interstate road system after 30 years of service and less than 5% deterioration of the joints after 35 years.

4. **Particle Shape:** Aggregate shapes are described as either cubic, flat, or elongated. The use of flat and elongated particles should be limited to 15% the total mass of aggregate in an effort to reduce water demands. Rough textured, angular, and elongated particles require more water to make concrete mixtures smooth and workable. Angular aggregates have higher flexural and compressive strengths along with a higher skid resistance.

5. **Surface Texture:** Different textures may be used in a mixture as long as the mixture is properly proportioned with the varying textures. Rough textures are advantageous because of better

bonding and interlocking. If coarse surfaces are used, the particle sizes should be reduced and drainage should be improved around the base.

6. **Absorption:** It is important to know what state of moisture the aggregates are in during batching so that the w/cm ratio can be adjusted.

   a. Wet aggregates contribute undesired moisture.

   b. Saturated surface dry - neither absorbing water from nor contributing water to the concrete mix - is the ideal moisture level.

   c. An increased w/c ratio increases shrinkage and reduces strength.

   d. Aggregates with high absorption values result in variations of concrete quality.

7. **Coefficient of Thermal Expansion (CTE):** Similar to freeze-thaw resistance, an aggregate's CTE is how much it changes in size during temperature changes. Low CTE values are desirable because their size changes the least and tend to crack less. In Iowa, CTE is generally not a problem because of the prevalent use of limestone aggregate, which has a low CTE as illustrated in Table 5E-1.05.

Table 5E-1.05: Typical CTE Values for Common PCC Ingredients

	Coefficient of Thermal Expansion 10^{-6}/°F
Aggregate	
Granite	4 to 5
Basalt	3.3 to 4.4
Limestone	3.3
Dolomite	4 to 5.5
Sandstone	6.1 to 6.7
Quartzite	6.1 to 7.2
Marble	2.2 to 4
Cement paste (saturated)	10 to 11
Steel	6.1 to 6.7

Note: These values are for aggregates from specific sources, and different aggregate sources may provide values that vary widely from these values.

Source: Taylor et al, 2006

8. **Freeze-thaw Resistance:** The freeze-thaw resistance of aggregates is related to its porosity, absorption, permeability, and pore structure. If too much water is absorbed and the concrete aggregates freeze, the expanding aggregates can potentially destroy the concrete. This degradation of the concrete aggregate is known as D-cracking (see photo below). D-cracking can be reduced by selecting aggregates that have a better freeze-thaw resistance, reducing the maximum particle size, and by installing an effective drainage system to pull water out from underneath the pavement. The use of higher quality aggregates is recommended to increase freeze-thaw resistance. As noted, Class III limestone aggregates will provide greater freeze-thaw resistance than Class II. Some gravel sources will also outperform the Class II aggregates, but the actual freeze-thaw resistance should be verified prior to use.

E. Chemical Admixtures

Any ingredient other than portland cement, supplementary cementitious materials, water, and aggregates is considered an admixture. There are eleven different types of chemical admixtures, the four most common are air-entraining admixtures, water-reducing admixtures, retarding admixtures, and accelerating admixtures. Reasons for using admixtures are to reduce the cost of concrete construction, assist in construction operations, obtain certain properties in concrete, and maintain the quality of concrete over longer periods of time. Admixtures are used to complement acceptable cementing practices and should not be used to substitute them.

1. **Air-entraining Admixtures:** Air-entraining admixtures are the most common type of admixture used. Air-entraining admixtures have the ability to control and entrap air bubbles in concrete, providing the user with a more durable and workable concrete. These admixtures affect concrete by improving freeze-thaw resistance, increasing workability, improving deicer resistance, and reducing sulfate and alkali reactivity.

   Keep in mind for every 1% entrained air, concrete loses about 5% of its compressive strength. The Iowa DOT and SUDAS Specifications require air content of the unconsolidated concrete ahead of the paver to be 8% ± 2%, with the goal to have a minimum of 5% air entrained in the hardened concrete.

2. **Water-reducing Admixtures:** Water-reducing admixtures are implemented to control and reduce the amount of water in a concrete mixture. They typically reduce water content 5% to 10% by sacrificing the reduction of slump. However, strength is increased anywhere from 10% to 25% because of the reduction in the w/cm ratio, and concrete with water-reducing admixtures tend to have good air retention.

3. **Retarding and Accelerating Admixtures:** Retarding admixtures delay the setting time of concrete; therefore decreasing the rate of slump loss and extending the workability of the concrete. The bleeding capacity and rate of concrete is also increased. Retarding admixtures allow more time to place concrete on difficult jobs, allow for special finishing techniques, and offset the adverse effects hot weather has on setting time.

   Accelerating admixtures are used to accelerate the setting of concrete. The accelerating admixtures speed up the hydration process, setting, and strength gains at early ages. Calcium Chloride ($CaCl_2$) is the most commonly used accelerating admixture. It should be added as part of the mixing water.

F. Water

Any drinkable, potable water may be used as mixing water for concrete. ASTM C 1602 provides specifications for using mixing water in concrete mixtures. Sources of water in a concrete mixture come from batch water, ice (if used during high-temp weather), free moisture on aggregate, and water in admixtures.

Any non-potable water may have adverse effects on the strength and set time of the concrete and should be tested for strength, setting time, alkali levels, sulfate levels, chloride levels, total solids, and corrosion of reinforcements

G. Air-entrainment

Air-entrained concrete is used to improve freeze-thaw resistance when exposed to water and deicing chemicals. Air-entrained concrete is produced by using air-entrained cement or by adding an air-entraining admixture. The Iowa DOT and SUDAS Specifications require air content of the unconsolidated concrete ahead of the paver to be 8% ± 2%. The goal is to have a minimum of 5% entrained air in the hardened concrete.

1. **Benefits:** The primary benefits of air-entrained concrete include the following.

   a. Significant improvement of freeze-thaw and deicer-scaling resistance
      1) Air bubbles are created during the entrainment process. The air bubbles allow the pressure of freezing water to be released in air voids instead of in the concrete, which would eventually destroy it. Figure 11-16 in *Design and Control of Concrete Mixtures* demonstrates the improved durability of air-entrained concrete.
      2) Air voids on the surface of the concrete relieve pressure buildups and reduce surface scaling that have detrimental effects on the life of the concrete.

   b. Higher resistance to sulfate and alkali-silica reactivity

   c. Improved workability

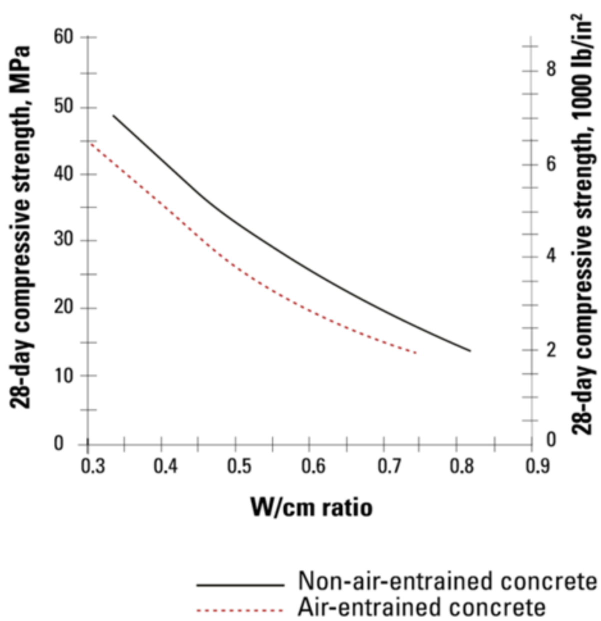
   d. Reduced segregation and bleeding in freshly mixed concrete

2. **Factors that Affect the Air Content:**
   - Cement
   - Aggregates
   - Mixing water
   - Slump
   - Temperature
   - Supplementary cementitious materials
   - Admixtures
   - Mixing acting
   - Transportation and handling
   - Finishing techniques

H. Slump

The slump of concrete is an indicator of the workability and a measurement of concrete consistency. Slump is not an indicator of the quality of the concrete. The slump of concrete is affected by changes in aggregates, cements, admixtures, water, and air. Workability, along with air content, is one of the primary concrete properties that can be manipulated during the process. Keep in mind that adding water to the mix will increase the w/cm ratio and lower the strength of the concrete. The SUDAS Specifications require that slump be no less than 1/2 inch or no more than 2 1/2 inches for machine finish and no less than 1/2 inch and no more than 4 inches for hand finish. Figure 5E-1.02 is an example of the strength values for mixes with differenct w/cm ratios; the higher the w/cm ratio, the lower the strength.

Figure 5E-1.02: Effect of w/cm Ratio on Strength of Concrete



I. Concrete Mixtures

1. **Concrete Mix Design and Mix Proportioning:** The terms mix design and mix proportioning are often incorrectly used interchangeably.

   a. **Mix Design:** Mix design is the process of determining required and specifiable properties of a concrete mixture, i.e., concrete properties required for the intended use, geometry, and exposure conditions. Workability, placement conditions, strength, durability, and cost should be considered in concrete mix designs.

   b. **Mix Proportioning:** Mix proportioning is the process of determining the quantities of concrete ingredients for a given set of requirements. The objective of proportioning concrete mixtures is to determine the most economical and practical combination of readily available materials to produce a concrete that will have the required properties.

2. **Concrete Mix Specifications:** There are two types of concrete mix specifications, prescriptive and performance mixes. With prescriptive mixes, the materials, proportions, and construction methods are specified, and satisfactory performance is anticipated. If performance requirements are utilized, functional requirements, such as strength, durability, and volume changes are specified, and the contractor and concrete producer are expected to develop concrete mixtures that meet those requirements.

   a. **SUDAS Concrete Mix Design Specifications:** The SUDAS Specifications for PCC pavement refer to the Iowa DOT's PCC pavement specifications and materials instructional memorandums (I.M.s) for concrete mix designs, proportions, and materials. Iowa DOT mixes are prescriptive and include specifications for concrete mix materials, proportions, and construction methods. There are no concrete performance requirements, and satisfactory performance is expected if all specifications are followed.

   b. **SUDAS Concrete Mix Material Specifications:** SUDAS references the Iowa DOT Specifications for concrete mix material, including the following.

Material	Iowa DOT Specifications Section	Iowa DOT Materials I.M.
Cement	4101	401
SCMs		
Fly Ash	4108	491.17
GGBF Slag	4108	491.14
Fine Aggregate	4110	409 (Source)
Coarse Aggregate (3 gradations)	4115	409 (Source)
Water	4102	
Admixtures		
Air entrainment	4103	403
Retarding and water reducing	4103	403
Accelerating (calcium chloride)	2529.02	403

c. SUDAS Concrete Mix Proportioning Specifications: The concrete mixes currently used in Iowa were developed in the 1950s. Classes A, B, and C were specified for concrete paving. As originally developed, Classes A and B, with minimum design compressive strengths of 3,500 psi and 3,000 psi respectively, were utilized for rural county paving. Class C concrete, with a higher compressive strength of a minimum of 4,000 psi and a w/cm ratio of less than 0.45, was the standard for primary roads. With its history of proven performance, Class C concrete is now the standard for all concrete road paving in Iowa. In areas where early opening strength is desired, such as intersections and driveways, an M mix can be substituted for C mix. M mix has a higher cement content, which accelerates the heat of hydration and set time of the concrete.

Unless the designer otherwise specifies, the contractor can choose any of the Iowa DOT Class C mixes and the materials that are allowed within the specifications. Generally, economy, workability, and availability of materials are key factors in the decision making process of the contractor and the concrete supplier.

Iowa DOT Materials I.M. 529 establishes the mix proportions for the various concrete mixes used by the Iowa DOT and SUDAS. Each mixture has specific requirements for the coarse and fine aggregates as well as the type of cement, including SCMs. The mix proportions include unit volumes for all materials.

1) **Mix Designation:**
   Example: C-4WR-S35
   - The first letter indicates the class of concrete
   - The first number indicates the percentages of fine aggregate and coarse aggregate
     - 2 is composed of 40% fine and 60% coarse
     - 3 is composed of 45/55
     - 4 is composed of 50/50
     - 5 is composed of 55/45
     - 6 is composed of 60/40
     - 7 is composed of 65/35
     - 8 is composed of 70/30
     - 57 is composed of 50/50
   - The WR indicates water reducer is used in the mixture
   - SCMs are then indicated with their percentage of cementitious material substitution. C and F fly ashes are indicated with a C and F, respectively. GGBF slag is indicated with an S. The percentage of substitution is indicated after the SCM letter.
   - The example designates a Class C concrete mix, a combined aggregate composed of 50% fine aggregate and 50% coarse aggregate, water reducer admixture, and 35% GGBF slag cementitious material substitution.
2) **Mix Proportions:** Iowa DOT Materials I.M. 529 provides material proportioning for the various Iowa DOT concrete mixes and includes basic absolute volumes of cement, water, air, and fine and coarse aggregate per unit volume of concrete (cy/cy). Target and maximum w/cm ratios are provided for each of the mix classes. Also included is guidance for calculation of fly ash and GGBF slag cementitious material substitution of cement.
3) **Admixtures:** Sources of Iowa DOT approved are provided in Iowa DOT Materials I.M. 403, along with their maximum dosages. Generally, the maximum dosages are as recommended by the manufacturers. Do not exceed the maximum dosages unless further consulted with the manufacturer and the Engineer.

3. **Modification of the Standard Concrete Mix Specifications:** While care should be exercised, achieving the required properties in the concrete may require making adjustments to the materials selected, to materials proportions, or even to other factors such as temperature, as follows.

   a. **Workability:** Water content, proportion of aggregate and cement, aggregate properties, cement characteristics, admixtures, and time and temperature can be adjusted to achieve the desired workability. The slump test (ASTM C 143 / AASHTO T 119) is most often used to measure the workability of fresh concrete.

   b. **Stiffening and Setting:** The rates of stiffening and setting of a concrete mixture are important because they affect its ability to be placed, finished, and sawed. Stiffening and setting can be affected by the following in the concrete mixture: cementitious materials, chemical admixtures, aggregate moisture, temperature, and water-cementitious materials (w/cm) ratios.

   c. **Bleeding:** Techniques can be used to prevent and minimize bleeding. These techniques (Kosmatka 1994) include reducing the water content, w/cm, and slump; increasing the amount of cement or supplementary cementitious materials in the mix; increasing the fineness of the cementitious materials; using properly graded aggregate; and using certain chemical admixtures such as air-entraining agents may reduce bleeding.

   d. **Air-void System:** The air-void system is important to concrete durability in environments subject to freezing and thawing. It includes total air content, spacing factors, and specific surface. The air-void system can be controlled with cement, supplementary cementitious materials, aggregates, and workability. The air-void system in the field will be affected by changes in the grading of the aggregate, water, admixture dosage, delays, and temperature.

   e. **Density:** Conventional concrete used in pavements has a density in the range of 137 to 150 lb/yd^3. Density varies depending on the amount and density of the aggregates, the amount of entrained air, the amount of water, and the cement content. Density is affected by the following factors: density of the material in the mixture, mostly from coarse aggregates; moisture content of the mixture; and relative proportions of the materials, mainly water.

   f. **Strength:** Strength and rate of strength gain are influenced by water-cementitious materials ratio, cement chemistry, SCMs, chemical admixtures, aggregates, and temperature. Changes in the environmental conditions and variation in materials, consolidation, and curing affect the strength at a specified age and affect strength development with age. Increased temperatures will increase early strength but may decrease long-term strength gain.

   g. **Volume Stability:** Concrete experiences volume changes as a result of temperature and moisture variations. To minimize the risk of cracking, it is important to minimize the tendency to change in volume by considering paste content, aggregates, and curing.

   h. **Permeability and Frost Resistance:** Permeability is a direct measure of the potential durability of a concrete mixture. Lower permeability is achieved by the following factors.
      - Increasing the cementitious materials content
      - Reducing the water-cementitious materials ratio
      - Using supplementary cementitious material at dosages appropriate to the expected likelihood of freezing water
      - Using good curing practices
      - Using materials resistant to the expected form of chemical attack

- Using aggregates that have proven to resist D-cracking. Reducing maximum coarse aggregate size will reduce the risk of damage if aggregates prone to damage are unavoidable.
- Ensuring that a satisfactory air-void system is provided

J. References

Kosmatka, S.H., B. Kerkhoff, W.C. Panarese. *Design and Control of Concrete Mixtures*. EB001.14. Skokie, IL: Portland Cement Association. 2002.

Taylor, P.C., S.H. Kosmatka, G.F. Voigt, et al. *Integrated Materials and Construction Practices for Concrete Pavement: A State-of-the-Practice Manual*. FHWA HIF-07-004. National Concrete Pavement Technology Center/Center for Transportation Research and Education, Iowa State University. 2006.

Pavement Thickness Design

A. General

The AASHO road test (completed in the 1950s) and subsequent AASHTO *Guide for the Design of Pavement Structures* (AASHTO Design Guide) provide the basis for current pavement design practices. To design a pavement by the AASHTO method, a number of design parameters must be determined or assumed. This section will explain the parameters required to design the pavement thickness of both concrete and hot mix asphalt roadways. The same parameters can be used for input data in computer programs on pavement determinations. The program used should be based on AASHTO design methods.

Even though the AASHTO Design Guide is several years old, it is still used throughout the industry for pavement thickness design. A newer design program called the Mechanistic-Empirical Pavement Design Guide (MEPDG) is available, however, it is costly and requires a great deal of data to be effective. The MEPDG does not generate a pavement thickness, it is set up to analyze the failure potential for a given thickness design. It is not generally used by local agencies. Each of the paving associations provides software programs for calculating pavement thickness. One HMA program is called I-Pave and was developed for the Asphalt Paving Association of Iowa. Another HMA program is called PaveXPress and was developed for the National Asphalt Pavement Association (NAPA). The American Concrete Pavement Association (ACPA) developed the PCC software, which is called StreetPave. The programs can be accessed through the respective websites of the paving associations. Users should be aware of the required inputs for the software programs, as well as the specific system defaults that cannot be changed or do not fit the project design criteria. If the program defaults do not match the project circumstances, the software program should not be used.

Historically municipalities have resorted to a one-size-fits-all approach by constructing standard pavement thicknesses for certain types of roadways without regard to traffic volumes or subgrade treatments. In an effort to show the effect of varying traffic loads and subgrade treatments on pavement thickness, this section provides comparison tables showing the various rigid and flexible pavement thicknesses calculated according to the AASHTO pavement design methodology. The ESAL and pavement thickness values shown in the tables are dependent upon the design parameters used in the calculations. The assumed parameters are described in the corresponding tables. The pavement designer should have a thorough understanding of the parameters and their reflection of actual site conditions prior to using them to select a pavement thickness. Projects that have traffic or site conditions that differ significantly from the values assumed herein should be evaluated with a site specific pavement design.

Engineers need to examine their agency's standard pavement foundation support system based on good engineering practices and the level of service they desire for the life of both HMA and PCC pavements. It is important to understand the characteristics of the soil and what cost-effective soil manipulation can be achieved, whether an aggregate subbase is used or not. If different soil types are encountered, and an aggregate subbase is not used, properly blending and compacting the soil will help reduce differential movement and help prevent cracking. Good designs, followed by good construction practices with a proper inspection/observation program, are critical to realize the full performance potential of either pavement type.

Designs that improve the foundation will extend the pavement life, improve the level of service throughout the life of the pavement, and provide more economical rehabilitation strategies at the end of the pavement's life for both HMA and PCC pavements. Although the initial cost to construct the pavement will undoubtedly be higher than placing the pavement on natural subgrade, the overall life cycle costs will be greatly improved.

Definitions of the pavement thickness design parameters are contained in Section 5F-1, B. Section 5F-1, C defines the process for calculating ESAL values. Section 5F-1, D provides the comparison tables discussed in the previous paragraph. Finally, example calculations are shown in Section 5F-1, E.

The pavement designer should be aware of the parameters that are required for the project under design. If those project design parameters differ from the parameters used to calculate the typical pavement thicknesses provide in this section, then a specific design set to meet the specific project parameters should be undertaken.

B. Pavement Thickness Design Parameters

Some of the pavement thickness design parameters required for the design of a rigid pavement differ from those for a flexible pavement. Table 5F-1.01 summarizes the parameters required for the design of each pavement structure.

Table 5F-1.01: Summary of Design Parameters for Pavement Thickness

Section	Description	Flexible HMA	Rigid JPCP/JRCP
5F-1, B, 1	**Performance Criteria**		
	a. Initial Serviceability Index	X	X
	b. Terminal Serviceability Index	X	X
5F-1, B, 2	**Design Variables**		
	a. Analysis Period	X	X
	b. Design Traffic	X	X
	c. Reliability	X	X
	d. Overall Standard Deviation	X	X
5F-1, B, 3	**Material Properties for Structural Design**		
	a. Soil Resilient Modulus	X	
	b. Modulus of Subgrade Reaction		X
	c. Concrete Properties		X
	d. Layer Coefficients	X	
5F-1, B, 4	**Pavement Structural Characteristics**		
	a. Coefficient of Drainage	X	X
	b. Load Transfer Coefficients for Jointed		X
	c. Loss of Support		X

The following considerations should be used when designing pavement thickness for flexible and rigid pavements.

1. **Performance Criteria (Serviceability Indexes):** Condition of pavements are rated with a present serviceability index (PSI) ranging from 5 (perfect condition) to 0 (impossible to travel).

   a. **Initial Serviceability Index (P_o):** The initial serviceability index (P_o) is the PSI immediately after the pavement is open. At the AASHO road test, values of 4.5 for rigid pavement and

4.2 for flexible pavement were assumed. These values are listed in the 1993 AASHTO Design Guide.

b. Terminal Serviceability Index (P_t): The terminal serviceability index (P_t) is considered to be the PSI that represents the lowest acceptable level before resurfacing or reconstruction becomes necessary.

The following values are recommended for terminal serviceability index.

Table 5F-1.02: Terminal Serviceability Indexes (P_t) for Street Classifications

P_t	Classifications
2.00	Secondary Roads and Local Residential Streets
2.25	Minor Collectors, Industrial, and Commercial Streets
2.50	Major Collectors and Arterials

c. Serviceability Loss: The predicted loss or drop in serviceability (ΔPSI) is the difference between initial and terminal serviceability (P_o - P_t). The ΔPSI is the basis for the pavement design.

2. Design Variables:

a. Analysis Period: This refers to the period of time for which the analysis is to be conducted. The recommended analysis period is 50 years for both concrete and asphalt pavements.

b. Design Traffic: An estimate of the number of Equivalent 18,000 pound Single Axle Loads (ESALs) during the analysis period is required. This value can be estimated based on:

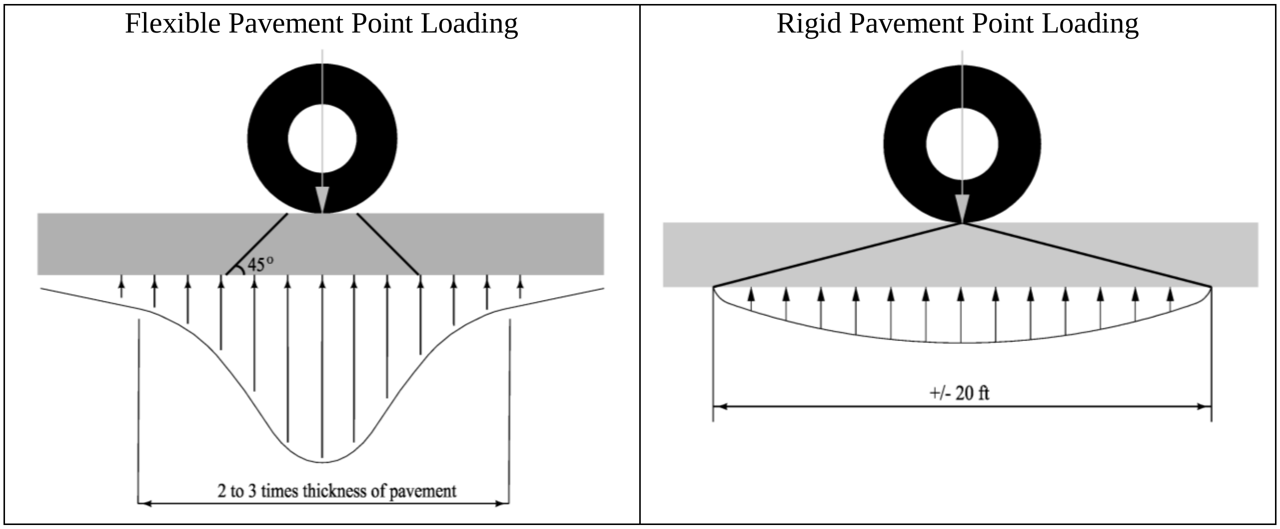
- the Average Annual Daily Traffic (AADT) in the base year,
- the average percentage of trucks expected to use the facility,
- the average annual traffic growth rate, and
- the analysis period.

It should be noted that it is not the wheel load but rather the damage to the pavement caused by the wheel load that is of particular concern. As described above, the ESAL is the standard unit of pavement damage and represents the damage caused by a single 18,000 pound axle load. Therefore, a two-axle vehicle with both axles loaded at 18,000 pounds would produce two ESALs. However, since vehicle configurations and axle loads vary, AASHTO has established a method to convert different axle loads and configurations to ESALs. For example, a 34,000 pound tandem axle produces approximately 1.9 ESALs for rigid pavement (1.1 for flexible pavement). Summing the different ESAL values for each axle combination on a vehicle provides a vehicle's Load Equivalency Factor (LEF). The LEF can then be applied to the assumed truck mix and the AADT to determine ESALs.

Section 5F-1, C details the steps involved in ESAL calculations and provides examples for both rigid and flexible pavements. ESAL tables for rigid and flexible pavements, and the corresponding assumptions used to create them, are provided for both two lane and four lane facilities.

The need for separate ESAL tables for flexible and rigid pavements is based on the inherent ability of each type of pavement to distribute a point loading. Rigid pavements have the ability to distribute the load across the slab. A point loading on a flexible pavement is more localized. This results in different ESAL factors for the two types of pavements. This is shown graphically in Figure 5F-1.01.

Figure 5F-1.01: Flexible vs. Rigid Point Loading Distribution



c. Reliability [R (%)]: Reliability is the probability that the design will succeed for the life of the pavement. Because higher roadway classification facilities are considered more critical to the transportation network, a higher reliability is used for these facilities. The following reliability values were assumed for the calculations.

Table 5F-1.03: Reliability for Flexible and Rigid Pavement Design

Street Classification	Reliability
Local Streets	80%
Collector Streets	88%
Arterial Streets	95%

d. Overall Standard Deviation (S_o): The Overall Standard Deviation is a coefficient that describes how well the AASHO Road Test data fits the AASHTO Design Equations. The lower the overall deviation, the better the equations models the data. The following ranges are recommended by the AASHTO Design Guide.

Table 5F-1.04: Overall Standard Deviation (S_o) for Rigid and Flexible Pavements

Pavement Type	Range of Values		Value Used
	Low	***High***	
Rigid Pavements	0.30	0.40	0.35
Flexible Pavements	0.40	0.50	0.45

3. Material Properties for Structural Design:

a. Soil Resilient Modulus (M_R): The important variable in describing the foundation for pavement design is the Soil Resilient Modulus (M_R). M_R is a property of the soil that indicates the stiffness or elasticity of the soil under dynamic loading.

The Soil Resilient Modulus measures the amount of recoverable deformation at any stress level for a dynamically loaded test specimen. The environment can affect pavement performance in several ways. Temperature and moisture changes can have an effect on the strength, durability, and load-carrying capacity of the pavement and roadbed materials. Another major environmental impact is the direct effect roadbed swelling, pavement blowups, frost heave, disintegration, etc. can have on loss of riding quality and serviceability. If any of these environmental effects have the potential to be present during the life cycle of the pavement, the M_R should be evaluated on a season by season basis, and a seasonal modulus developed.

The purpose of using seasonal modulus is to qualify the relative damage a pavement is subject to during each season of the year and treat it as part of the overall design. An effective soil modulus is then established for the entire year, which is equivalent to the combined effects of all monthly seasonal modulus values.

For the purposes of this section, the M_R value was calculated based on the proposed CBR values of 3, 5, and 10. The normal soils in Iowa have in situ CBR values of 1 to 3. In order to successfully develop a foundation CBR of 10, it is most likely going to involve use of a subgrade that is stabilized with cement, fly ash, or other product. NCHRP Project 1-28 provides a relationship between CBR value and M_R value (see equation below). Using this equation, the corresponding M_R values for CBR values of 3, 5, and 10 are shown. For further information regarding the relationship between soil types and bearing values, see Section 6E-1.

Relationship between CBR and M_R values per NCHRP Project 1-28:

$$M_R = 1941.488\ X\ CBR^{0.6844709}$$

CBR Value	**M_R Value**
3	4120
5	5840
10	9400

For flexible pavement design, Figure 2.3 from Chapter 2 of the 1993 AASHTO Design Guide was used to estimate the effective M_R value taking into account seasonal variability. Frozen conditions were assumed for the months of December, January, and February. Due to spring wetness and thawing conditions, the M_R value for the months of March and April were assumed to be 30% of normal conditions.

For rigid pavement design, the M_R value is used to calculate the modulus of subgrade reaction, k.

b. Modulus of Subgrade Reaction (k, k_c): Several variables are important in describing the foundation upon which the pavement rests:

- k - The modulus of subgrade reaction for the soil;
- k_c - A composite k that includes consideration of subbase materials under the new pavement
- M_R - Soil resilient modulus

1) Modulus of Subgrade Reaction, k: For concrete pavements, the primary requirement of the subgrade is that it be uniform. This is the fundamental reason for specifications on subgrade compaction. In concrete pavement design, the strength of the soil is characterized by the modulus of subgrade reaction or, as it is more commonly referred to, "k". An approximate relationship between k and M_R published by AASHTO is:

$$k = M_R/19.4$$

The resilient modulus is used to calculate the modulus of subgrade reaction. Refer to Section 6E-1 for the relationship between k and M_R.

2) **Composite Modulus of Subgrade Reaction, k_c:** In many highway applications the pavement is not placed directly on the subgrade. Instead, some type of subbase material is used. When this is done, the k value actually used for design is a "composite k" (k_c), which represents the strength of the subgrade corrected for the additional support provided by the subbase.

The analysis of field data completed as a part of the Iowa Highway Research Board (IHRB) Project TR-640 showed that the modulus of subgrade reaction and the drainage coefficient for 16 PCC sites, which ranged in ages between 1 and 42 years, were variable and found to be lower in-situ than typical parameters used in thickness design. This indicates a loss of support over time. This change in support is already partially reflected in the AASHTO serviceability index to a degree.

Similar to the procedures used to estimate the effective M_R value for flexible pavement design, the AASHTO Design Guide provides procedures for estimating the k_c value taking into account potential seasonal variability. For the purposes of this manual, the k_c value was calculated using Table 3.2 and Figures 3.3 through 3.6 from Chapter 3 of the 1993 AASHTO Design Guide. Again, frozen conditions were assumed for the months of December, January, and February. Due to spring wetness conditions, the M_R value for the months of March and April were assumed to be 30% of normal conditions.

c. Concrete Properties: PCC - Modulus of Elasticity (E_c) and Modulus of Rupture (S'_c).

The Modulus of Rupture (S'_c) used in the AASHTO Design Guide equations is represented by the <u>average</u> flexural strength of the pavement determined at 28 days using third-point loading (ASTM C 78).

The Modulus of Elasticity for concrete (E_c) depends largely on the strength of the concrete. Typical values are from 2 to 6 million psi. The following equation provides an approximate value for E_c:

$E_c = 6{,}750\ (S'_c)$

where:

S'_c = modulus of rupture [28 day flexural strength of the concrete using third point loading (psi)]

The approximate relation between modulus of rupture (S'_c) and compressive strength (f_c) is

$S'_c = 2.3\ f_c^{0.667}$(psi)

d. Layer Coefficients: Structural layer coefficients (a_i values) are required for flexible pavement structural design. A value for these coefficients is assigned to each layer material in the pavement structure in order to convert actual layer thickness into the structural number (SN). These historical values have been used in the structural calculations. If specific elements, such as a Superpave mix or polymer modified mix are used, the designer should adjust these values to reflect differing quality of materials.

The following table shows typical values for layer coefficients.

Table 5F-1.05: Layer Coefficients

Component	Coefficient	Minimum Thickness Allowed
Surface / Intermediate Course		
HMA with Type A Aggregate	0.44*	2
HMA with Type B Aggregate	0.44*	2
HMA with Type B Class 2 Aggregate	0.40	
Base Course		
Type A binder placed as base	0.40	2
HMA with Type B Aggregate	0.40	2
Asphalt Treated Base Class I	0.34*	4
Bituminous Treated Aggregate Base	0.23	6
Asphalt Treated Base Class II	0.26	4
Cold-Laid Bituminous Concrete Base	0.23	6
Cement Treated Granular (Aggregate) Base	0.20*	6
Soil-Cement Base	0.15	6
Crushed (Graded) Stone Base	0.14*	6
Macadam Stone Base	0.12	6
PCC Base (New)	0.50	
Old PCC	0.40**	
Crack and Seated PCC	0.25 to 0.30	
Rubblized PCC	0.24	
Cold-in-Place Recycled Asphalt Pavement	0.22 to 0.27	
Full Depth Reclamation	0.18	
Subbase Course		
Soil-Cement Subbase (10% cement)	0.10	6
Soil-Lime Subbase (10% lime)	0.10	6
Modified Subbase	0.14	4
Soil-Aggregate Subbase	0.05*	4

* Indicates coefficients taken from AASHTO Interim Guide for the Design of Flexible Pavement Structures.

** This value is for reasonably sound existing concrete. Actual value used may be lower, depending on the amount of deterioration that has occurred.

Source: AASHTO, Kansas State University, and Iowa DOT

4. **Pavement Structural Characteristics:**

   a. **Coefficient of Drainage:** Water under the pavement is one of the primary causes of pavement failure. Water, either from precipitation or groundwater, can cause the subgrade to become saturated and weaken. This can contribute to pavement pumping under heavy loads.

   C_d - The coefficient of drainage for rigid pavement design used to account for the quality of drainage.

   M_i - The coefficient of drainage for flexible pavement design used to modify layer coefficients.

At the AASHO road test, the pavements were not well drained as evidenced by the heavy pumping that occurred on some of the test sections. The cross-sections were elevated and drainage ditches were provided. However, edge drains, which are used frequently in today's street and highway construction, were not evaluated at the AASHO road test. Edge drains are an effective deterrent to pumping and associated pavement distress.

In selecting the proper C_d or M_i value, consideration must be given to two factors: 1) how effective is the drainage, and 2) how much of the time is the subgrade and subbase in a saturated condition? For example, pavements in dry areas with poor drainage may perform as well as pavements built in wet areas with excellent drainage.

The following definitions are offered as a guide.

- Excellent Drainage: Material drained to 50% of saturation in 2 hours.
- Good Drainage: Material drained to 50% of saturation in 1 day.
- Fair Drainage: Material drained to 50% of saturation in 7 days.
- Poor Drainage: Material drained to 50% of saturation in 1 month.
- Very Poor Drainage: Material does not drain.

Based on these definitions, the C_d or M_i value for the road test conditions would be 1.00. A value of 1.00 would have no impact on pavement thickness or the number of ESALs a section would carry. Lower values increase the required pavement thickness; higher values decrease the required pavement thickness. Based on Tables 2.4 and 2.5 from the 1993 AASHTO Design Guide, the analysis assumed a fair quality of drainage and 1% to 5% exposure to saturation for the drainable base sections.

b. Load Transfer Coefficients for Jointed and Jointed Reinforced Pavements: One item that distinguishes PCC pavement is the type of joint used to control cracking and whether or not steel dowels are used in the joint for load transfer. Each of these designs provides a different level of transfer of load from one side of a pavement joint to the other. To adjust projected pavement performance for these various designs, the load transfer coefficient or "J" factor is used.

c. Loss of Support: The loss of support factor is included in the design of rigid pavements to account for the potential loss of support arising from subgrade erosion and/or differential vertical soil movement. It is treated in the design process by diminishing the composite k-value based on the size of the void that may develop beneath the slab. The primary failure mode of rigid pavements at the AASHTO road test was loss of support caused by pumping of fines from underneath the slab. The loss of support variable was only applied on natural subgrades since the presence of water in the joint area contributes to the pumping of the subgrade fines through the joints and cracks. With granular subbase and longitudinal drains, the water is drained away from the joints and cracks much quicker.

Pavement design parameters within the PCC thickness design software programs often do not adequately reflect actual pavement foundation conditions except immediately after initial construction. Field data from testing completed at 16 Iowa sites showed lower coefficient of drainage values than those assumed in design, indicating that a potential migration of natural soils into the aggregate subbase over time may cause some loss of support. This in turn lowers the overall modulus of subgrade reaction. The results of the field testing indicating this loss of support due to mixing of the subgrade and subbase will need to be further validated by additional research. In order to maintain a high drainage coefficient, it is important to maintain separation between the soil subgrade and the aggregate subbase. One method of providing the separation is with a geotextile layer.

In most cases for local, low volume PCC roads, aggregate subbases do not influence thickness design to any measurable degree. According to MEPDG analysis for low volume PCC roadways (less than 1,000 ADT and 10% trucks), aggregate subbase thicknesses greater than 5 inches do not appear to improve the International Roughness Index (IRI) or reduce slab cracking.

Based on the IHRB TR-640 research with a limited data set of 16 Iowa sites, it was noted that a PCC pavement with an optimized foundation of granular subbase, subdrains, and a geotextile separation layer between the subgrade and subbase is likely to maintain a higher pavement condition index (PCI) over time than a PCC pavement on natural subgrade. The lower the variability and the higher the coefficient of drainage with an optimized foundation, the higher the pavement condition will be for a given period of time. Since the PCI prediction model from the IHRB research was developed based on a limited data set, it must be further validated with a larger pool of data. However, designers should consider the benefits of optimizing the foundations under their pavements to improve long-term serviceability.

C. Calculating ESAL Values

To estimate the design ESALs, the following procedure may be used. A more thorough analysis may also be performed using the procedures found in Appendix D of the 1993 AASHTO Design Guide or computer programs based on that procedure.

1. Obtain an estimate of the design AADT for the beginning, or base year of the analysis period.

2. Obtain an estimate of the average percentage of the AADT that will be trucks.

3. Select the base year design lane ESALs from Tables 5F-1.07 through 5F-1.10, depending upon whether the facility is two lane, four lane, rigid, or flexible.

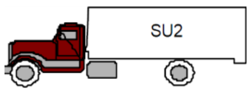
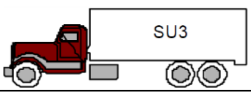
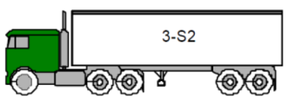
4. Select the growth factor from Table 5F-1.11 based on the average annual traffic growth rate and the analysis period.

5. Multiply the base year design lane ESALs, by the growth factor to obtain the total ESALs for the analysis period.

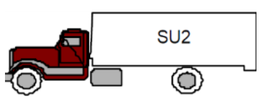
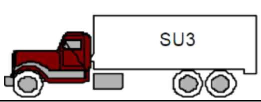
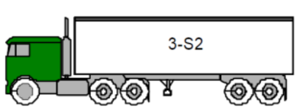
Table 5F-1.06 summarizes the inputs and calculations that went into creating Tables 5F-1.07 through 5F-1.10.

Table 5F-1.06: Truck Mixture for Urban Roadways and Determination of Truck ESAL Factor

A. Low Volume Streets

Vehicle Type		Percent of Total Trucks	Loading	Percent of Truck Type	Vehicle Weight (lbs)	Axle Type *S - Single* *TA - Tandem*	Axle Load (lbs)	ESAL Factor (per axle) Rigid	ESAL Factor (per axle) Flexible	LEF (by Vehicle) Rigid	LEF (by Vehicle) Flexible
	Single Unit (2 axles) (Class 5/6 Truck)	65%	Empty	30%	14,500	Front - S	7,000	0.019	0.024		
						Rear - S	7,500	0.025	0.032		
			Partial Load (50% Capacity)	50%	20,500	Front - S	8,000	0.033	0.041		
						Rear - S	12,500	0.212	0.242		
			Fully Loaded	20%	26,000	Front - S	9,000	0.053	0.066		
						Rear - S	17,000	0.785	0.799	*0.3033*	*0.3313*
	Dump Trucks - 3 axles (Class 7/8 truck) (doesn't address cheater axles)	15%	Empty	50%	22,000	Front - S	10,000	0.083	0.101		
						Rear - TA	12,000	0.026	0.018		
			Fully Loaded	50%	54,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099	*1.7835*	*1.369*
	Semis (5 axles)	20%	Empty	20%	26,000	Front - S	12,000	0.178	0.206		
						Rear - TA	7,000	0.003	0.002		
						Trailer - TA	7,000	0.003	0.002		
			Partial Load (50% Capacity)	60%	53,000	Front - S	13,000	0.251	0.282		
						Rear - TA	20,000	0.208	0.138		
						Trailer - TA	20,000	0.208	0.138		
			Fully Loaded	20%	80,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099		
						Trailer - TA	34,000	1.9	1.099	*1.5086*	*1.1204*
Composite Load Equivalency Factor (LEF) for "Trucks"										**0.76639**	**0.644775**

B. High Volume Streets

Vehicle Type		Percent of Total Trucks	Loading	Percent of Truck Type	Vehicle Weight (lbs)	Axle Type *S - Single* *TA - Tandem*	Axle Load (lbs)	ESAL Factor (per axle) Rigid	ESAL Factor (per axle) Flexible	LEF (by Vehicle) Rigid	LEF (by Vehicle) Flexible
	Single Unit (2 axles) (Class 5/6 Truck)	30%	Empty	30%	14,500	Front - S	7,000	0.019	0.024		
						Rear - S	7,500	0.025	0.032		
			Partial Load (50% Capacity)	50%	20,500	Front - S	8,000	0.033	0.041		
						Rear - S	12,500	0.212	0.242		
			Fully Loaded	20%	26,000	Front - S	9,000	0.053	0.066		
						Rear - S	17,000	0.785	0.799	*0.3033*	*0.3313*
	Dump Trucks - 3 axles (Class 7/8 truck) (doesn't address cheater axles)	10%	Empty	50%	22,000	Front - S	10,000	0.083	0.101		
						Rear - TA	12,000	0.026	0.018		
			Fully Loaded	50%	54,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099	*1.7835*	*1.369*
	Semis (5 axles)	60%	Empty	20%	26,000	Front - S	12,000	0.178	0.206		
						Rear - TA	7,000	0.003	0.002		
						Trailer - TA	7,000	0.003	0.002		
			Partial Load (50% Capacity)	60%	53,000	Front - S	13,000	0.251	0.282		
						Rear - TA	20,000	0.208	0.138		
						Trailer - TA	20,000	0.208	0.138		
			Fully Loaded	20%	80,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099		
						Trailer - TA	34,000	1.9	1.099	*1.5086*	*1.1204*
Composite Load Equivalency Factor (LEF) for "Trucks"										**1.1745**	**0.90853**

Assumptions:
ESAL factors for individual axles were determined from AASHTO Design Guide.
Assumed an initial pavement thickness of 8 inches and SN of 3.25 (does not significantly impact ESAL factor - +/- 3%)
Assumed a terminal serviceability of 2.25.
ESAL factors do not account for directional split. i.e. if 2 lane roadway with total AADT and 50/50 directional split, divide ESAL factor in half.
Vehicle weights and payload obtained from US Department of Energy Fact #621: Gross Vehicle Weight vs. Empty Vehicle Weight.
Assumed maximum single axle load of 20,000 lbs (including steering axle) and tandem axle load of 34,000 lbs.
Percent of fully loaded, partially loaded, and empty vehicles were estimated. No supporting documentation.

Source: 2010 Iowa DOT traffic count data (unless otherwise noted).

Table 5F-1.07: Base Year Design ESALs for Two Lane *Rigid Pavement*

% Trucks	Two Way, Base Year AADT							
	1,000	*2,000*	*3,000*	*4,000*	*5,000*	*10,000*	*15,000*	*20,000*
1	1,000	3,000	4,000	9,000	11,000	21,000	32,000	43,000
2	3,000	6,000	8,000	17,000	21,000	43,000	64,000	86,000
3	4,000	8,000	13,000	26,000	32,000	64,000	96,000	129,000
4	9,000	17,000	26,000	34,000	43,000	86,000	129,000	171,000
5	11,000	21,000	32,000	43,000	54,000	107,000	161,000	214,000
6	13,000	26,000	39,000	51,000	64,000	129,000	193,000	257,000
7	15,000	30,000	45,000	60,000	75,000	150,000	225,000	300,000
8	17,000	34,000	51,000	69,000	86,000	171,000	257,000	343,000
9	19,000	39,000	58,000	77,000	96,000	193,000	289,000	386,000
10	21,000	43,000	64,000	86,000	107,000	214,000	322,000	429,000
12	26,000	51,000	77,000	103,000	129,000	257,000	386,000	514,000
14	30,000	60,000	90,000	120,000	150,000	300,000	450,000	600,000
16	34,000	69,000	103,000	137,000	171,000	343,000	514,000	686,000
18	39,000	77,000	116,000	154,000	193,000	386,000	579,000	772,000
20	43,000	86,000	129,000	171,000	214,000	429,000	643,000	857,000
22	47,000	94,000	141,000	189,000	236,000	472,000	707,000	943,000
24	51,000	103,000	154,000	206,000	257,000	514,000	772,000	1,029,000
26	56,000	111,000	167,000	223,000	279,000	557,000	836,000	1,115,000
28	60,000	120,000	180,000	240,000	300,000	600,000	900,000	1,200,000
30	64,000	129,000	193,000	257,000	322,000	643,000	965,000	1,286,000

Assume two lane roadway with 50/50 directional split of base year AADT
Values within "box" assume a low volume mix of trucks

Table 5F-1.08: Base Year Design ESALs for Two Lane *Flexible Pavement*

% Trucks	Two Way, Base Year AADT							
	1,000	*2,000*	*3,000*	*4,000*	*5,000*	*10,000*	*15,000*	*20,000*
1	1,000	2,000	4,000	7,000	8,000	17,000	25,000	33,000
2	2,000	5,000	7,000	13,000	17,000	33,000	50,000	66,000
3	4,000	7,000	11,000	20,000	25,000	50,000	75,000	99,000
4	7,000	13,000	20,000	27,000	33,000	66,000	99,000	133,000
5	8,000	17,000	25,000	33,000	41,000	83,000	124,000	166,000
6	10,000	20,000	30,000	40,000	50,000	99,000	149,000	199,000
7	12,000	23,000	35,000	46,000	58,000	116,000	174,000	232,000
8	13,000	27,000	40,000	53,000	66,000	133,000	199,000	265,000
9	15,000	30,000	45,000	60,000	75,000	149,000	224,000	298,000
10	17,000	33,000	50,000	66,000	83,000	166,000	249,000	332,000
12	20,000	40,000	60,000	80,000	99,000	199,000	298,000	398,000
14	23,000	46,000	70,000	93,000	116,000	232,000	348,000	464,000
16	27,000	53,000	80,000	106,000	133,000	265,000	398,000	531,000
18	30,000	60,000	90,000	119,000	149,000	298,000	448,000	597,000
20	33,000	66,000	99,000	133,000	166,000	332,000	497,000	663,000
22	36,000	73,000	109,000	146,000	182,000	365,000	547,000	730,000
24	40,000	80,000	119,000	159,000	199,000	398,000	597,000	796,000
26	43,000	86,000	129,000	172,000	216,000	431,000	647,000	862,000
28	46,000	93,000	139,000	186,000	232,000	464,000	696,000	929,000
30	50,000	99,000	149,000	199,000	249,000	497,000	746,000	995,000

Assume two lane roadway with 50/50 directional split of base year AADT
Values within "box" assume a low volume mix of trucks

Table 5F-1.09: Base Year Design ESALs for Four Lane *Rigid Pavement*

	Two Way, Base Year AADT							
% Trucks	*2,000*	*5,000*	*10,000*	*15,000*	*20,000*	*25,000*	*30,000*	*35,000*
1	3,000	6,000	13,000	19,000	26,000	32,000	39,000	45,000
2	5,000	13,000	26,000	39,000	51,000	64,000	77,000	90,000
3	8,000	19,000	39,000	58,000	77,000	96,000	116,000	135,000
4	10,000	26,000	51,000	77,000	103,000	129,000	154,000	180,000
5	13,000	32,000	64,000	96,000	129,000	161,000	193,000	225,000
6	15,000	39,000	77,000	116,000	154,000	193,000	231,000	270,000
7	18,000	45,000	90,000	135,000	180,000	225,000	270,000	315,000
8	21,000	51,000	103,000	154,000	206,000	257,000	309,000	360,000
9	23,000	58,000	116,000	174,000	231,000	289,000	347,000	405,000
10	26,000	64,000	129,000	193,000	257,000	322,000	386,000	450,000
12	31,000	77,000	154,000	231,000	309,000	386,000	463,000	540,000
14	36,000	90,000	180,000	270,000	360,000	450,000	540,000	630,000
16	41,000	103,000	206,000	309,000	412,000	514,000	617,000	720,000
18	46,000	116,000	231,000	347,000	463,000	579,000	694,000	810,000
20	51,000	129,000	257,000	386,000	514,000	643,000	772,000	900,000
22	57,000	141,000	283,000	424,000	566,000	707,000	849,000	990,000
24	62,000	154,000	309,000	463,000	617,000	772,000	926,000	1,080,000
26	67,000	167,000	334,000	502,000	669,000	836,000	1,003,000	1,170,000
28	72,000	180,000	360,000	540,000	720,000	900,000	1,080,000	1,260,000
30	77,000	193,000	386,000	579,000	772,000	965,000	1,157,000	1,350,000

Table 5F-1.10: Base Year Design ESALs for Four Lane *Flexible Pavement*

	Two Way, Base Year AADT							
% Trucks	*2,000*	*5,000*	*10,000*	*15,000*	*20,000*	*25,000*	*30,000*	*35,000*
1	2,000	5,000	10,000	15,000	20,000	25,000	30,000	35,000
2	4,000	10,000	20,000	30,000	40,000	50,000	60,000	70,000
3	6,000	15,000	30,000	45,000	60,000	75,000	90,000	104,000
4	8,000	20,000	40,000	60,000	80,000	99,000	119,000	139,000
5	10,000	25,000	50,000	75,000	99,000	124,000	149,000	174,000
6	12,000	30,000	60,000	90,000	119,000	149,000	179,000	209,000
7	14,000	35,000	70,000	104,000	139,000	174,000	209,000	244,000
8	16,000	40,000	80,000	119,000	159,000	199,000	239,000	279,000
9	18,000	45,000	90,000	134,000	179,000	224,000	269,000	313,000
10	20,000	50,000	99,000	149,000	199,000	249,000	298,000	348,000
12	24,000	60,000	119,000	179,000	239,000	298,000	358,000	418,000
14	28,000	70,000	139,000	209,000	279,000	348,000	418,000	487,000
16	32,000	80,000	159,000	239,000	318,000	398,000	478,000	557,000
18	36,000	90,000	179,000	269,000	358,000	448,000	537,000	627,000
20	40,000	99,000	199,000	298,000	398,000	497,000	597,000	696,000
22	44,000	109,000	219,000	328,000	438,000	547,000	657,000	766,000
24	48,000	119,000	239,000	358,000	478,000	597,000	716,000	836,000
26	52,000	129,000	259,000	388,000	517,000	647,000	776,000	905,000
28	56,000	139,000	279,000	418,000	557,000	696,000	836,000	975,000
30	60,000	149,000	298,000	448,000	597,000	746,000	895,000	1,045,000

Assumes four lane roadway with 50/50 directional split of two way base year ADT and 60% of trucks in design lane

Table 5F-1.11: Growth Factor

Design Period Years (n)	Average Annual Traffic Growth Rate, Percent (r)					
	No Growth	*1%*	*2%*	*3%*	*4%*	*5%*
1	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.0	2.0	2.0	2.0	2.1
3	3.0	3.0	3.1	3.1	3.1	3.2
4	4.0	4.1	4.1	4.2	4.2	4.3
5	5.0	5.1	5.2	5.3	5.4	5.5
6	6.0	6.2	6.3	6.5	6.6	6.8
7	7.0	7.2	7.4	7.7	7.9	8.1
8	8.0	8.3	8.6	8.9	9.2	9.5
9	9.0	9.4	9.8	10.2	10.6	11.0
10	10.0	10.5	10.9	11.5	12.0	12.6
11	11.0	11.6	12.2	12.8	13.5	14.2
12	12.0	12.7	13.4	14.2	15.0	15.9
13	13.0	13.8	14.7	15.6	16.6	17.7
14	14.0	14.9	16.0	17.1	18.3	19.6
15	15.0	16.1	17.3	18.6	20.0	21.6
16	16.0	17.3	18.6	20.2	21.8	23.7
17	17.0	18.4	20.0	21.8	23.7	25.8
18	18.0	19.6	21.4	23.4	25.6	28.1
19	19.0	20.8	22.8	25.1	27.7	30.5
20	20.0	22.0	24.3	26.9	29.8	33.1
21	21.0	23.2	25.8	28.7	32.0	35.7
22	22.0	24.5	27.3	30.5	34.2	38.5
23	23.0	25.7	28.8	32.5	36.6	41.4
24	24.0	27.0	30.4	34.4	39.1	44.5
25	25.0	28.2	32.0	36.5	41.6	47.7
26	26.0	29.5	33.7	38.6	44.3	51.1
27	27.0	30.8	35.3	40.7	47.1	54.7
28	28.0	32.1	37.1	42.9	50.0	58.4
29	29.0	33.5	38.8	45.2	53.0	62.3
30	30.0	34.8	40.6	47.6	56.1	66.4
31	31.0	36.1	42.4	50.0	59.3	70.8
32	32.0	37.5	44.2	52.5	62.7	75.3
33	33.0	38.9	46.1	55.1	66.2	80.1
34	34.0	40.3	48.0	57.7	69.9	85.1
35	35.0	41.7	50.0	60.5	73.7	90.3
36	36.0	43.1	52.0	63.3	77.6	95.8
37	37.0	44.5	54.0	66.2	81.7	101.6
38	38.0	46.0	56.1	69.2	86.0	107.7
39	39.0	47.4	58.2	72.2	90.4	114.1
40	40.0	48.9	60.4	75.4	95.0	120.8
41	41.0	50.4	62.6	78.7	99.8	127.8
42	42.0	51.9	64.9	82.0	104.8	135.2
43	43.0	53.4	67.2	85.5	110.0	143.0
44	44.0	54.9	69.5	89.0	115.4	151.1
45	45.0	56.5	71.9	92.7	121.0	159.7
46	46.0	58.0	74.3	96.5	126.9	168.7
47	47.0	59.6	76.8	100.4	132.9	178.1
48	48.0	61.2	79.4	104.4	139.3	188.0
49	49.0	62.8	81.9	108.5	145.8	198.4
50	50.0	64.5	84.6	112.8	152.7	209.3

Growth Factor = $\frac{[(1+r)^n]-1}{r}$ for values of n > 0

D. Determining Pavement Thickness

Once the ESALs have been determined, the pavement thickness may be determined by comparing the calculated ESAL value to Tables 5F-1.13 through 5F-1.18. These tables provide recommended pavement thicknesses for various subgrade conditions, roadway types, and pavement types. Due to established policies in many jurisdictions across the state, the minimum pavement thickness for streets on natural subgrade was set at 7 inches for rigid pavement and 8 inches for flexible pavement. For pavements with a granular subbase, the minimum thickness was set at 6 inches for both pavement types. As noted in the thickness tables, whenever a thickness was calculated that was less than the minimum, the minimum was used.

Tables 5F-1.13 through 5F-1.18 were developed according to the guidelines of the AASHTO Design Guide. The AASHTO pavement design methodology is based upon the results of the AASHO Road Test, which was a series of full scale experiments conducted in Illinois in the 1950s. The design methodology that grew out of the Road Test considers numerous factors that affect the performance of a pavement. Table 5F-1.12 describes the assumptions used in the development of the pavement thickness tables. An explanation of each variable, as well as a recommended range, is provided in the AASHTO Guide.

For projects with unique conditions such as unusual soils, high truck volumes, significant drainage problems, or where specialized subgrade or subbase treatments are utilized, a special design may be warranted. The values in the tables above have been selected to represent typical conditions. An effort has been made not to be overly conservative in the establishment of the design parameters. For this reason, the designer is cautioned against deviating from the values presented in the tables above unless materials testing and/or project site conditions warrant such deviation.

Table 5F-1.12: Parameter Assumptions Used for Pavement Thickness Design Tables

Subbase:	**Natural**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granular**		
CBR Value:	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
Rigid Pavement Parameters																		
Initial Serviceability Index, P_o	4.5																	
Terminal Serviceability Index, P_t	Local Roads = 2.00 Collector Roads = 2.25 Arterials = 2.50																	
Reliability, R	Local Roads = 80% Collector Roads = 88% Arterial Roads = 95%																	
Overall Standard Deviation, S_o	0.35																	
Loss of Support, LS	1			0														
Soil Resilient Modulus, M_R Per NCHRP Project 128 $M_R = 1941.488 \times CBR^{0.6844709}$	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400
Subbase Resilient Modulus, E_{SB} * Assumed	Not Applicable			30,000														
Modulus of Subgrade Reaction, k, and Composite Modulus of Subgrade Reaction, k_c Use AASHTO Chapter 3, Table 3.2 and Figures 3.3 - 3.6 to determine	252	327	469	263	332	455	284	354	477	308	379	504	332	406	535	356	433	566
Adjusted k or k_c for Loss of Support Use AASHTO Part 2, Figure 3.6	85	105	160	263	332	455	284	354	477	308	379	504	332	406	535	356	433	566
Coefficient of Drainage, C_d	1.00			1.10														
Modulus of Rupture, S'_c $S'_c = 2.3 \times f_c^{0.667}$ * Assumed 4,000 psi concrete	580																	
Modulus of Elasticity, E_c $E_c = 6{,}750 \times S'_c$ * Assumed 4,000 psi concrete	3,915,000																	
Load Transfer, J	J = 3.1 (Pavement Thickness < 8") J = 2.7 (Pavement Thickness ≥ 8")																	
Flexible Pavement Parameters																		
Initial Serviceability Index, P_o	4.2																	
Terminal Serviceability Index, P_t	Local Roads = 2.00 Collector Roads = 2.25 Arterials = 2.50																	
Reliability, R	Local Roads = 80% Collector Roads = 88% Arterial Roads = 95%																	
Overall Standard Deviation, S_o	0.45																	
Layer Coefficients	Surface/Intermediate Course = 0.44 Base Course = 0.40 Granular Subbase = 0.14																	
Soil Resilient Modulus, M_R Per NCHRP Project 128 $M_R = 1941.488 \times CBR^{0.6844709}$	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400
Effective Soil Reslient Modulus, MR Use AASHTO Chapter 2, Figure 2.3 to determine	2460	3480	5580	2460	3480	5580	2460	3480	5580	2460	3480	5580	2460	3480	5580	2460	3480	5580
Coefficient of Drainage, M_i	1.00			1.15														

The following flowchart depicts a summary of the analysis process.

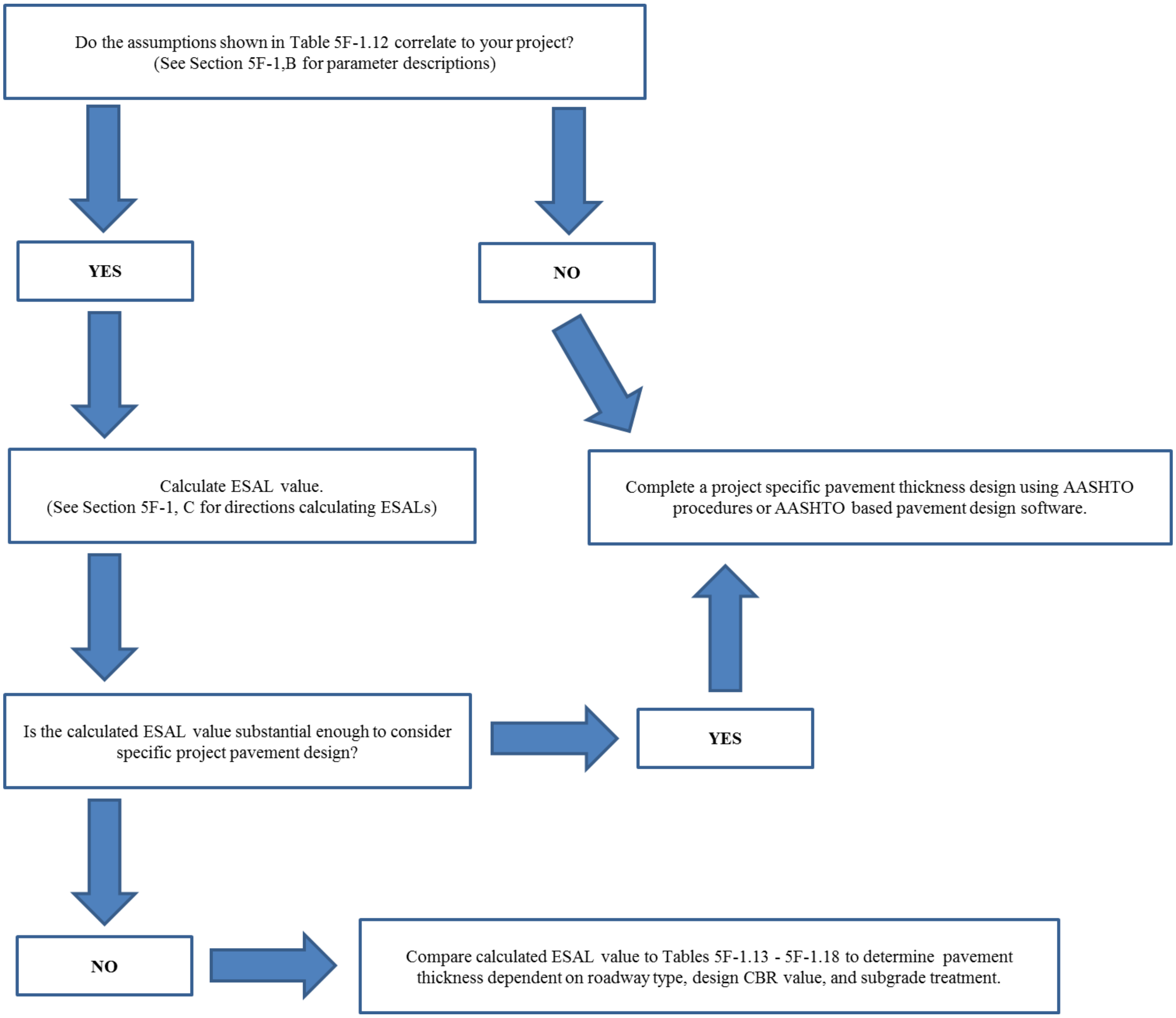


Table 5F-1.13: Required Thickness for Rigid Pavement - *Local Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
300,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
500,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
750,000	7	7**	7**	6*	6*	6*	6*	6*	6*
1,000,000	7	7	7	6	6	6*	6	6*	6*
1,500,000	7.5	7.5	7.5	6.5	6.5	6	6.5	6	6
2,000,000	8	8	7.5	7	6.5	6.5	7	6.5	6.5
3,000,000	8	8	8	7.5	7	7	7.5	7	7
4,000,000	8	8	8	7.5	7.5	7.5	7.5	7.5	7
5,000,000	8.5	8.5	8	8	8	7.5	8	8	7.5

***** The value shown represents a 6 inch minimum; the actual value is less.
****** The value shown represents a 7 inch minimum; the actual value is less.

Table 5F-1.14: Required Thickness for Rigid Pavement - *Collector Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
750,000	7	7	7	6	6	6*	6	6	6*
1,000,000	7.5	7.5	7.5	6.5	6	6	6.5	6	6
1,500,000	8	8	8	7	6.5	6.5	7	6.5	6.5
2,000,000	8	8	8	7.5	7	7	7	7	7
3,000,000	8.5	8	8	8	7.5	7.5	8	7.5	7.5
4,000,000	8.5	8.5	8.5	8	8	8	8	8	8
5,000,000	9	9	8.5	8	8	8	8	8	8
7,500,000	9.5	9.5	9.5	8.5	8.5	8	8.5	8	8
10,000,000	10	10	9.5	9	8.5	8.5	9	8.5	8.5
12,500,000	10.5	10	10	9	9	9	9	9	9
15,000,000	10.5	10.5	10.5	9.5	9.5	9	9.5	9.5	9
17,500,000	11	10.5	10.5	9.5	9.5	9.5	9.5	9.5	9.5
20,000,000	11	11	11	10	10	9.5	10	9.5	9.5

***** The value shown represents a 6 inch minimum; the actual value is less.

Table 5F-1.15: Required Thickness for Rigid Pavements - *Arterial Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
1,000,000	8	8	8	7	7	6.5	7	6.5	6.5
1,500,000	8	8	8	7.5	7.5	7	7.5	7.5	7
2,000,000	8.5	8.5	8	8	8	7.5	8	7.5	7.5
3,000,000	9	9	8.5	8	8	8	8	8	8
4,000,000	9.5	9	9	8.5	8	8	8	8	8
5,000,000	9.5	9.5	9.5	8.5	8.5	8	8.5	8.5	8
7,500,000	10	10	10	9	9	9	9	9	9
10,000,000	10.5	10.5	10.5	9.5	9.5	9	9.5	9.5	9
12,500,000	11	11	11	10	10	9.5	10	10	9.5
15,000,000	11.5	11.5	11	10.5	10	10	10	10	10
17,500,000	11.5	11.5	11.5	10.5	10.5	10	10.5	10.5	10
20,000,000	12	12	11.5	10.5	10.5	10.5	10.5	10.5	10.5
22,500,000	12	12	12	11	11	10.5	11	11	10.5
25,000,000	12	12	12	11	11	11	11	11	10.5
27,500,000	---	---	12	11.5	11	11	11	11	11
30,000,000	---	---	---	11.5	11.5	11	11.5	11.5	11

Table 5F-1.16: Required Thickness for Flexible Pavements - *Local Roads*

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granular**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
300,000	9.5	8.5	8**	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*	6*	6*	6*
500,000	10	9	8	8.5	7.5	6	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*
750,000	10.5	9.5	8	9	8	6.5	8.5	7.5	6	7.5	6.5	6*	7	6*	6*	6	6*	6*
1,000,000	11	10	8.5	9.5	8.5	7	9	7.5	6	8	7	6*	7.5	6	6*	6.5	6*	6*
1,500,000	11.5	10.5	9	10	9	7.5	9.5	8	7	8.5	7.5	6	8	6.5	6*	7	6	6*
2,000,000	12	11	9.5	10.5	9.5	8	10	8.5	7	9	8	6.5	8.5	7	6*	7.5	6.5	6*
3,000,000	---	11.5	10	11	10	8.5	10.5	9	7.5	9.5	8.5	7	9	8	6	8	7	6*
4,000,000	---	12	10.5	11.5	10.5	9	11	9.5	8	10	9	7.5	9.5	8	6.5	8.5	7.5	6*
5,000,000	---	---	10.5	12	11	9	11.5	10	8.5	10.5	9	7.5	10	8.5	7	9	8	6

***** The value shown represents a 6 inch minimum; the actual value is less.
****** The value shown represents an 8 inch minimum; the actual value is less.

Table 5F-1.17: Required Thickness for Flexible Pavements - *Collector Roads*

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granula**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**1**
750,000	11.5	10.5	9	10	9	7.5	9	8	6.5	8.5	7.5	6*	8	6.5	6*	7	6*	[illegible]
1,000,000	12	10.5	9	10.5	9	7.5	9.5	8.5	7	9	7.5	6	8	7	6*	7.5	6	[illegible]
1,500,000	12.5	11.5	9.5	11	10	8	10.5	9	7.5	9.5	8.5	6.5	9	7.5	6	8	7	[illegible]
2,000,000	13	12	10	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6.5	8.5	7.5	[illegible]
3,000,000	14	12.5	10.5	12.5	11	9	11.5	10	8.5	11	9.5	8	10	8.5	7	9	8	
4,000,000	---	13	11	13	11.5	9.5	12	10.5	9	11.5	10	8	10.5	9	7.5	10	8.5	[illegible]
5,000,000	---	13.5	11.5	13.5	12	10	12.5	11	9	11.5	10.5	8.5	11	9.5	8	10	8.5	
7,500,000	---	14	12	14	12.5	10.5	13.5	12	10	12.5	11	9	11.5	10	8.5	11	9.5	[illegible]
10,000,000	---	---	12.5	---	13	11	14	12.5	10.5	13	11.5	9.5	12.5	10.5	9	11.5	10	
12,500,000	---	---	13	---	13.5	11.5	---	12.5	10.5	13.5	12	10	13	11	9	12	10.5	[illegible]
15,000,000	---	---	13.5	---	14	12	---	13	11	14	12.5	10.5	13	11.5	9.5	12.5	10.5	[illegible]
17,500,000	---	---	13.5	---	14	12	---	13.5	11.5	---	12.5	10.5	13.5	12	10	12.5	11	
20,000,000	---	---	14	---	---	12.5	---	13.5	11.5	---	13	11	14	12	10	13	11.5	

***** The value shown represents a 6 inch minimum; the actual value is less.

Table 5F-1.18: Required Thickness for Flexible Pavements - *Arterial Roads*

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granular**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
1,000,000	13.5	12	10	11.5	10.5	8.5	11	9.5	8	10	9	7	9.5	8	6.5	8.5	7.5	5.5
1,500,000	14	12.5	11	12.5	11	9	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6
2,000,000	---	13	11	13	11.5	9.5	12.5	11	9	11.5	10	8	10.5	9	7.5	10	8.5	6.5
3,000,000	---	14	12	14	12.5	10.5	13	11.5	9.5	12.5	10.5	9	11.5	10	8	10.5	9	7.5
4,000,000	---	---	12.5	---	13	11	13.5	12	10	13	11	9.5	12	10.5	8.5	11.5	9.5	8
5,000,000	---	---	13	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9	11.5	10	8
7,500,000	---	---	13.5	---	14	12	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9
10,000,000	---	---	14	---	---	12.5	---	14	11.5	---	13	11	14	12.5	10	13.5	11.5	9.5
12,500,000	---	---	---	---	---	13	---	---	12	---	13.5	11.5	---	13	10.5	14	12	10
15,000,000	---	---	---	---	---	13.5	---	---	12.5	---	14	11.5	---	13	11	14	12.5	10
17,500,000	---	---	---	---	---	13.5	---	---	13	---	---	12	---	13.5	11	---	12.5	10.5
20,000,000	---	---	---	---	---	14	---	---	13	---	---	12.5	---	14	11.5	---	13	10.5
22,500,000	---	---	---	---	---	14	---	---	13.5	---	---	12.5	---	14	11.5	---	13.5	11
25,000,000	---	---	---	---	---	---	---	---	13.5	---	---	12.5	---	---	12	---	13.5	11
27,500,000	---	---	---	---	---	---	---	---	13.5	---	---	13	---	---	12	---	14	11.5
30,000,000	---	---	---	---	---	---	---	---	14	---	---	13	---	---	12.5	---	14	11.5

E. Example Pavement Thickness Design Calculations

1. Two Lane Local Roadway, PCC

AADT = 1,000
Trucks = 2%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.07) = 3,000
Growth Factor (from Table 5F-1.11) = 84.6
3,000 ESALs X 84.6 = 253,800 ESALs

By referring to Table 5F-1.13 and rounding up the ESAL calculation to 300,000 (see below), the pavement thickness alternatives are either 6 inches or 7 inches depending on the presence of granular subbase.

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
300,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
500,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
750,000	7	7**	7**	6*	6*	6*	6*	6*	6*

***** The value shown represents a 6 inch minimum; the actual value is less.
****** The value shown represents a 7 inch minimum; the actual value is less.

2. Two Lane Local Roadway, HMA

AADT = 1,000
Trucks = 2%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.08) = 2,000
Growth Factor (from Table 5F-1.1) = 84.6
2,000 ESALs X 84.6 = 169,200 ESALs

By referring to Table 5F-1.16 and rounding up the ESAL calculation to 300,000 (see below), the pavement thickness alternatives range from 6 inches to 9.5 inches depending on the CBR value and subbase treatment selected.

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granular**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
300,000	9.5	8.5	8**	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*	6*	6*	6*
500,000	10	9	8	8.5	7.5	6	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*
750,000	10.5	9.5	8	9	8	6.5	8.5	7.5	6	7.5	6.5	6*	7	6*	6*	6	6*	6*

***** The value shown represents the 6 inch minimum; the actual value is less.
****** The value shown represents an 8 inch minimum; the actual value is less.

3. Two Lane Collector Roadway, PCC

AADT = 5,000
Trucks = 4%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.07) = 43,000
Growth Factor (from Table 5F-1.11) = 84.6
43,000 ESALs X 84.6 = 3,637,800 ESALs

By referring to Table 5F-1.14 and rounding up the ESAL calculation to 4,000,000 (see below), the pavement thickness alternatives range from 8 inches to 8.5 inches depending on the CBR value and subbase treatment selected.

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
750,000	7	7	7	6	6	6*	6	6	6*
1,000,000	7.5	7.5	7.5	6.5	6	6	6.5	6	6
1,500,000	8	8	8	7	6.5	6.5	7	6.5	6.5
2,000,000	8	8	8	7.5	7	7	7	7	7
3,000,000	8.5	8	8	8	7.5	7.5	8	7.5	7.5
4,000,000	8.5	8.5	8.5	8	8	8	8	8	8
5,000,000	9	9	8.5	8	8	8	8	8	8

***** The value shown represents a 6 inch minimum; the actual value is less.

4. Two Lane Collector Roadway, HMA

AADT = 5,000
Trucks = 4%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.08) = 33,000
Growth Factor (from Table 5F-1.11) = 84.6
33,000 ESALs X 84.6 = 2,791,800 ESALs

By referring to Table 5F-1.17 and rounding up the ESAL calculation to 3,000,000 (see below), the pavement thickness alternatives range from 6 inches to 14 inches depending on the CBR value and subbase treatment selected.

Subbase	**Natural Subgrade**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granula**		
ESAL/CBR	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**1**
750,000	11.5	10.5	9	10	9	7.5	9	8	6.5	8.5	7.5	6*	8	6.5	6*	7	6*	6
1,000,000	12	10.5	9	10.5	9	7.5	9.5	8.5	7	9	7.5	6	8	7	6*	7.5	6	6
1,500,000	12.5	11.5	9.5	11	10	8	10.5	9	7.5	9.5	8.5	6.5	9	7.5	6	8	7	6
2,000,000	13	12	10	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6.5	8.5	7.5	6
3,000,000	14	12.5	10.5	12.5	11	9	11.5	10	8.5	11	9.5	8	10	8.5	7	9	8	[illegible]
4,000,000	---	13	11	13	11.5	9.5	12	10.5	9	11.5	10	8	10.5	9	7.5	10	8.5	6

***** The value shown represents the 6 inch minimum; the actual value is less.

5. Four Lane Arterial Roadway, PCC

AADT = 15,000
Trucks = 5%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.09) = 96,000
Growth Factor (from Table 5F-1.11) = 84.6
96,000 ESALs X 84.6 = 8,121,600 ESALs

By referring to Table 5F-1.15 and rounding up the ESAL calculation to 10,000,000 (see below), the pavement thickness alternatives range from 9 inches to 10.5 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
1,000,000	8	8	8	7	7	6.5	7	6.5	6.5
1,500,000	8	8	8	7.5	7.5	7	7.5	7.5	7
2,000,000	8.5	8.5	8	8	8	7.5	8	7.5	7.5
3,000,000	9	9	8.5	8	8	8	8	8	8
4,000,000	9.5	9	9	8.5	8	8	8	8	8
5,000,000	9.5	9.5	9.5	8.5	8.5	8	8.5	8.5	8
7,500,000	10	10	10	9	9	9	9	9	9
10,000,000	10.5	10.5	10.5	9.5	9.5	9	9.5	9.5	9
12,500,000	11	11	11	10	10	9.5	10	10	9.5

6. Four Lane Arterial Roadway, HMA

AADT = 15,000
Trucks = 5%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.10) = 75,000
Growth Factor (from Table 5F-1.11) = 84.6
75,000 ESALs X 84.6 = 6,345,000 ESALs

By referring to Table 5F-1.18 and rounding up the ESAL calculation to 7,500,000 (see below), the pavement thickness alternatives range from 9 inches to 14 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10
1,000,000	13.5	12	10	11.5	10.5	8.5	11	9.5	8	10	9	7	9.5	8	6.5	8.5	7.5	5.5
1,500,000	14	12.5	11	12.5	11	9	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6
2,000,000	---	13	11	13	11.5	9.5	12.5	11	9	11.5	10	8	10.5	9	7.5	10	8.5	6.5
3,000,000	---	14	12	14	12.5	10.5	13	11.5	9.5	12.5	10.5	9	11.5	10	8	10.5	9	7.5
4,000,000	---	---	12.5	---	13	11	13.5	12	10	13	11	9.5	12	10.5	8.5	11.5	9.5	8
5,000,000	---	---	13	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9	11.5	10	8
7,500,000	---	---	13.5	---	14	12	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9
10,000,000	---	---	14	---	---	12.5	---	14	11.5	---	13	11	14	12.5	10	13.5	11.5	9.5

F. References

American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide*. 1993.

Van Til, C.J., et al. *NCHRP Report 1-28: Evaluation of AASHO Interim Guides for Design of Pavement Structures* (AASHTO Design Guide). The American Association of State Highway and Transportation Officials. 444 North Capitol St NW Suite 225, Washington, D. C.1972.

White, D. and Vennapusa, P. *Optimizing Pavement Base, Subbase, and Subgrade Layers for Cost and Performance of Local Roads*. IHRB Project TR-640. Iowa Highway Research Board and Iowa Department of Transportation, Ames, Iowa. 2014.

General Information for Joints

A. General Information

The need for a jointing system in concrete pavements results from the desire to control the location and geometry of transverse and longitudinal cracking. Without jointing, uncontrolled cracking occurs due to stresses in the pavement from shrinkage, temperature and moisture differentials, and applied traffic loadings.

A good jointing plan will ease construction by providing clear guidance. The development of a jointing plan requires the designer to think about not only the specific project requirements but also the entire project jointing system. Jointing layouts in some parts of a project can have a substantial impact on other parts. In order to control concrete pavement cracking and subsequently maintain structural integrity, designers need to develop an understanding of how to complete jointing layouts of mainline pavements and intersections to obtain a comprehensive jointing system. This will allow a check on the pattern, type of joints, and matching joints to their purpose.

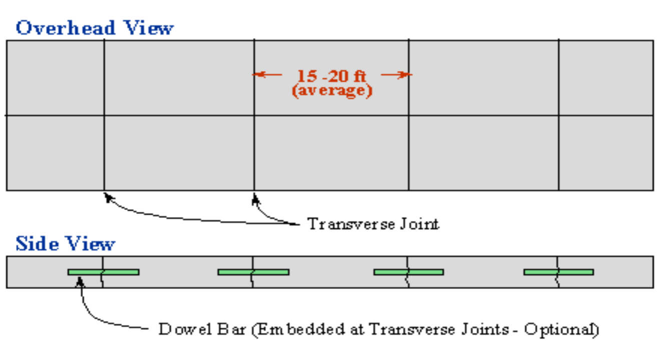
There are three types of jointing systems for concrete pavements, including:

- Jointed plain concrete pavement
- Jointed reinforced concrete pavement
- Continuously reinforced concrete pavement

This section deals primarily with jointed plain concrete pavements (JPCP) with tie bars or dowel bars only at joints as shown in Figure 5G-1.01. The function of the bars in JPCP is to provide load transfer across the joints, either through tie bars that hold the adjacent slabs together and maintain aggregate interlock or through dowel bars that provide mechanical load transfer even with slab movement.

Some cities specify jointed reinforced concrete pavements (JRCP), sometimes referred to as distributed steel reinforcing pavements. Section 5G-2 discusses jointed reinforced pavements. Jointed reinforced pavements allow for longer spacing between transverse joints by utilizing bar mats to hold midpanel cracks together and maintain structural integrity of the slab. Jointed reinforced pavements should not be confused with continuously reinforced concrete pavement, CRCP, which has very few or no joints.

Figure 5G-1.01: Jointed Plain Concrete Pavement (JPCP)



The primary benefits of jointing include:

- Crack control.
- Accommodating slab movements.
- Providing desirable load transfer.

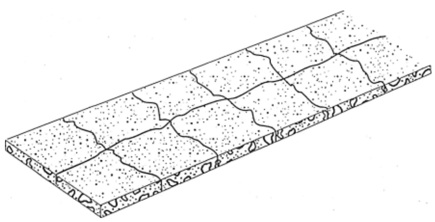
Secondary benefits of jointing include:

- Dividing the pavement into practical construction increments (i.e. traffic lanes, pavement widening).
- Providing traffic guidance.

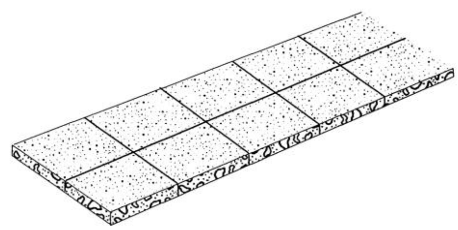
B. Crack Development

Crack development results from stress that exceeds the strength of the concrete due to concrete drying shrinkage, subgrade restraint, temperature/moisture differentials, applied traffic loads, and the combined effects of restrained curling and warping. It is highly desirable to control the location and geometry of transverse and longitudinal cracking in pavements by using properly designed and constructed joints. Without this control, cracking occurs in a random pattern similar to Figure 5G-1.02.

Figure 5G-1.02: Effect of Jointing on Crack Control



Crack Pattern Without Jointing



Properly Jointed Pavement

Cracking can be broken into two categories - initial and mature.

1. **Initial Cracking:** Initial cracking occurs within a few hours to a few months after the pavement has been placed. It may be caused by the following conditions.

   a. **Concrete Shrinkage (loss of volume):** Concrete shrinkage is caused by contraction of concrete from the following.

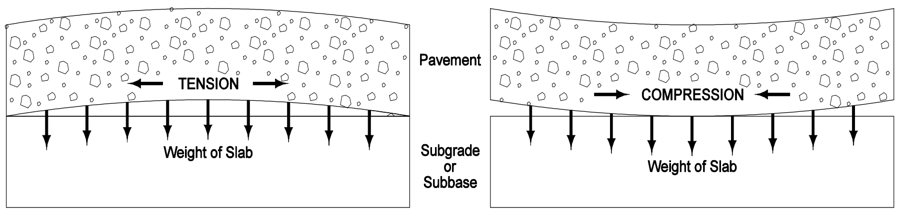
      1) **Temperature Change During Hydration:** The heat of hydration and temperature of pavement normally peak a short time after final set. After peaking, the temperature of concrete declines due to reduced hydration activity and lower air temperature during the first night of pavement life. As the temperature of concrete drops, the concrete contracts or shrinks. If severe air temperature changes occur within the first few hours after construction, high tensile stresses may cause transverse cracking to occur.

      2) **Loss of Water During Hydration (drying shrinkage):** Drying shrinkage results from the reduction of volume through loss of mix water. Concrete mixes for roadway applications require more mix water than is required for hydration (water consumed through chemical reactions with cement). The extra water helps provide adequate workability for placing and finishing operations. During consolidation and hardening, most of the excess water bleeds to the surface and evaporates. With the loss of the water, the concrete has less volume.

b. Subgrade and Subbase Restraint: Subgrade or subbase friction resists the contraction of the pavement from reduced volume and temperature. This resistance produces tensile stresses within the concrete.

c. Curling and Warping: Curling is the result of temperature changes through the depth of the pavement. Daytime curling occurs when the top portion of the slab is at a higher temperature than the bottom portion. Because of the higher temperature, the top expands more than the bottom, causing the tendency to curl. Subgrade and subbase friction and the weight of the slab are factors that help to counteract the daytime curling. During the night, the effects of curling are reversed. See Figure 5G-1.03.

Figure 5G-1.03: Daytime and Nighttime Curling



Daytime Curling Nighttime Curling

Warping results from a moisture differential from the top to the bottom of the slab. The top of the slab is normally drier than the bottom. The decrease in moisture content causes contraction at the top of the slab, which helps to counteract daytime curling. This contraction causes stresses in the concrete, which can lead to cracking.

2. **Mature Cracking:** Mature cracking occurs several months or years after pavement is placed. As traffic loads are applied to the pavement, along with temperature and moisture changes, tensile strain/stress will develop in concrete as the result of:

   a. Curling and warping in combination with repetitive traffic loads.

   b. Poorly designed and constructed pavement joints that do not provide proper load transfer across the joints and pavement slab.

   c. Poor foundation support due to unsuitable or non-uniform soils and excessive subgrade moisture.

C. Crack Control

Cracking can be minimized by the following:

1. Properly designed and constructed joints and joint layout that account for load transfer.
2. Proper timing of sawing of joints.
3. Sawing of joints in the correct locations.
4. Proper curing of concrete to prevent high initial shrinkage and cracking of hardening concrete.
5. Constructing a quality foundation with uniform, stable subgrade, drainable subbase, and longitudinal subdrains. See Chapter 6 - Geotechnical for additional guidance.

D. Considerations for Good Pavement Jointing

In order to design a suitable pavement jointing system, the following considerations have been included in the jointing layout steps covered in this design section. The following elements need to be considered for adequate jointing:

1. **Joint Purpose:** Transverse and longitudinal joints are used to control cracking of the pavement by relieving internal curing and loading stresses. Transverse joints serve to control cracking resulting from contraction of the pavement.

2. **Climate and Environmental Conditions:** Depending upon temperature and moisture changes that occur at the time of construction, expansion and contraction of the slab will occur, resulting in stress concentrations, warping, and curling.

3. **Slab Thickness:** Pavement thickness counteracts curling stresses and deflections. Thicker pavements are less prone to curling.

4. **Load Transfer:** Load transfer is desirable across any concrete pavement joint. However, the amount of load transfer provided varies for each joint type, aggregate interlock, and the type of bar.

5. **Joint Spacing vs. Thickness:** Maximum joint spacing is dependent on pavement thickness; thinner pavement requires closer joint spacing than thick pavement.

6. **Traffic:** Traffic is an extremely important consideration in joint design. Traffic classification, channelization, and, particularly, the amount of truck traffic influence the load transfer requirements for long-term performance.

7. **Concrete Material and Construction Characteristics:** Specific materials and their combinations can affect concrete strength and joint requirements. When special mixes outside of standard mixes are proposed to meet project conditions and requirements, the materials selected for the concrete can influence slab shrinkage. Substandard materials and construction practices can have a detrimental effect on joint performance. For example, poor coarse aggregate can lead to D-cracking, which initially occurs along pavement joints, or over-vibration can lead to low air content, which can lead to early deterioration of pavement joints.

8. **Subbase Type:** The support values and interface friction characteristics of different subbase types affect movement and support of the slabs.

9. **Shoulder Design:** The shoulder type (curb, tied concrete, asphalt, granular, or earth) affects edge support and ability of mainline joints to transfer load. Widened outer lanes are also effective for helping maintain load transfer.

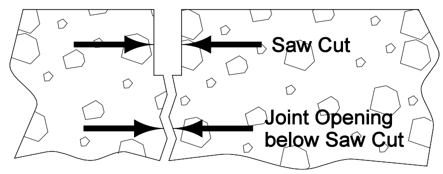
10. **Past Performance:** Performance observations and records can be used to establish standard joint design (what has worked and what has not).

E. Load Transfer

For jointed concrete pavement to perform adequately, traffic loadings must be transferred effectively from one side of the joint to the other. This is commonly referred to as load transfer, which is measured by joint effectiveness. If a joint is 100% effective, it will transfer approximately one half of the applied load. Field evaluation of load transfer is calculated by measuring the deflection on each side of a joint from the applied load. Load transfer is necessary for jointed concrete pavements to perform well. Adequate load transfer lowers deflections and reduces faulting, spalling, and corner breaks. Table 5G-1.01 shows that joint efficiency drops considerably when the joint opening below the sawcut line starts to exceed 1/8 inch.

Table 5G-1.01: Joint Opening Below the Saw Cut vs. Joint Efficiency

Joint Opening Below Saw Cut	Joint Efficiency
1/16”	> 50%
1/8”	< 50%
1/4”	0%



The following factors contribute to load transfer across joints:
- Aggregate interlock
- Mechanical load transfer devices
- Uniform, stable foundation, including quality subgrade and drainable subbase

1. **Aggregate Interlock:** Aggregate interlock is the interlocking action between aggregate particles at the face of the joint. It relies on the shared interaction between aggregate particles at the irregular crack face that forms below the sawcut. This form of load transfer has been found to be the most effective form of load transfer on streets with short joint spacings and low truck volumes. Increased aggregate interlock load transfer and minimized faulting will result from the following:

   a. Longitudinal tiebars and/or keyways.
      1) Typically used in longitudinal contraction and construction joints.
      2) Tiebars provide little load transfer themselves, but they do hold the slabs relatively tight together to maintain aggregate interlock.

   b. Shorter joint spacings (e.g. 15 feet or less).

   c. Larger crushed stone in the concrete mix
      1) Larger (greater than 1 inch) aggregates are helpful in maintaining load transfer, especially for larger joint openings.
      2) Generally, crushed stone aggregates perform better for aggregate interlock than rounded aggregates because the angular aggregates create a rougher joint face.

2. **Mechanical Load Transfer:** Aggregate interlock alone may not always provide sufficient load transfer in transverse joints for highway pavements and streets subject to heavy truck traffic. Under these circumstances, dowel bars should be used.

   a. Dowel Bar Benefits: Dowel bars are smooth round bars placed across joints to transfer loads without restricting horizontal joint movement. The benefits of dowel bars are as follows:
      1) They keep slabs in horizontal and vertical alignment.
      2) Since dowel bars span the joint, daily and seasonal joint openings do not affect load transfer across doweled joints as much as they do undoweled joints.

3) Dowel bars lower deflection and stress in concrete slabs, and reduce the potential for faulting, pumping, and corner breaks.
4) Dowel bars increase pavement life by effectively transferring the load across the joint.

b. Dowel Bar Use: Historically, dowel bars have been used to provide additional mechanical load transfer where traffic exceeds 200 trucks per day (or 100 trucks per lane), or accumulated design traffic exceeds 4 to 5 million ESALs. Typically, this truck traffic level will require at least an 8 inch thick slab, which is generally regarded as the minimum thickness to accommodate dowels.

3. **Quality Subgrades and Subbases:** A proper foundation for pavements reduces joint deflection, assists in aggregate interlock, and improves and maintains joint effectiveness under repetitive loads. Quality subgrades and subbases not only provide this support but also provide an all-weather working platform and stable smooth trackline for paving equipment. For design guidance for pavement subgrades and subbases, see Chapter 6 - Geotechnical.

It should be noted the subbase type and the subgrade support *k*-value have an effect on stresses in pavement slabs. The stiffer the foundation, the greater the slab's curl and warping stresses will be. Therefore, a shorter transverse joint spacing should be used.

4. Skewed Joints: Upon approval of the Jurisdictional Engineer, transverse contraction joints for undoweled pavements may be skewed counterclockwise (right ahead) 4 to 5 feet. Skewed joints are effective in decreasing the dynamic loading in the joint area by distributing the transfer of load. Each wheel on an axle crosses a skewed joint at a separate time. This reduces stresses and deflections in the concrete slab and helps reduce pumping and faulting. Also, the joints are 4 or 5 inches longer which increases the slab support area. The use of skewed joints is more appropriate for rural low volume roads and is not as practical for urban conditions due to the need to have right angle jointing patterns at intersections. A word of caution: If random cracks occur, they normally are at somewhat right angles and can create a pie shaped piece of pavement when they cross a skewed joint.

Types of Joints

A. Jointing

PCC pavement joints are necessary primarily to control the location of cracks that occur from natural and dynamic loading stresses. They accommodate stresses that develop from slab curling and warping due to moisture and temperature differentials and traffic loading. In addition, joints divide the pavement into suitable construction increments or elements. Standard design considerations include joint types, spacing, load transfer, and sealing. This section deals with the proper selection and layout of contraction, construction, and isolation joints.

B. Joint Spacing

Joint spacing for unreinforced concrete pavements depends on slab thickness, concrete aggregate, subgrade/subbase support, and environmental conditions. Transverse joint spacing should be limited to 24T (T is slab thickness) for pavements on subgrades and granular subbases or 21T if the pavement is placed on stabilized subbases, existing concrete, or asphalt. Transverse joint spacing is12 feet for pavements 6 inches thick, 15 feet for pavements 7 to 9 inches thick, and 20 feet for pavements over 9 inches thick. Longitudinal joint spacing for two lane streets, where lane delineation is not necessary, should be limited to a maximum of 10 feet. For multi-lane streets, where lane delineation is desired, longitudinal joint spacing is typically 10 to 13 feet. Generally, transverse joint spacing should not exceed 150% of the longitudinal joint spacing. Table 5G-2.01 provides transverse and longitudinal joint spacings for standard two lane streets.

Table 5G-2.01: Transverse Joint Requirements

Pavement Thickness	Transverse Joint Type	Transverse Joint Spacing
6”	C	12’
7”	C	15’
8”	CD[1]	15’
9”	CD[1]	15’
≥ 10”	CD[1]	20’

[1] No dowels within 24” of the back of curb

Source: SUDAS Specifications Figure 7010.901

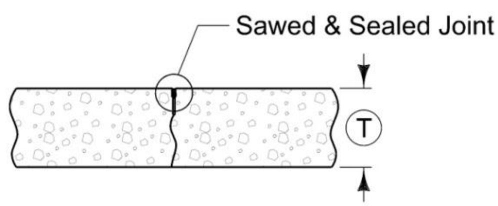
C. Joint Types

Contraction joints for concrete pavements are generally sawed. Transverse joints can be sawed with conventional sawing or early concrete sawing equipment. Longitudinal joints are formed with conventional sawing . Some joints, including construction joints, are formed. The figures in this subsection are derived from SUDAS Specifications Figure 7010.101.

1. **Transverse Contraction Joints:** Contraction joints constructed transversely across pavement lanes are spaced to control natural initial and mature cracking of the concrete pavement. Under certain conditions, such as rapidly dropping air temperature during the night, transverse cracks may occur early. Therefore, early formation of the transverse joints is required.

    a. **Plain Contraction Joints:** Plain contraction joints are normally used in local streets and minor collectors where load transfer is not a major factor. Load transfer for plain contraction joints occurs through the adjacent irregular fractured faces. Generally, they are used when the slab thickness is less than 8 inches. The joints are constructed by sawing to a depth of T/3. Plain contraction joints are sometimes used when the pavement thickness is 9 inches or greater such as at intersections in boxouts near curbs where load transfer is not a concern. Approved early concrete sawing equipment may be used to cut the joint to a depth of 1 1/4 inch. For sealing, the joint width must be a minimum of 1/4 inch wide.

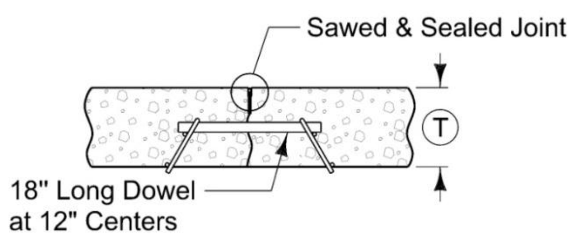
Figure 5G-2.01: 'C' Plain Contraction Joint



    b. **Doweled Contraction Joints:** Dowel bars are used to supplement the load transfer produced by aggregate interlock. The joints are sawed to a depth of T/3 and are spaced at 15 foot intervals for slab thickness of 9 inches or less and 20 feet for slabs greater than 9 inches thick. The dowels are placed at the mid-depth in the slab so they can resist shear forces as traffic loads cross the joint; thus helping reduce deflection and stress of the joint. The need for doweled contraction joints depends on subgrade/subbase support and the truck traffic loadings the roadway is to provide. They are usually used on streets or roadways where the pavement thickness is 8 inches or greater and where the pavement is subject to heavier truck traffic, generally more than 100 trucks per lane per day. Early entry concrete sawing can be used for 'CD' joints.

    Dowels should not be placed closer than 24 inches from the back of the curb on streets with quarter point or third point jointing. If gutterline jointing is used, place the first dowel in the traffic lane 6 inches from the joint.

Figure 5G-2.02: 'CD' Doweled Contraction Joint

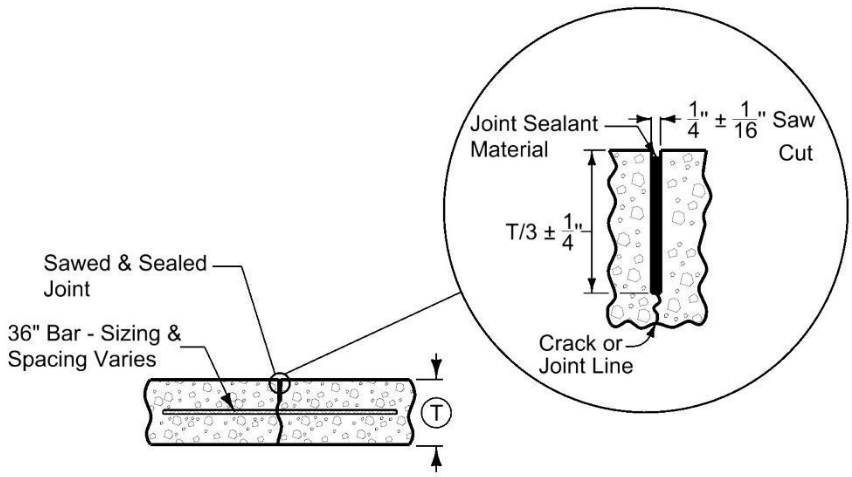


2. **Longitudinal Contraction Joints:** Longitudinal contraction joints release stresses from restrained warping and dynamic loading. Under certain conditions, such as rapidly dropping air temperature during the night, longitudinal cracks may occur early. Therefore, early formation of the joint is required.

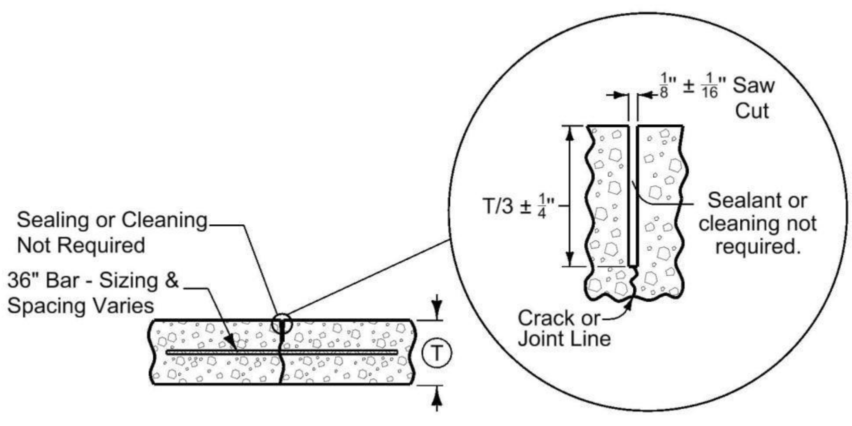
Typically, sawed longitudinal joints are sealed. However, since the slabs on either side of the longitudinal contraction joint are tied by a reinforcing bar, the Jurisdictional Engineer may approve not sealing the joint. The need to seal the joint is reduced due to the tied connection and the fact the joint will not open. The depth of cut for sawed longitudinal joints is T/3, regardless of the method of sawing used. The width of the sealed joints is 1/4 inch ± 1/16 inch. The maximum width of the unsealed joints is 1/8 inch ± 1/16 inch.

A longitudinal joint is usually placed at the center of the pavement to allow the pavement to hinge due to lane loading and help delineate separation of opposing traffic. Controlling cracking and proper constructability are the primary functions of longitudinal contraction joints. Lane delineation is a secondary function.

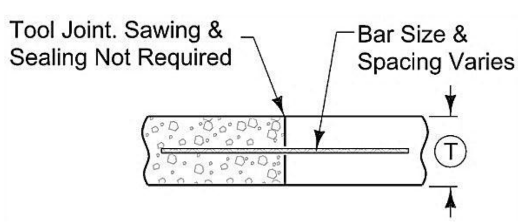
Figure 5G-2.03: Longitudinal Contraction Joints



'L' Joint with Sealant



'L' Joint without Sealant



'BT', Butt Joint

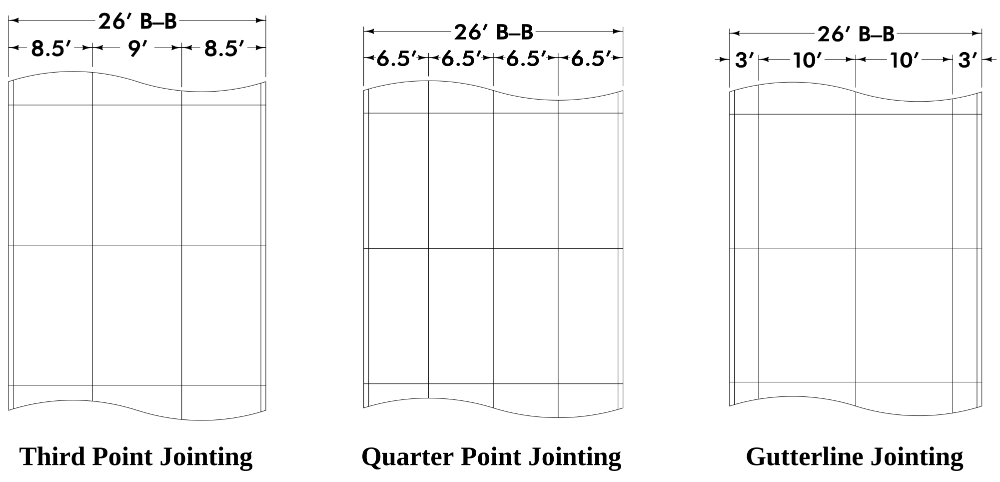
An important consideration when establishing the distance between longitudinal joints for jointed plain concrete pavements is the prevention of random longitudinal cracking at the quarter point, which is the midpoint between the centerline and the back of the curb. Pavements less than 9 inches thick may not crack through a longitudinal joint placed close to the gutter, which could cause longitudinal cracks at the quarter point. For this reason, it is preferred to use quarter point jointing for 31 foot wide pavements. Third point jointing, which eliminates the centerline joint, is frequently used for pavement narrower than 30 feet because of the narrower panel width and for 31 foot wide pavements with a depth greater than 8 inches. However, some jurisdictions desire a centerline joint and a gutterline joint, typically 3 to 3 1/2 feet from the back of curb. A gutterline joint should only be used if the pavement has depth of at least 9 inches or pavement widening is likely to occur.

The following examples depict jointing options for 26 foot and 31 foot wide pavements. The principles involved with jointing for these pavement widths can be extended to other pavement widths.

a. 26 Foot B-B Pavement: Three longitudinal joint options for 26 foot wide plain jointed concrete pavements are provided:

1) Third point jointing provides for a single 9 foot center panel with two joints, each 8 1/2 feet from the back of curb.
2) Quarter point jointing includes a centerline joint and two joints at the quarter points. This option is used when centerline crack control is desired.
3) Gutterline jointing provides two 10 foot lanes with a centerline joint and gutterline joints 3 feet from the back of curb. As stated above, care must be exercised with this option to prevent random cracking at the quarter point. This option is typically used for streets 9 inches or greater in thickness.

Figure 5G-2.04: 26 Foot B-B Pavement

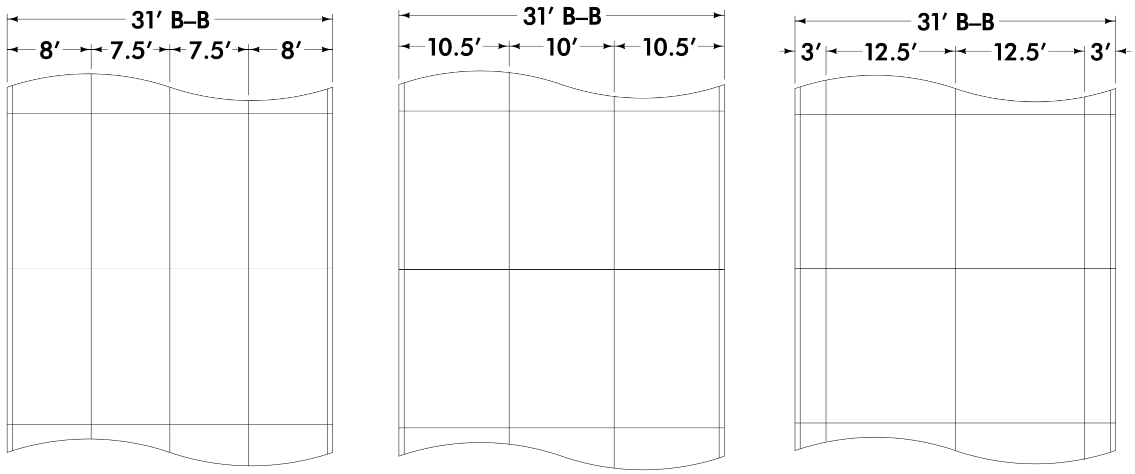


b. 31 Foot B-B Pavements: Three longitudinal joint options for 31 foot wide pavements are provided.

1) Quarter point jointing provides for a centerline longitudinal joint and two quarter point joints and is not intended to delineate driving lanes.
2) Third point jointing provides three nearly equally spaced panels, without a centerline joint. It typically is used as an option to quarter point jointing to minimize the number of longitudinal joints.
3) Gutterline jointing utilizes a centerline joint and gutterline joints 3 to 3 1/2 feet from the back of curb that delineate driving lanes.. This jointing pattern is typically used when the

pavement may be widened in the future, and the delineation of the lanes is desired. Care must be exercised with this option to prevent random cracking at the quarter point. Typically, gutterline jointing is used on streets with pavement thickness greater than or equal to 9 inches.

Figure 5G-2.05: 31 Foot B-B Pavements

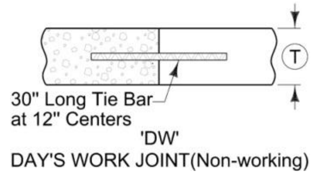
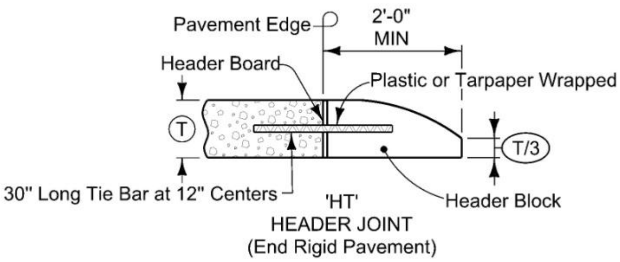
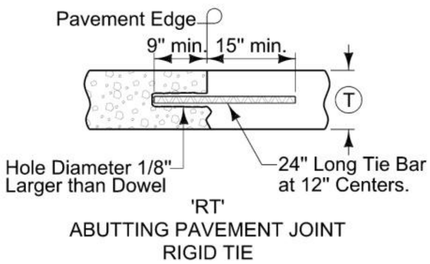
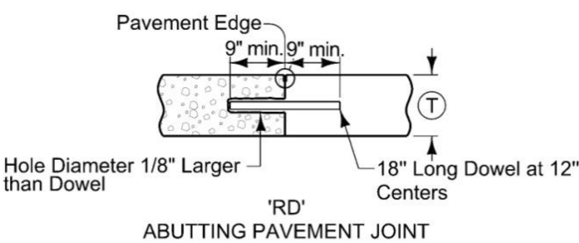
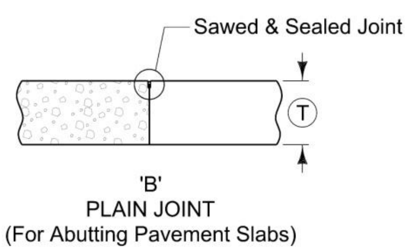


Quarter Point Jointing **Third Point Jointing** **Gutterline Jointing**

3. **Transverse and Longitudinal Construction Joints:** Construction joints are necessary for planned construction interruptions or widening/extending a pavement. Examples include construction of adjacent lanes at different times; box-outs for structures, radii, etc.; paving operation stoppages for over 30 minutes; and when a joint is needed between dissimilar materials. Construction joints are also used between an existing pavement and a new pavement. The joint is formed with the existing slab and is not sawed, except to accommodate joint sealing when required. Sawing and sealing of the joints are not required for those tied with deformed bars.

   a. **Transverse Construction Joints:** These types of joints are usually butt-type joints with deformed tie bars or dowels to provide load transfer and prevent vertical movement. When joint sealing is required, the depth of the saw cut (1 1/4 inches) is just deep enough to provide a reservoir for the joint sealant. The following are typical transverse construction joints.

Figure 5G-2.06: Transverse Construction Joints

 'DW' DAY'S WORK JOINT(Non-working)	Used at emergency (unplanned) stopping point. Ideally, it should be located at mid-panel, but it should not be located less than 5 feet from a planned contraction joint.
 'HT' HEADER JOINT (End Rigid Pavement)	Used when the pavement ends and traffic will cross the joint. The header is removed when the pavement is extended.
 'RT' ABUTTING PAVEMENT JOINT RIGID TIE	Typically used when an existing slab is extended.
 'RD' ABUTTING PAVEMENT JOINT	Functions as a CD joint when an existing slab is extended. Normally used when the pavement is 8 inches or greater in thickness.
 'B' PLAIN JOINT (For Abutting Pavement Slabs)	Typically used when two different pavement types or thicknesses abut or at the inside longitudinal edge of intake boxouts.

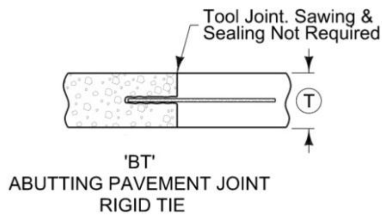
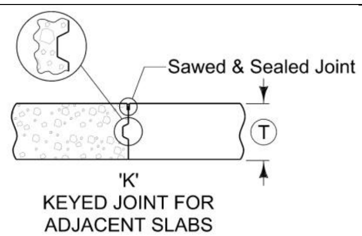
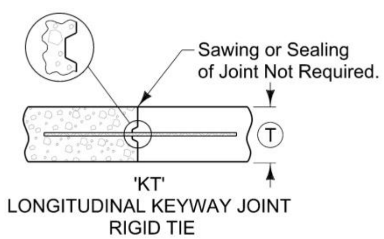
b. Longitudinal Construction Joints: These types of joints are used when adjacent lanes are constructed at different times. Tie-bars are primarily designed to resist horizontal movement but help with load transfer and vertical control. Under certain conditions, such as a drop in air temperature during the first night, longitudinal and transverse cracks may occur early. Early sawing of transverse joints is important when tied longitudinal construction joints are constructed in order to prevent the following two conditions from occurring.

1) **Sympathy Transverse Cracking in New Lane Construction:** When a new slab is longitudinally tied to an existing pavement, the existing transverse contraction joints can cause adjacent lane cracking in the new slab if early sawing of the transverse joints is not done. If there are transverse random cracks in an existing slab, the longitudinal

construction joint should be a plain butt joint or keyed joint (with no tie bars), if one exists in the old slab, to prevent sympathy cracks in the new pavement.

2) **Longitudinal Tie-bar Stress in Cooler Weather Conditions:** Care must be exercised to control cracking when utilizing longitudinal construction joints with tie bars, particularly in cool temperatures. For example, when a lane is constructed one day and the adjacent lane is constructed the following day or later, the existing lane could be expanding, particularly in the morning. If the new lane is in its final set (contracting) at the same time the existing pavement is expanding, stresses in the concrete at the tie bars can be significant. If the strength of the new concrete has not developed enough to resist the stresses, cracking could occur in the new concrete at the tie bars. During cooler weather conditions, care should be exercised when paving the new lane. Ideally, the new paving operation should take place at mid-day or later when the existing lane expansion is reduced.

Figure 5G-2.07: Longitudinal Construction Joints

	Used to tie existing and new parallel pavements together to prevent horizontal movements; and will provide some load transfer and resist vertical movement.
	Used when tie bars are not needed or desired and load transfer is required.
	Used in pavements under heavier traffic conditions and typically where the pavement thickness is 8 inches or greater. Also used at intersections under heavy turning movements where the pavement is less than 8 inches.

4. **Isolation Joints and Expansion Joints:** Expansion and isolation joints accommodate anticipated differential horizontal and vertical movements that occur between a pavement and structure. Their purpose is to allow movement without damaging adjacent structures or pavements. Contraction or control joints also absorb some movement; however, their main function is to control the location and geometry of the natural cracking pattern in the concrete slab. Because pavement performance can be significantly affected by the planned use and location of isolation and expansion joints, care should be taken in their design. Though the terms are sometimes used interchangeably, isolation joints are not expansion joints.

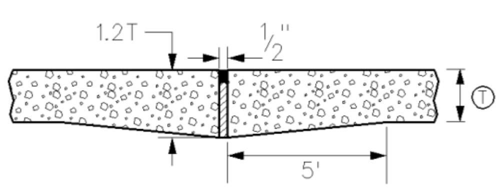
a. Isolation Joints: Isolation joints isolate the pavement from a structure, another paved area, or an immovable object. Isolation joints include full depth, full width joints found at bridge abutments, intersections, or between existing and new pavements. The term "isolation joint" also applies to joints around in-pavement structures such as drainage inlets, manholes, footings, and lighting structures. Isolation joints lessen compressive stresses that develop at T and unsymmetrical intersections, ramps, bridges, building foundations, drainage inlets, manholes, and anywhere differential movement between the pavement and a structure may take place. They are also placed adjacent to existing pavements, especially when it is not possible or desirable to match joint locations in the older pavement. Isolation joints should be 1/2 to 1 inch wide. Greater widths may cause excessive movement. They are filled with a pre-formed joint filler material to prevent infiltration of incompressibles.

At T-intersections, isolation joints should be used to isolate the T-intersecting street from the through street. Also, all legs of skewed streets should be isolated from the through street. Isolation joints used for this purpose should be placed one joint spacing back from the end of the intersection radii.

The joint filler material for expansion and isolation joints occupies the gap between the slabs and must be continuous from one pavement edge to the other and through curb and gutter sections. This filler material is usually a non-absorbent, non-reactive, non-extruding material typically made from either a closed-cell foam rubber or a bitumen-treated fiber board. No plug or sliver of concrete should extend over, under, through, around, or between sections of the filler, or it will cause spalling of the concrete. After the concrete hardens, the top of the filler may be recessed about 3/4 inch below the surface of the slab to allow space for the joint sealant to be placed later.

1) **Doweled Isolation Joints:** Isolation joints used at structures should have dowels to provide load transfer. The end of the dowel must be equipped with a closed-end expansion cap into which the dowel can move as the joint expands and contracts. The cap must be long enough to cover 2 inches of the dowel and have a suitable stop to hold the end of the cap at least the width of the isolation joint plus 1/4 inch away from the end of the dowel bar. The cap must fit the dowel bar tightly and be watertight. The half of the dowel with the capped end must be coated to prevent bonding and allow horizontal movement.
2) **Special Undoweled Isolation Joints:** Isolation joints at T and unsymmetrical intersections or ramps are not doweled so that horizontal movements can occur without damaging the abutting pavement. Undoweled isolation joints can be constructed with thickened edges to reduce the stresses developed at the slab bottom. The abutting edges of both pavements should be thickened by 20% starting with a taper 5 feet from the joint. The isolation filler material must extend completely through the entire thickened-edge slab.

Figure 5G-2.08: Thickened Edge Joint



a) **Undoweled Isolation Joints for Boxouts:** Isolation joints used at drainage inlets, manholes, and lighting structures do not have thickened edges or dowels.

b) Adjusting Isolation Joints for Utility Fixtures: After developing the jointing plan, plot any catch basins, manholes, or other fixtures that are within the intersection. Non-telescoping manholes will require a boxout or isolation joint to allow for vertical and horizontal slab movement. Consider using rounded boxouts to avoid crack-inducing corners. Also, for square boxouts, wire mesh or small-diameter reinforcing bars in the concrete around any interior corners will hold cracks tight should they develop. Telescoping manholes can be cast integrally within the concrete, and do not necessarily require a boxout. The multiple piece casting does not inhibit vertical movement and is less likely to create cracks within the pavement.

When a joint is within 5 feet of a fixture, it is desirable to adjust the joint so that it will pass through the fixture or the boxout surrounding the fixture. The following diagram shows several acceptable ways to skew or shift a joint to meet fixtures.

b. Expansion Joints: Expansion joints are defined as full depth, full width transverse joints placed at regular intervals of 50 to 500 feet (with contraction joints in between). This is an old practice that was used to relieve compressive forces in pavement. Unfortunately, this practice often caused other problems in the pavement such as spalling, pumping, faulting, and corner breaks.

Good design, construction, and maintenance of contraction joints has virtually eliminated the need for expansion joints, except under special conditions. In addition to the problems listed above, the improper use of expansion joints can lead to high construction and maintenance costs, opening of adjacent contraction joints, loss of aggregate interlock, sealant failure, joint infiltration, and pavement growth. By eliminating unnecessary expansion joints, these problems are removed and the pavement will provide better performance.

Pavement expansion joints are only needed when:

1) The pavement is divided into long panels (60 feet or more) without contraction joints in between to control transverse cracking.
2) The pavement is constructed while ambient temperatures are below 40°F.
3) The contraction joints are allowed to be infiltrated by large incompressible materials.
4) The pavement is constructed of materials that in the past have shown high expansion characteristics.

Under most normal concrete paving situations, these criteria do not apply. Therefore, expansion joints should not normally be used (PCA, 1992).

Figure 5G-2.09: Typical PCC Joint Layout at Manholes
(SUDAS Specifications Figure 7010.103)

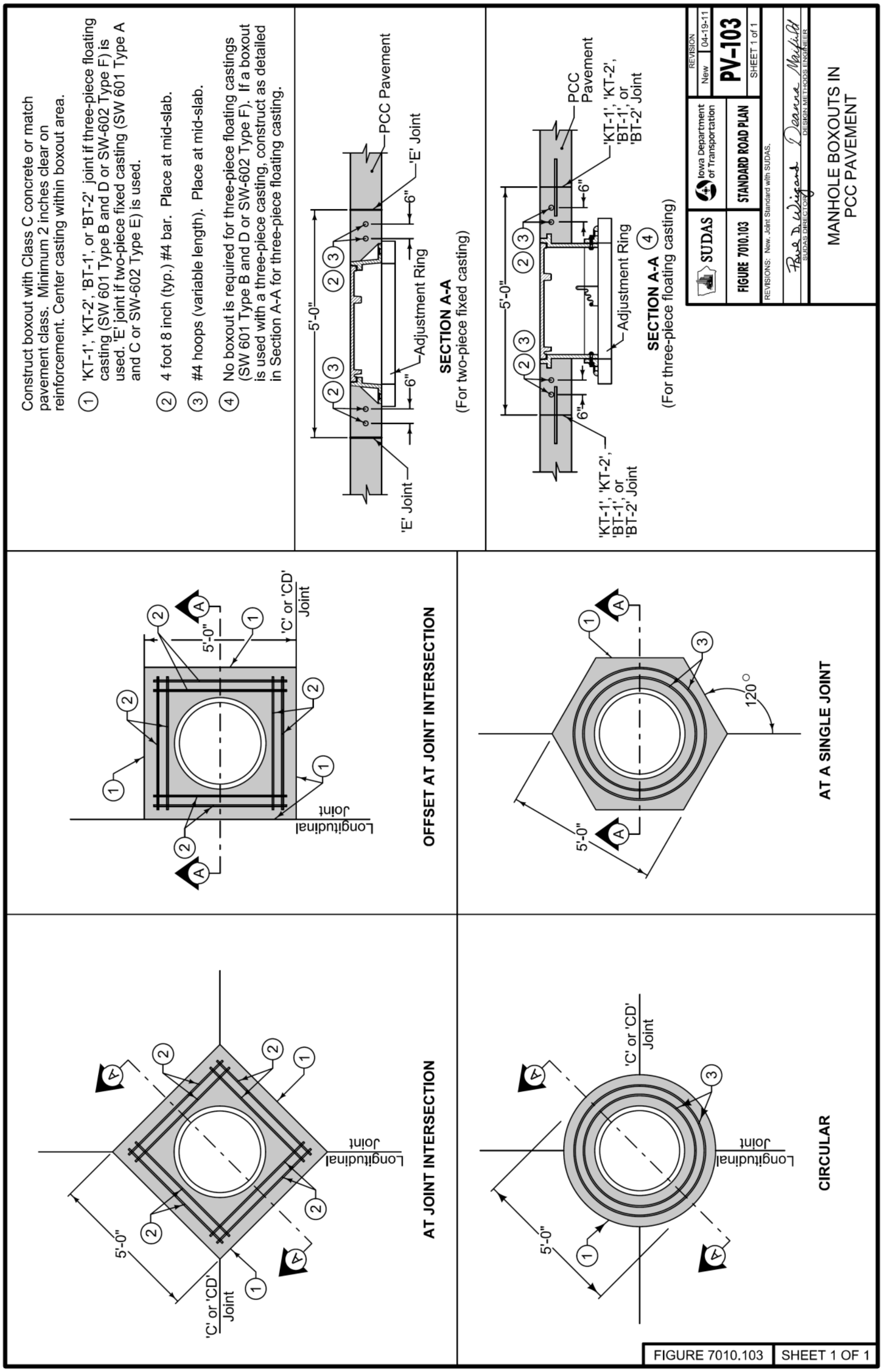
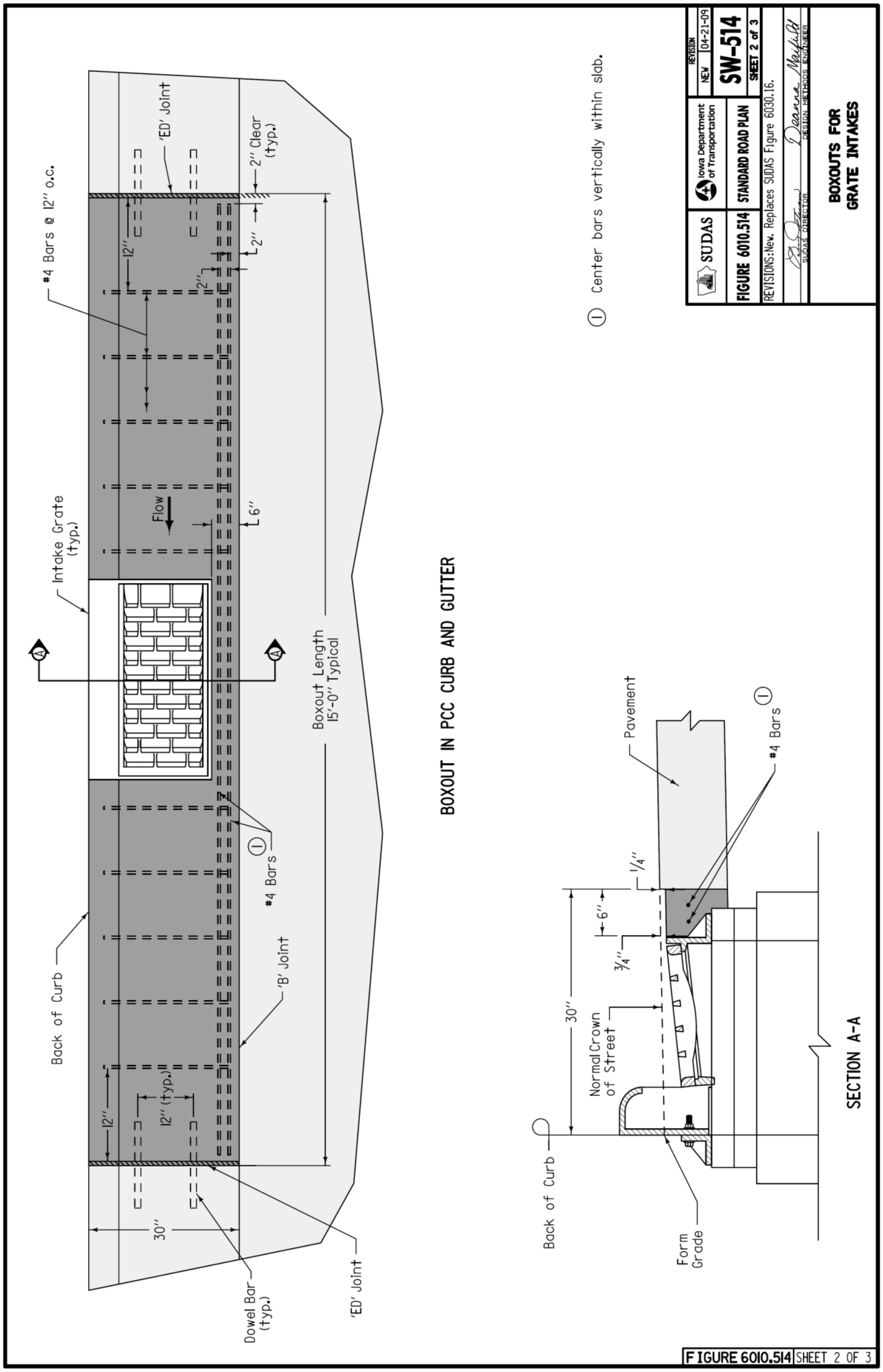
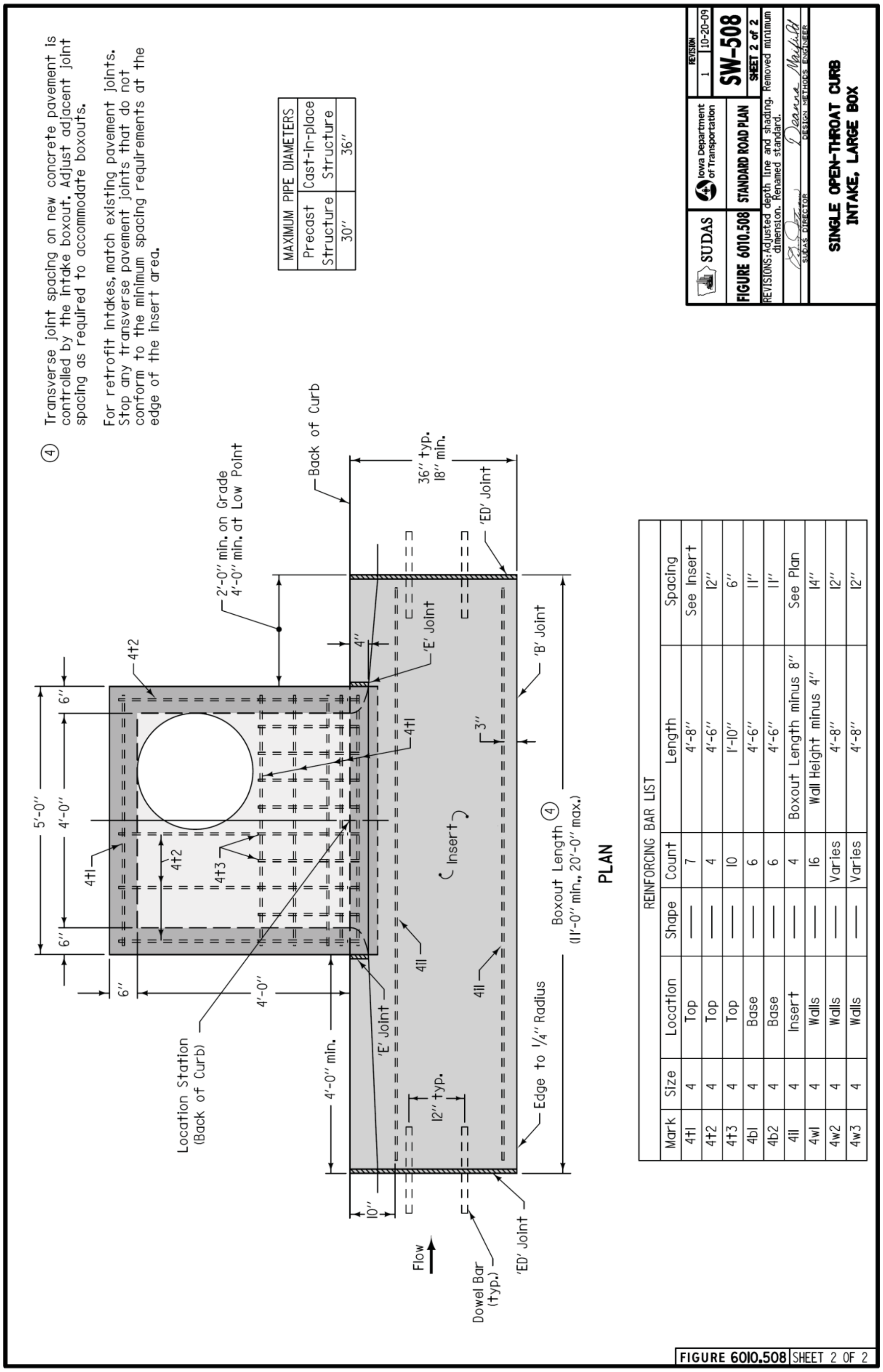


Figure 5G-2.10A: Typical PCC Joint Layout at Intakes - Boxout for Grate Intakes
(SUDAS Specifications Figure 6010.514, sheet 2*)



* SUDAS Specifications Figure 6010.514, sheets 1 and 3 include more boxout options.

Figure 5G-2.10B: Typical PCC Joint Layout at Intakes - Boxout for Open-throat Curb Intakes (SUDAS Specifications Figure 6010.508, sheet 2*)



* SUDAS Specifications Figure 6010.508, sheet 1 includes more information.

Table 5G-2.02: Summary of Joints
(Derived from the Iowa DOT Design Manual, Section 7A-2, Tables 1 and 2)

	Type			Method of Load Transfer				Thermal Movement				
Joint	Transverse	Longitudinal	Isolation/Expansion	Aggregate Interlock	Key	Tie Bar	Dowel Bar	Doweled to allow movement	Tied to prevent movement	Isolation/Expansion joint allows movement	Lack of reinforcing allows movement	**Comments**
B	x	x									x	Used between dissimilar materials or when other joints are not suitable.
C	x			x							x	Transverse joint used when T < 8 inches.
CD	x			x			x	x				Transverse joint used when T ≥ 8 inches.
CT	x			x		x			x			Specialty tied contraction joint.
DW	x					x			x			Used by contractor as a stopping point.
HT	x					x			x			Used at the end of rigid pavement prior to placement of second slab.
RD	x						x	x				Joint between new and existing pavements, dowels are used.
RT	x					x			x			Joint between new and existing pavements, tie bars are used.
BT-1		x							x			Longitudinal joint used when T < 8 inches, interchangeable with L-1 depending on paving sequence.
BT-2		x							x			Used when L-2 and the KT-2 are not possible, T ≥ 8 inches.
BT-3		x							x			Joint used between new and existing pavements. Tie bars are used when T ≥ 8 inches.
BT-4		x							x			Joint used between new and existing pavements. Tie bars are used when T ≥ 8 inches.
BT-5		x							x			Joint used between new and existing pavements. Tie bars are used when T < 8 inches.
K		x			x						x	T > 8 inches, minimal usage.
KS		x			x				x			Used in reinforced pavements.
KT-1		x			x				x			Longitudinal joint used when T < 8 inches, interchangeable with L-1 depending on paving sequence.
KT-2		x			x				x			Longitudinal joint used when T ≥ 8 inches, interchangeable with L-2 depending on paving sequence.
KT-3		x			x				x			Longitudinal joint used when T ≥ 8 inches, interchangeable with L-3 depending on paving sequence.
L-1		x		x					x			Longitudinal joint used when T < 8 inches, interchangeable with BT-1.
L-2		x		x					x			Longitudinal joint used when T ≥ 8 inches, interchangeable with KT-2 depending on paving sequence.
L-3		x		x					x			Longitudinal joint used with pavements of large width, interchangeable with KT-3 depending on paving sequence.
CF	x		x							x		4 inch expansion joint.
E	x	x	x							x		1 inch expansion joint.
ED	x		x				x	x		x		1 inch doweled expansion joint.
EE	x		x				x	x		x		2 inch doweled expansion joint.
EF	x		x				x	x		x		4 inch doweled expansion joint.
ES			x							x		Used in curb to match expansion joint in pavement.

D. Transverse Dowel Bar Size and Length

Table 5G-2.03 reflects the dowel bar size and length based on the pavement thickness. This information was obtained from the Portland Cement Association, the American Concrete Paving Association, and American Highway Technology. The SUDAS and Iowa DOT Specifications call for dowels when the slab is 8 inches or greater. Dowels are typically set at 12 inch spacing. The designer should note that a dowel bar that is too small induces high bearing stresses and causes the concrete matrix around the dowel to deteriorate or elongate. Elongation of the dowel bar hole then reduces the load transfer capabilities. Under special circumstances, smaller diameter and different shaped dowel bars may be used in thinner slabs.

Table 5G-2.03: Dowel Bar Size and Length

Pavement Thickness (inches)	Dowel Size (diameter in inches)	Dowel Length (inches)
8	1 1/4	18
9	1 1/4	18
10	1 1/2	18
11	1 1/2	18
12	1 1/2	18

E. Jointed Reinforced Concrete Pavements

Jointed reinforced concrete pavements (JRCP), sometimes referred to as distributed steel reinforcing, are not commonly used in Iowa jurisdictions. However, variations of JRCP are used effectively by several jurisdictions in Iowa. Therefore, the following is provided as an explanation of JRCP.

JRCPs utilize bar mats between transverse joints. Typically, the bar mats extend full width across the pavement, but with traditional JRCPs, they do not extend through the transverse joints. JRCPs use many of the same types of joints as jointed plain concrete pavements (JPCP), but the tie bars for longitudinal joints are replaced with the bar mats. Transverse joints, including doweled joints, are the same for both types of pavements since the bar mats of traditional JRCP do not extend through the transverse joints. Because of the bar mats, transverse joint spacing can be much longer than with JPCP, usually 27 feet to 45 feet. JRCP should not be confused with continuously reinforced pavement, which has very few or no joints.

JRCPs are used primarily to control cracking of concrete pavements, to provide for load transfer between joints, and to maintain the structural integrity of the slab between transverse joints. Just like JPCPs, random cracking of JRCPs may still occasionally occur even though the steel is present. The steel serves to hold the cracks close together, thus preventing the progressive opening of the cracks over time.

The added cost of the additional reinforcement for JRCPs is often offset by specifying a somewhat thinner slab. However, as pointed out by the American Concrete Institute (ACI), “the use of reinforcing steel will not add to the load-carrying capacity of the pavement nor compensate for poor subgrade preparation or poor construction practices.” By holding random cracks tightly closed, it will maintain the shear resistance of the slab, and, consequently, will maintain its load carrying capacity. This improves the ride when the vertical displacement is controlled.

As mentioned previously, several jurisdictions in Iowa specify a variation of JRCP. The Iowa variations of JRCP typically include extending the longitudinal reinforcing bars through the ‘C’ plain transverse contraction joints. When ‘CD’ doweled transverse joints are specified, the longitudinal reinforcement does not extend through the transverse joints. In addition, the transverse joint spacing is generally not lengthened as described for traditional JRCPs and follows the same guidelines as for JPCP. Figures 5G-2.11 and 5G-2.12 illustrate JRCP details typically used in Iowa.

Figure 5G-2.11: Iowa Jointed Reinforced Pavement Detail - 26’ Back-To-Back Street (formerly SUDAS Specifications Figure 7010.1F)

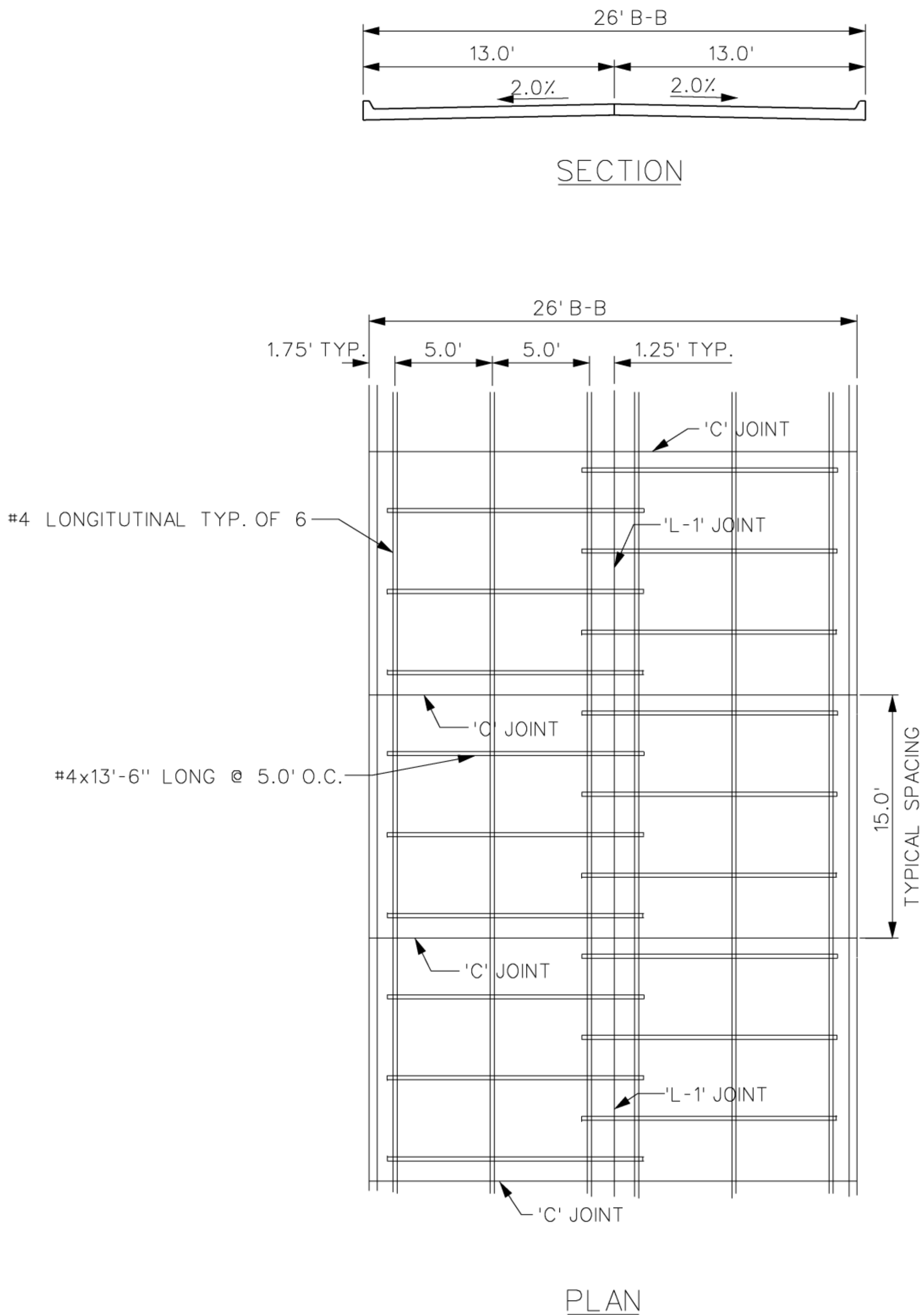
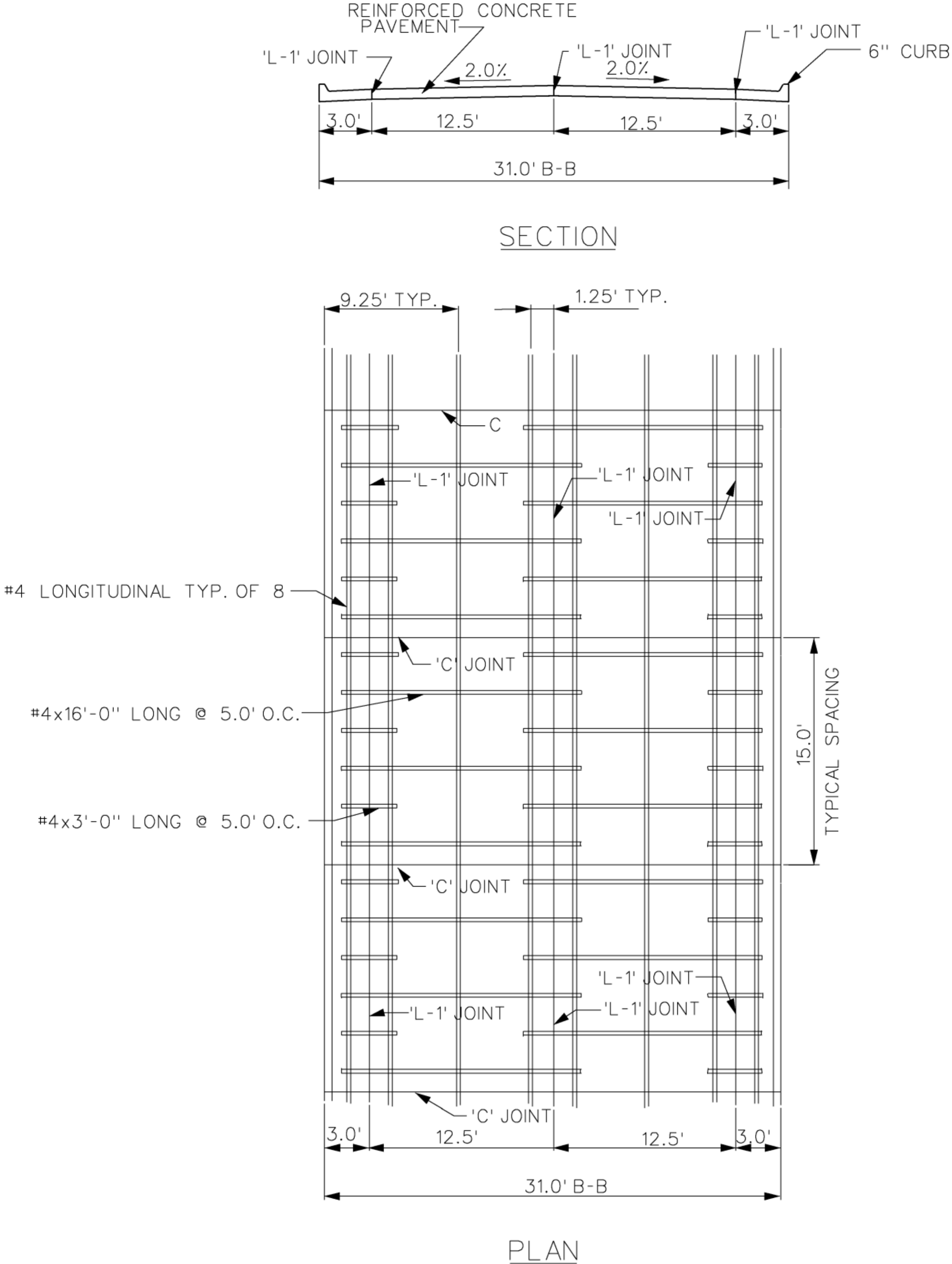
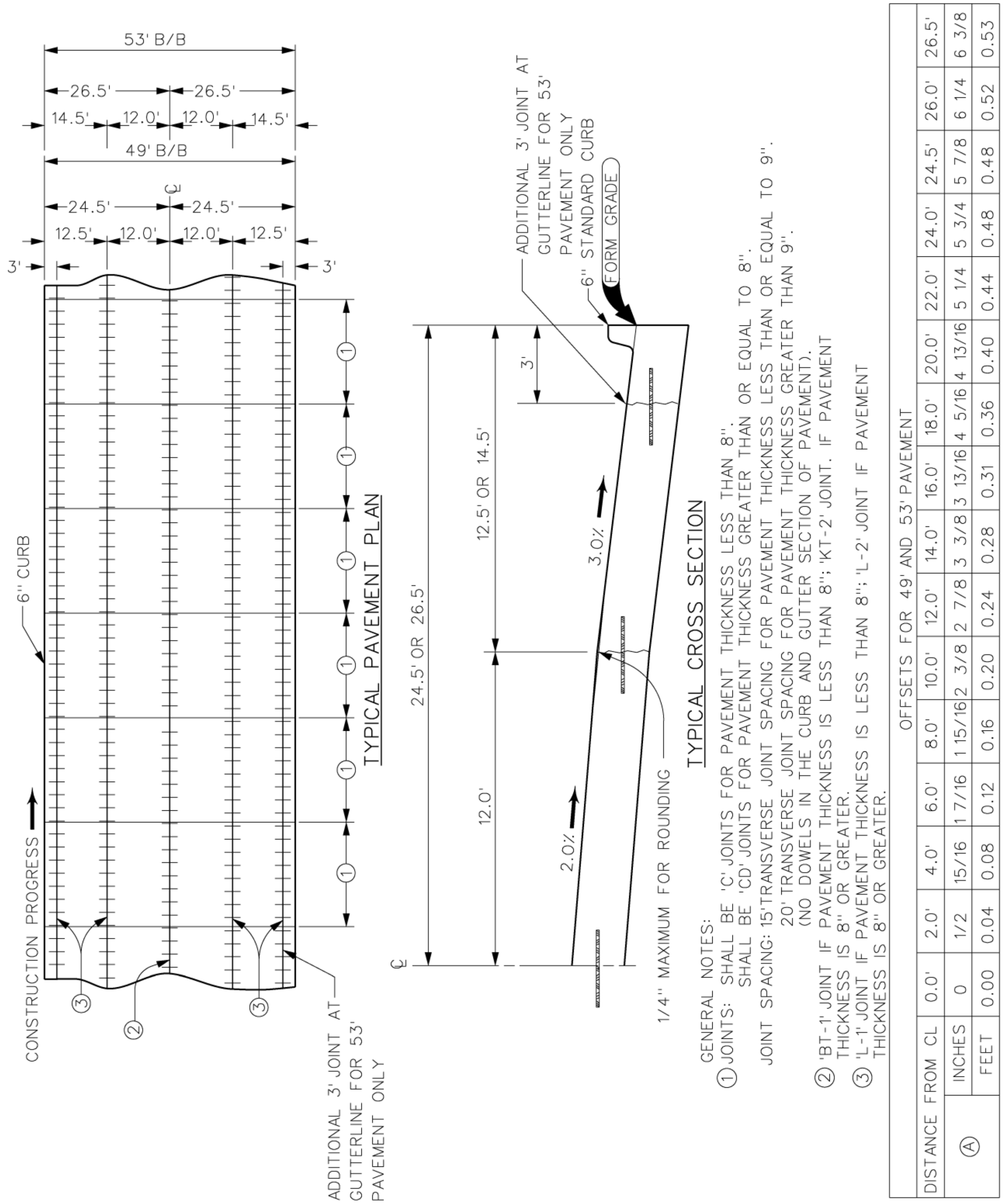


Figure 5G-2.12: Iowa Jointed Reinforced Pavement Detail - 31' Back-To-Back Street (formerly SUDAS Specifications Figure 7010.1G)



F. Miscellaneous PCC Pavement Jointing Figures

Figure 5G-2.13: 49’ B/B and 53’ B/B PCC Pavement Jointing and Crown Detail
(formerly SUDAS Specifications Figure 7010.1C)



	DISTANCE FROM CL	0.0'	2.0'	4.0'	6.0'	8.0'	10.0'	12.0'	14.0'	16.0'	18.0'	20.0'	22.0'	24.0'	24.5'	26.0'	26.5'
Ⓐ	INCHES	0	1/2	15/16	1 7/16	1 15/16	2 3/8	2 7/8	3 3/8	3 13/16	4 5/16	4 13/16	5 1/4	5 3/4	5 7/8	6 1/4	6 3/8
	FEET	0.00	0.04	0.08	0.12	0.16	0.20	0.24	0.28	0.31	0.36	0.40	0.44	0.48	0.48	0.52	0.53

Figure 5G-2.14: 49’ B/B and 53’ B/B C&G/HMA Pavement
(formerly SUDAS Specifications Figure 7020.2B)

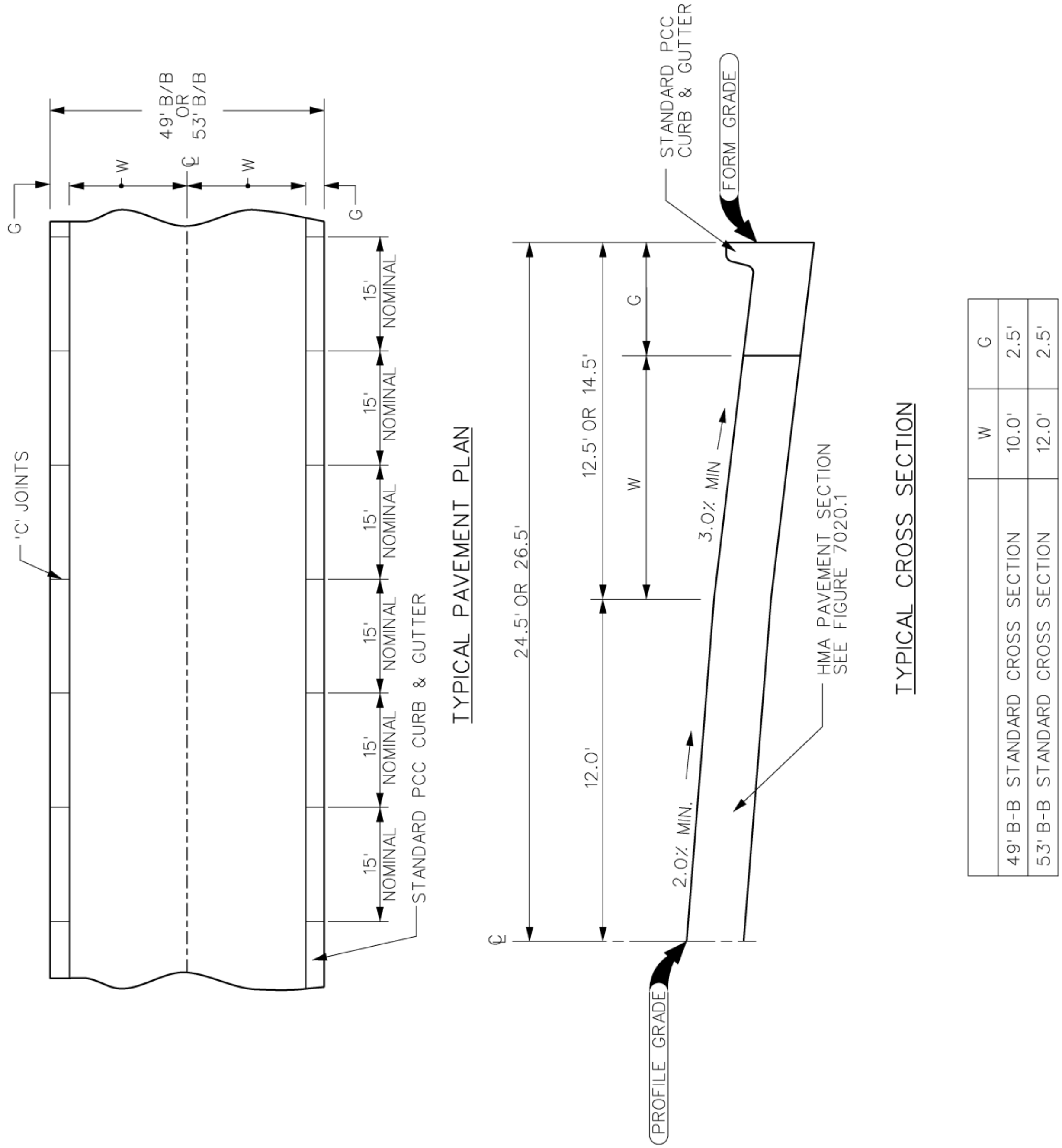


Figure 5G-2.15: 24’ Rural PCC Pavement Jointing and Crown Detail
(formerly SUDAS Specifications Figure 7010.1D)

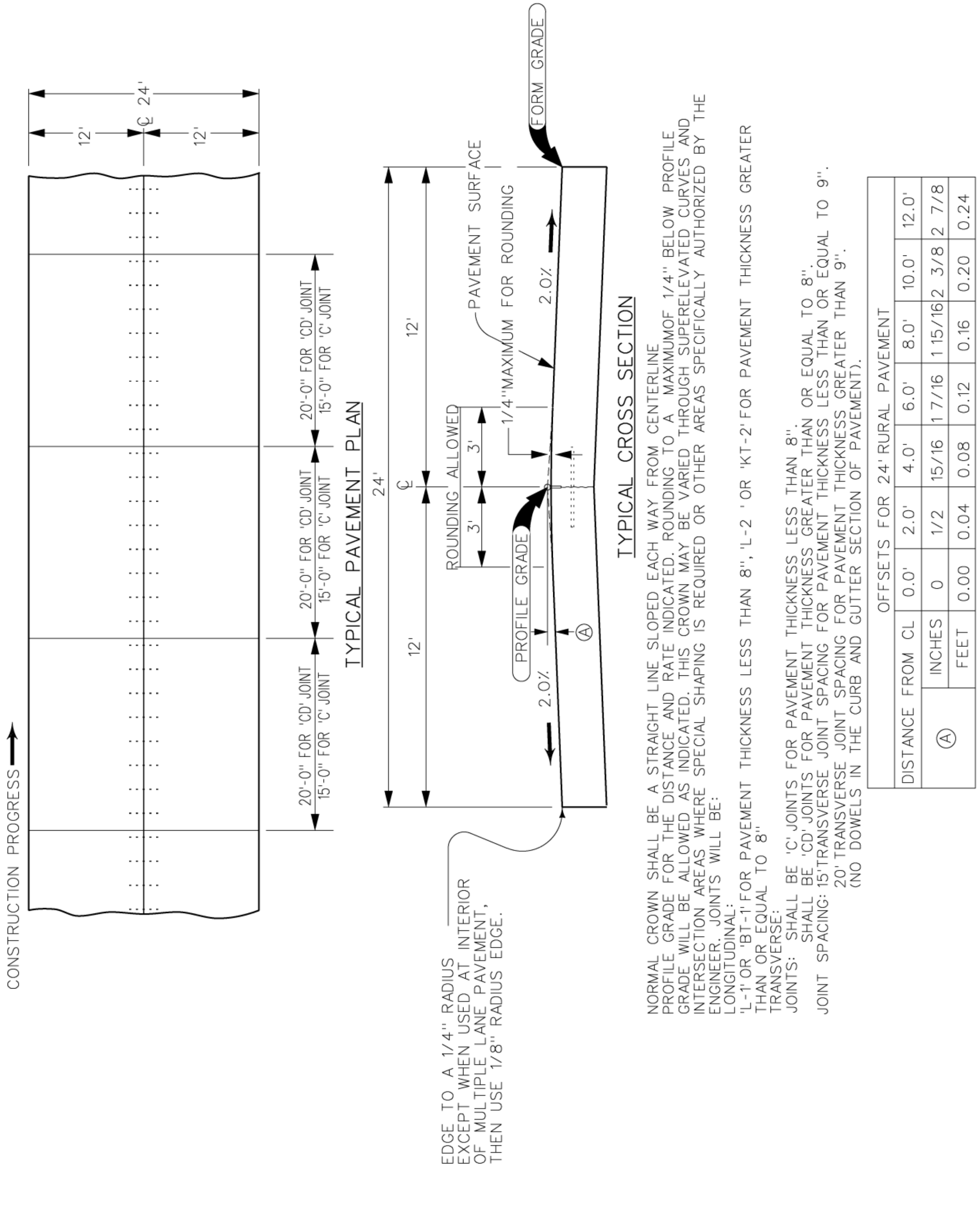
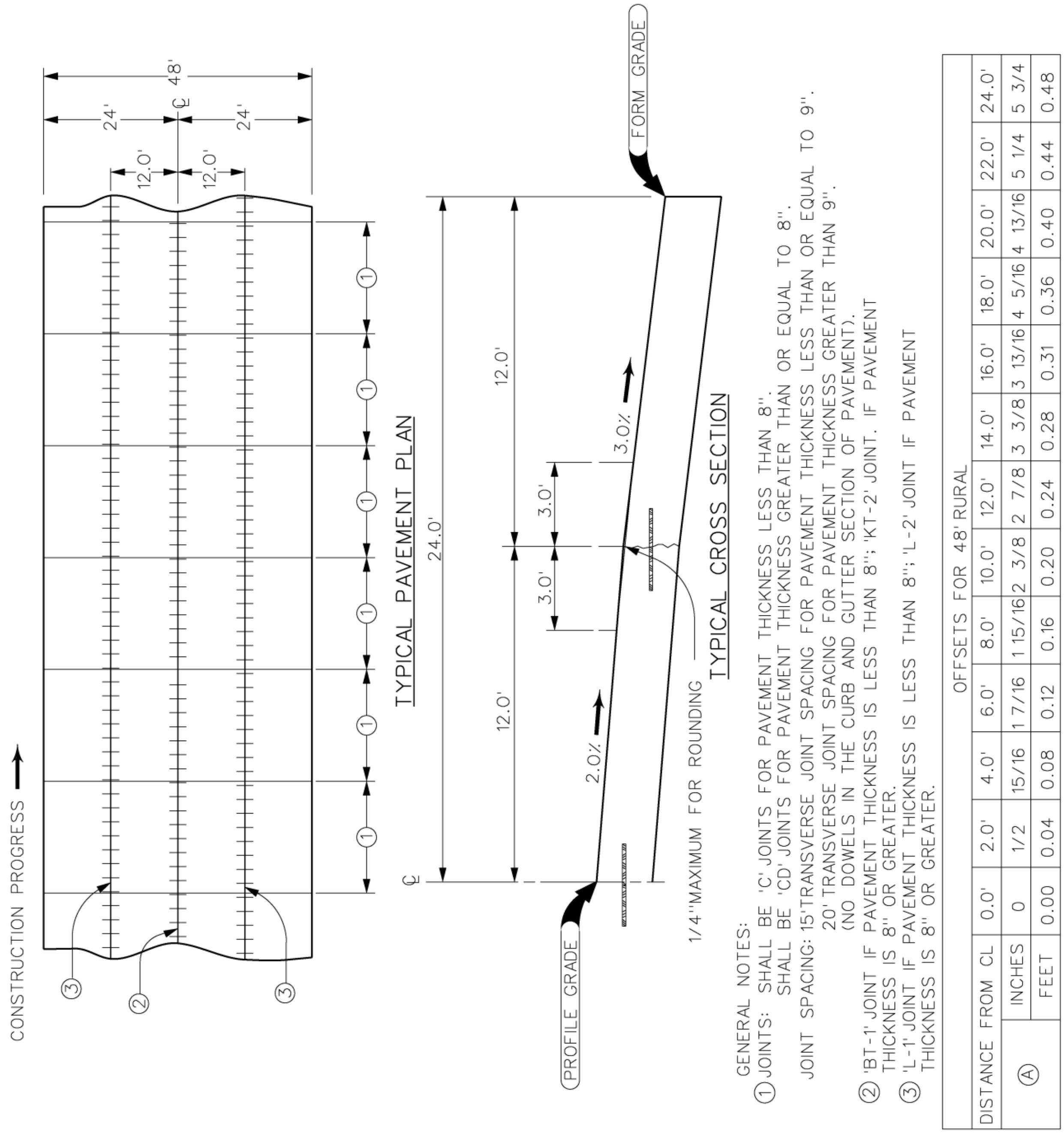


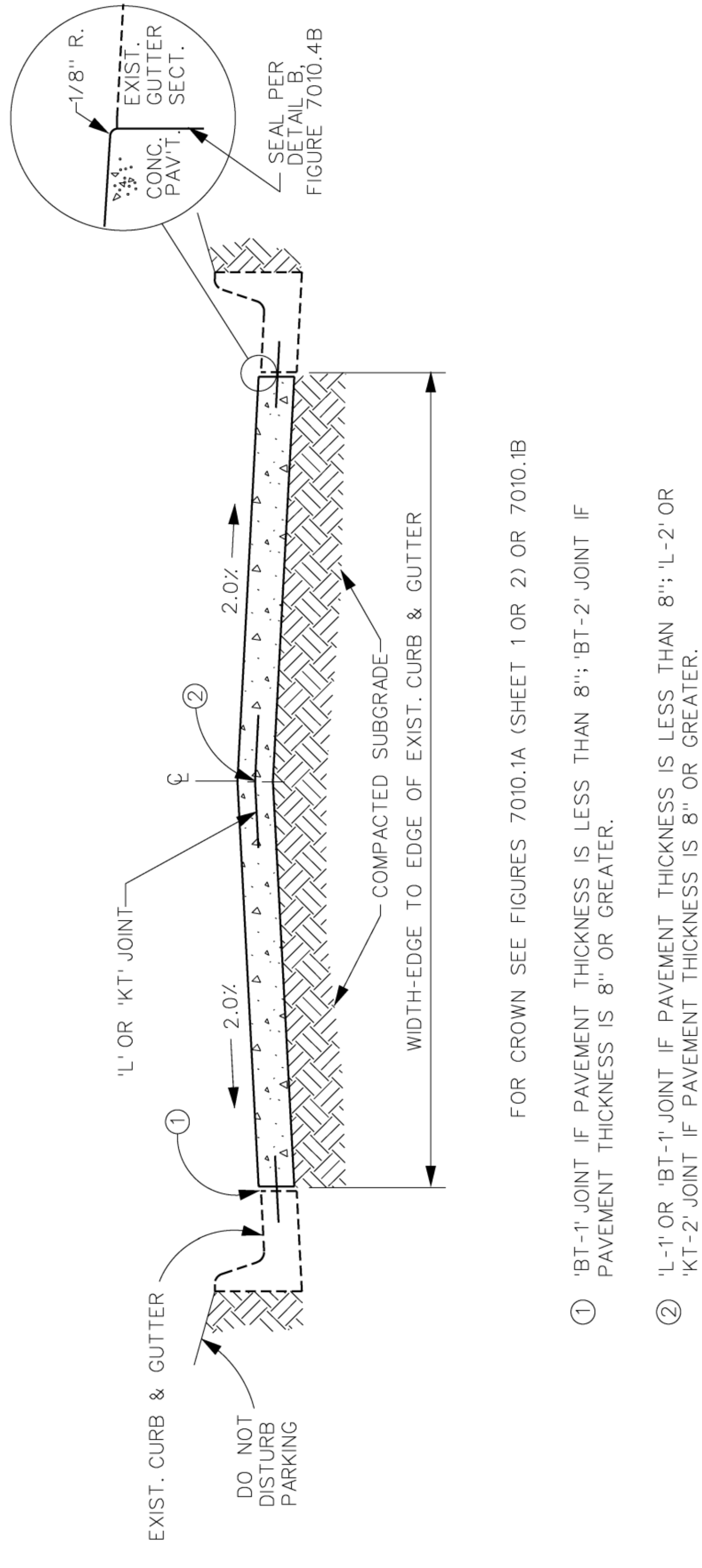
Figure 5G-2.16: 48' Rural PCC Pavement Jointing and Crown Detail
(formerly SUDAS Specifications Figure 7010.1E)



OFFSETS FOR 48' RURAL

DISTANCE FROM CL		0.0'	2.0'	4.0'	6.0'	8.0'	10.0'	12.0'	14.0'	16.0'	18.0'	20.0'	22.0'	24.0'
A	INCHES	0	1/2	15/16	1 7/16	1 15/16	2 3/8	2 7/8	3 3/8	3 13/16	4 5/16	4 13/16	5 1/4	5 3/4
	FEET	0.00	0.04	0.08	0.12	0.16	0.20	0.24	0.28	0.31	0.36	0.40	0.44	0.48

Figure 5G-2.17: PCC Pavement Section Between Existing Curb and Gutter
(formerly SUDAS Specifications Figure 7010.5)



G. References

Portland Cement Association. *Portland Cement Association Manual*.1992.

Jointing Urban Intersections

This section describes examples on how to joint urban intersections. The process will be illustrated through examples of different types of streets, pavement thickness, and intakes. Even though not all urban intersections will be exactly like the one used in these examples, the process described is applicable to other layouts.

During construction, it is likely that location changes will be necessary for some joints within an intersection. The primary reason is to ensure that joints pass through fixtures like manholes or drainage inlets that are embedded in the pavement. As a result, it will be desirable for the construction crew to adjust the location of some joints so they coincide with the actual location of a nearby manhole. The designer should consider placing a note on the plans to give the field engineer and contractor the latitude to make appropriate adjustments.

It is common practice for some designers to leave intersection joint layout to the field engineer and contractor. These designers often justify this practice by citing the many field adjustments that occur during construction, which they contend negates the usefulness of a jointing plan. However, it is not desirable to eliminate the jointing plan except for very simple intersections. A jointing plan and appropriate field adjustments are both necessary for more complex intersections because islands, medians, and turning lanes complicate joint layout and require some forethought before construction. The jointing plan will also enable contractors to more accurately bid the project.

Example: This example is an intersection of a multi-lane street and a two-lane side street. The intersection is curbed, includes several intakes, and the pavement thickness is 10 inches.

Step 1: Set Joints with Predetermined Locations

Because the location of longitudinal joints for both streets is normally predetermined, these joints should be set first.

Within the intersection, the street that is paved first determines which joints are longitudinal and which are transverse. Generally, the mainline street will be paved prior to the side street. Therefore, the longitudinal joints running down the side street define the locations of the first transverse joints for the mainline (see Figure 5G-3.01).

To determine an appropriate longitudinal joint to use, refer to SUDAS Specifications Figure 7010.901. The type of joint used may depend on the pavement thickness. Since the pavement thickness is greater than 8 inches in this case, either a KT-2 or an L-2 joint is appropriate.

Step 2: Locate Difficult Joints

Intake locations and the boxouts at the corner radii of the intersection are addressed next. After joints have been placed at these locations, the rest of the joints can be worked in around them.

1. **Joints at Intakes:** The location of intakes is determined before the joints are laid out, so joints have to be worked in around them. To start out with, straddle the intake with two transverse joints spaced according to the standard joint length. These joints can be repositioned later if it

helps with the placement of other joints. In the final layout, the intake should be centered between the joints, and adjacent joints should be adjusted accordingly. See the appropriate intake boxout figure in SUDAS Specifications Section 6010 for boxout length requirements.

CD joints should be used on the mainline since the pavement thickness is greater than 8 inches. However, the CD joints straddling the intake do not extend all the way through the curb and gutter. The joints immediately surrounding the intake are specified on the detail plates and are shown in the example.

2. **Joints at Boxouts:** Before the mainline is paved, small areas near the corners are boxed-out. These boxed-out areas (shaded in Figure 5G-3.01) are poured later, after the mainline has been paved. If the paver were to proceed straight through this area, instead of using boxouts, the returns of the city street would narrow to a point where they meet the mainline. Pavement less than 2 feet in width is weak and cracks readily. By using boxouts, this situation can be avoided without the expense of stopping the paver at the intersection.

   Although the width of boxouts is normally the same as the roadway's gutter width, the size and shape of boxouts varies depending on where they are used. If placing joints around the boxout, remember to maintain intersecting angles greater than 70 degrees and joints at least 2 feet long. KT-2 or L-2 joints are used around the boxout. Figure 5G-3.01 illustrates joints properly placed, both around the boxouts and extending outward from them.

Step 3: Locate Remaining Joints

After the joints at intakes and boxouts are located, the remaining joints (generally transverse joints) are located in appropriate locations. The maximum spacing for CD joints is 20 feet (greater than 9 inch pavement) and the minimum spacing is typically 12 feet. Therefore, the remaining areas on the mainline that need transverse joints should have CD joints spaced within this range. Since the design year truck volume on the adjoining street is less than 200 vpd, C transverse joints are used there.

In Figure 5G-3.02, the C joints on the city street nearest the corners are skewed perpendicular to the free edge of the pavement. If this joint were carried straight through, instead of skewed, the acute angle between the joint and the free edge of the pavement would be less than 70 degrees, which is not acceptable.

After all joints are located, the layout should be checked to ensure that all joint spacings and angles are acceptable. Figure 5G-3.02 shows all of the transverse joints appropriately located.

Step 4: Label Joints

The completed jointing layout of the intersection is shown in Figure 5G-3.02. For pavements 8 inches or greater, the L-2 and KT-2 joints may be used interchangeably, at the contractor's discretion, depending on the paving sequence. Therefore, the designer may identify the longitudinal joints as either L-2 or KT-2 on the jointing layout.

It is not necessary to identify every joint on the jointing layout. A few key joints on the diagram should be identified and whenever a series of joints changes to a different type of joint, the joint at the location of the change should be identified. Also, any joint that may be a source of confusion should be identified.

Joint lengths are also shown on the jointing layout, normally rounded to the nearest foot. Similar to labeling joint types, not every length needs to be identified. However, any length that cannot be inferred from the diagram should be labeled.

Figure 5G-3.01: Locating Predetermined and Difficult Joints

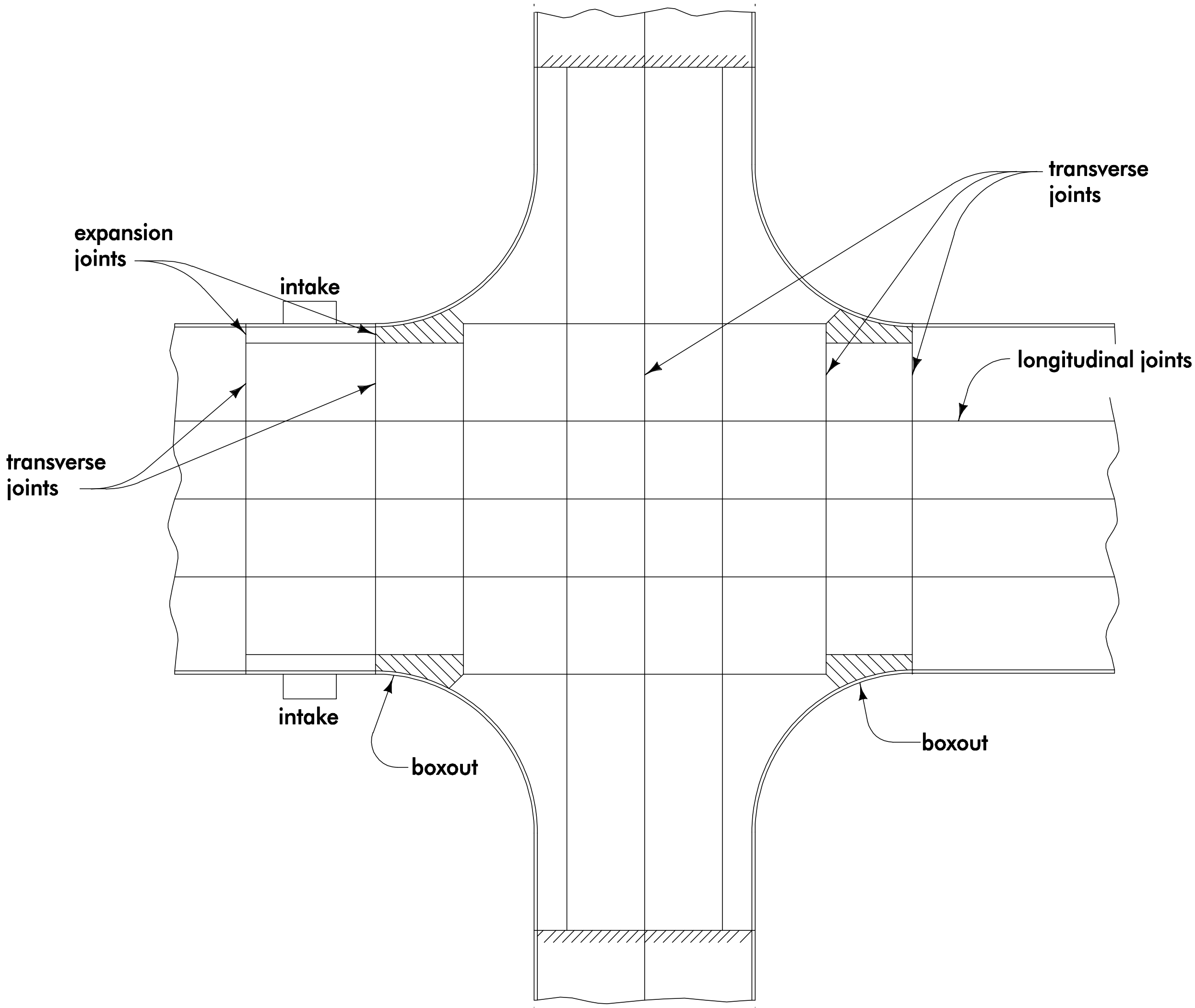
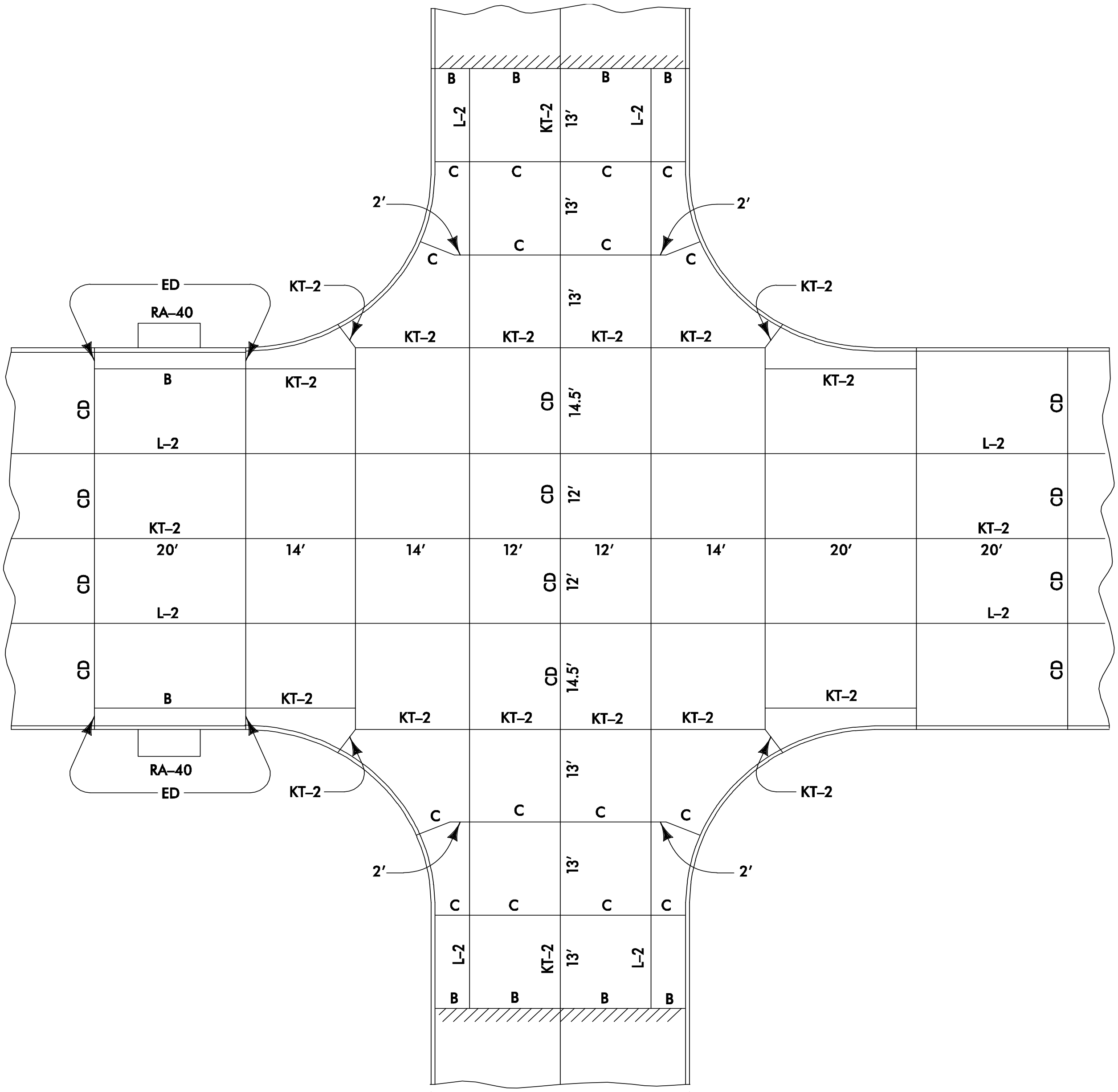
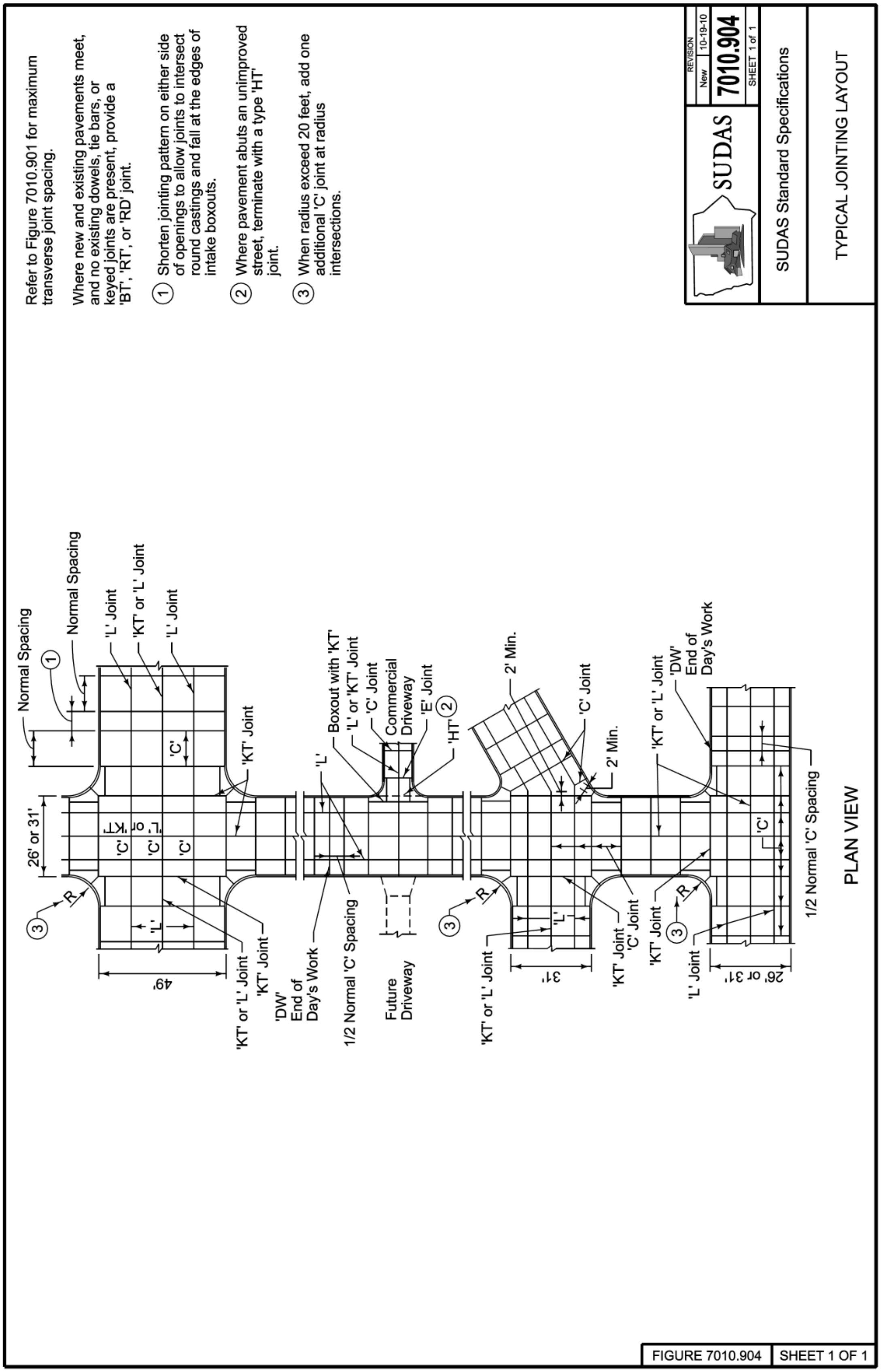


Figure 5G-3.02: Final Jointing Layout



Note: All longitudinal joints will be either KT-2 or L-2 unless otherwise indicated.

Figure 5G-3.03: Typical Municipal Jointing at Multiple Intersection Locations
(SUDAS Specifications Figure 7010.904)



A. Jointing Urban Transition Areas

This section provides examples of how to joint transition areas, such as approaches, to intersections. Many times, approaches to intersections are wider than the street and thus require a transition section.

The importance of considering constructability when developing jointing layouts for transition areas cannot be overstated. As previously noted, lane delineation with jointing should not be the predominate factor in joint layouts, particularly in urban areas. Critical lane delineation can be handled with other methods, such as pavement markings and a raised island.

Therefore, adequate jointing should be governed by the function of the joint, proper load transfer, and constructability.

Two basic widening types (with and without medians) are shown in the following figures. There are:

1. **Two-lane to Three-lane:** (i.e. 31 foot to 41 foot)
    - Quarter-point jointing
        - Concentric widening (Figure 5G-3.04)
        - One side widening (Figure 5G-3.05)
    - Third-point jointing
        - Concentric widening (Figure 5G-3.06)
        - One side widening (Figure 5G-3.07)
    - Gutterline jointing
        - Concentric widening (Figure 5G-3.08)
        - One side widening (Figure 5G-3.09)

2. **Four-lane to Five-lane:**
    - Concentric widening (Figure 5G-3.10)
    - Widening one side (Figure 5G-3.11)

Figure 5G-3.04: Quarter-Point Jointing - Concentric Widening (31 Foot to 41 Foot)

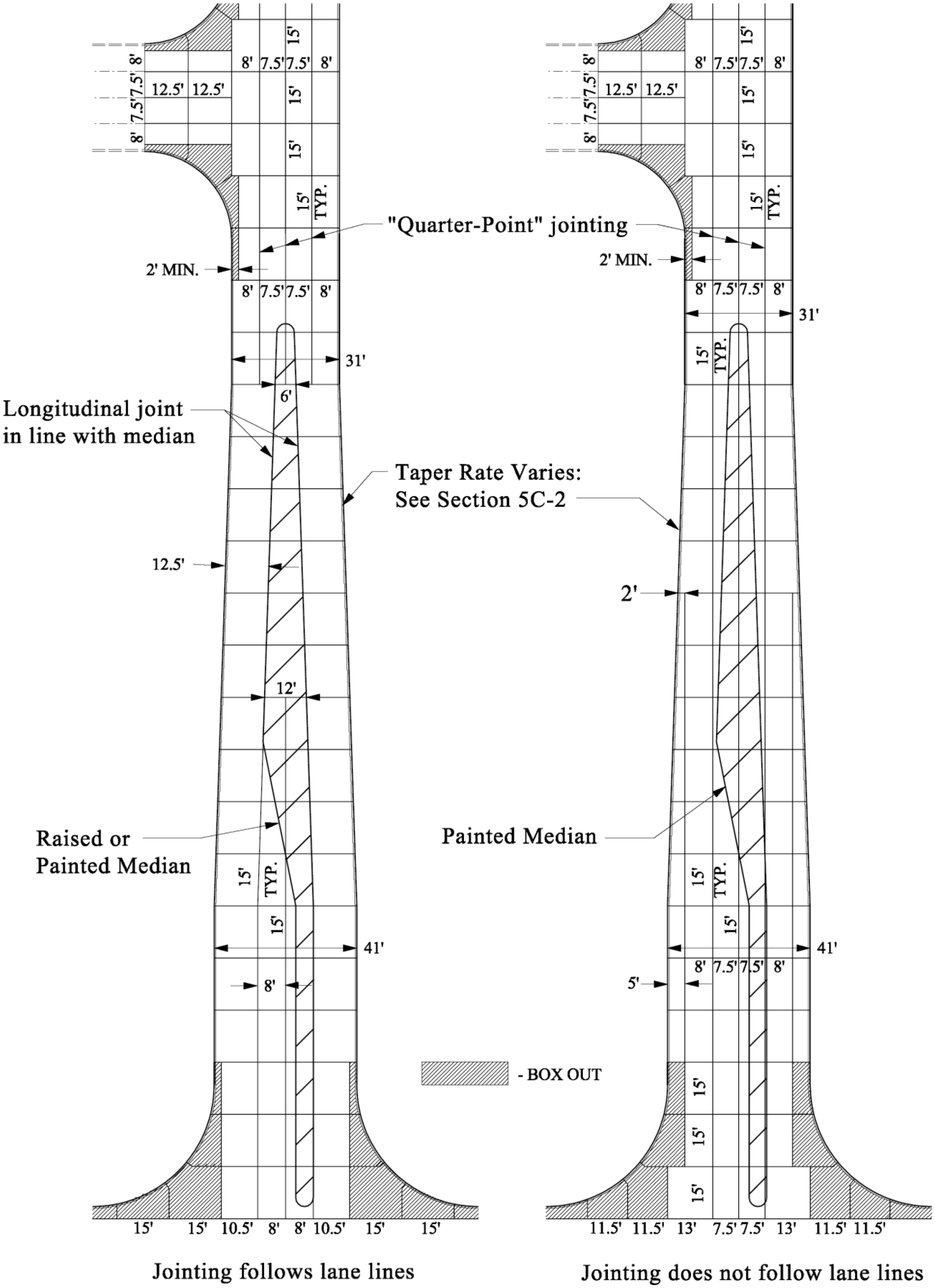


Figure 5G-3.05: Quarter-Point Jointing - Widening One Side (31 Foot to 41 Foot)

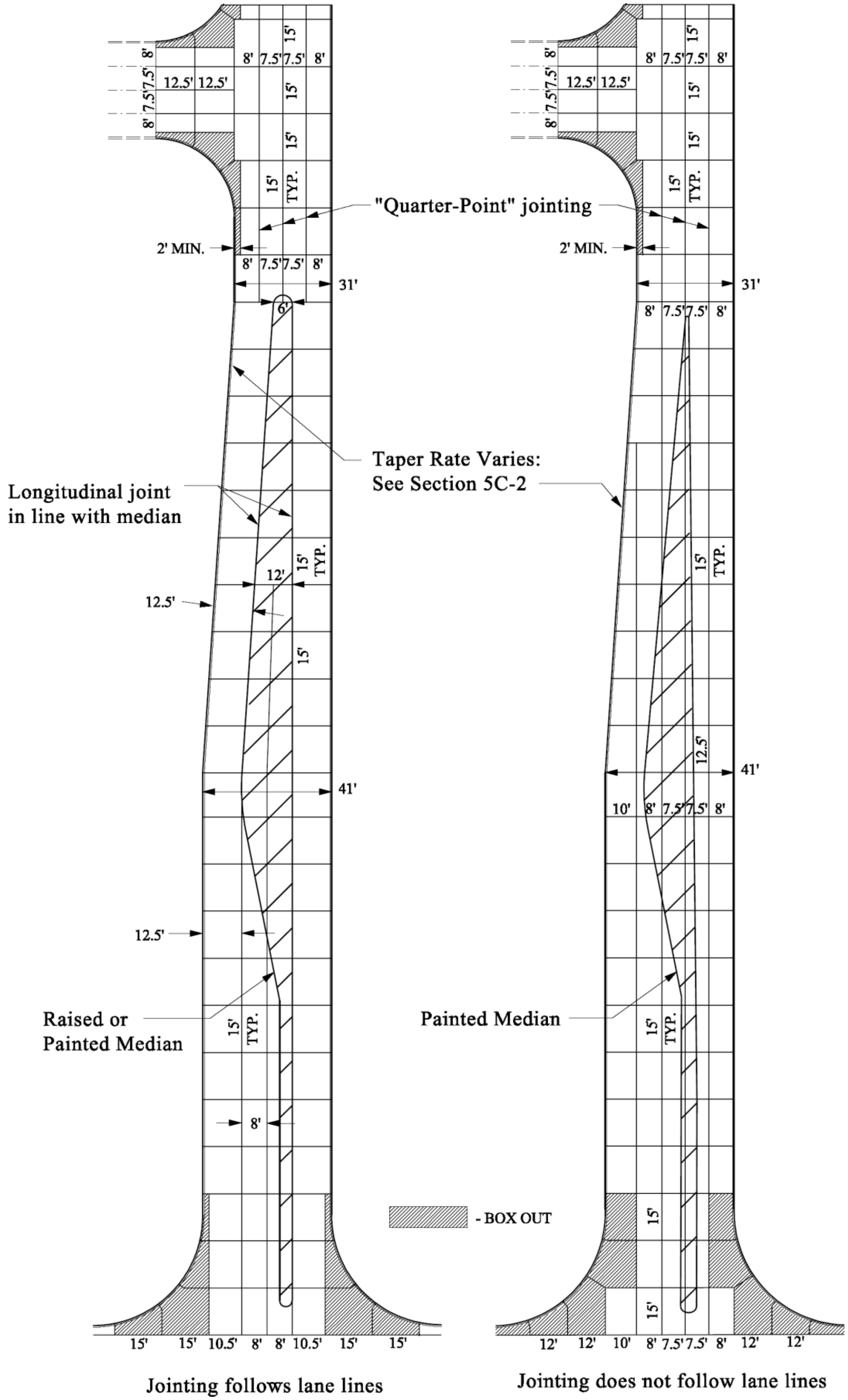


Figure 5G-3.06: Third-Point Jointing - Concentric Widening (31 Foot to 41 Foot)

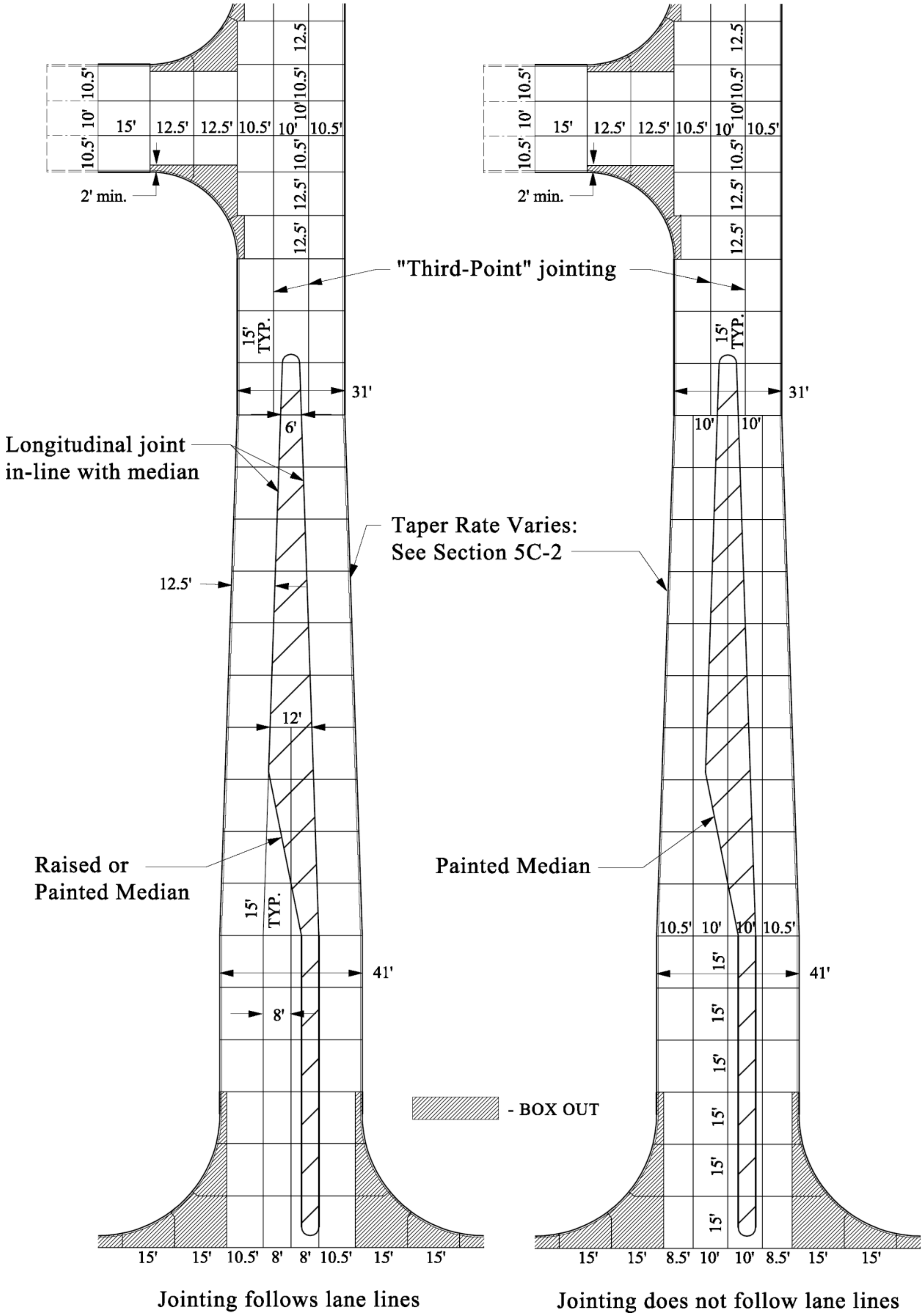


Figure 5G-3.07: Third-Point Jointing - Widening One Side (31 Foot to 41 Foot)

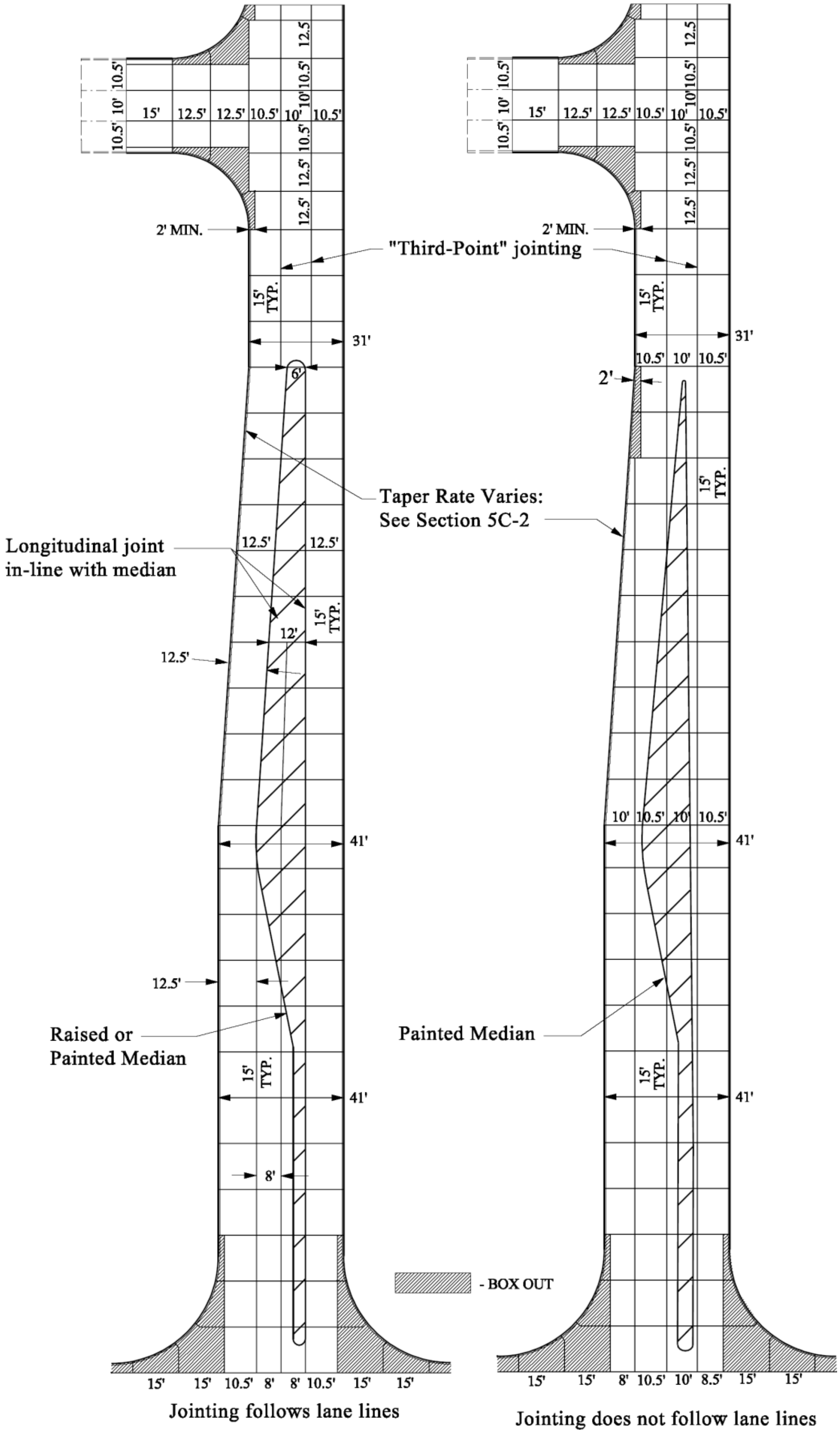


Figure 5G-3.08: Gutterline Jointing - Concentric Widening (31 Foot to 41 Foot)

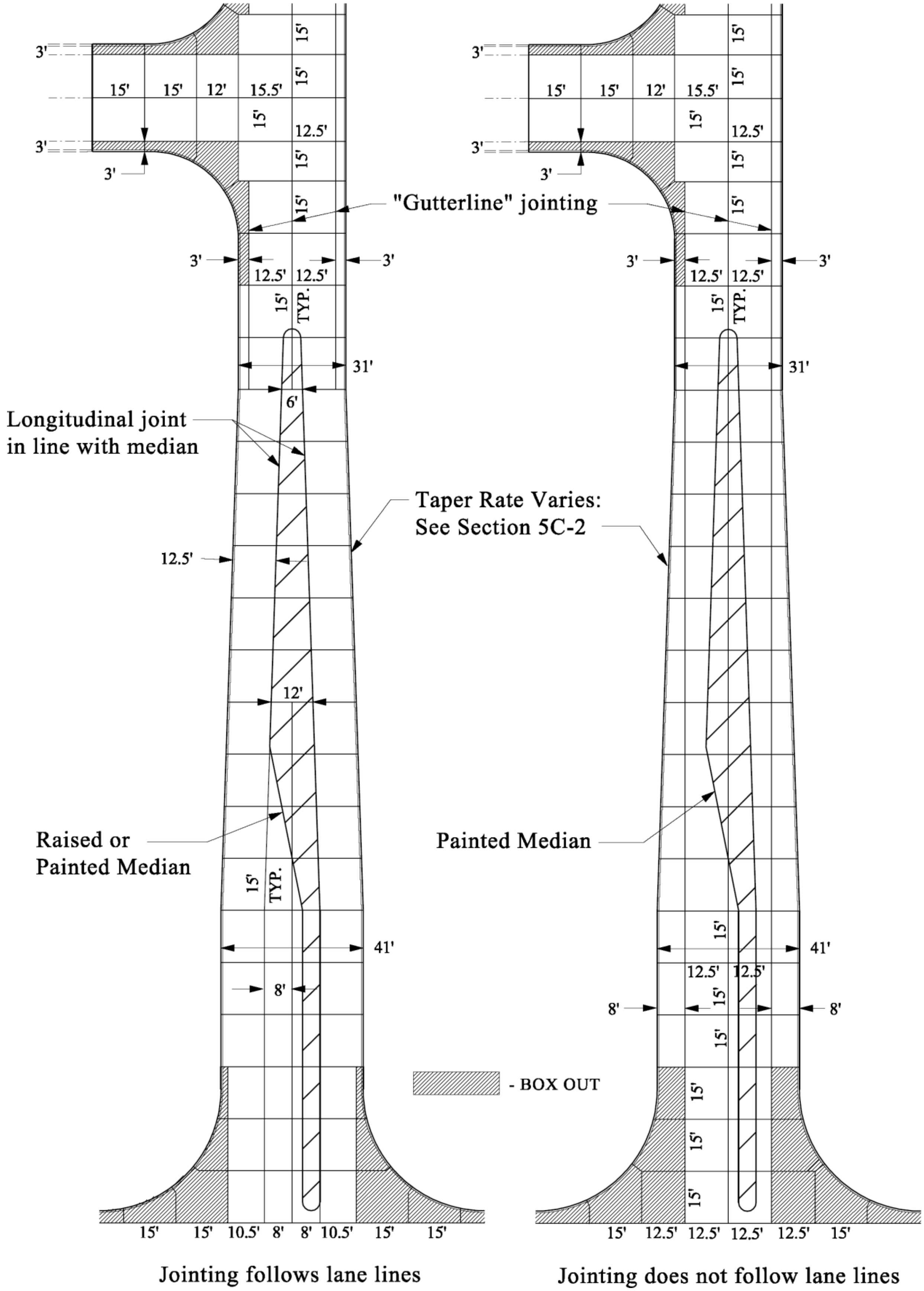


Figure 5G-3.09: Gutterline Jointing - Widening One Side (31 Foot to 41 Foot)

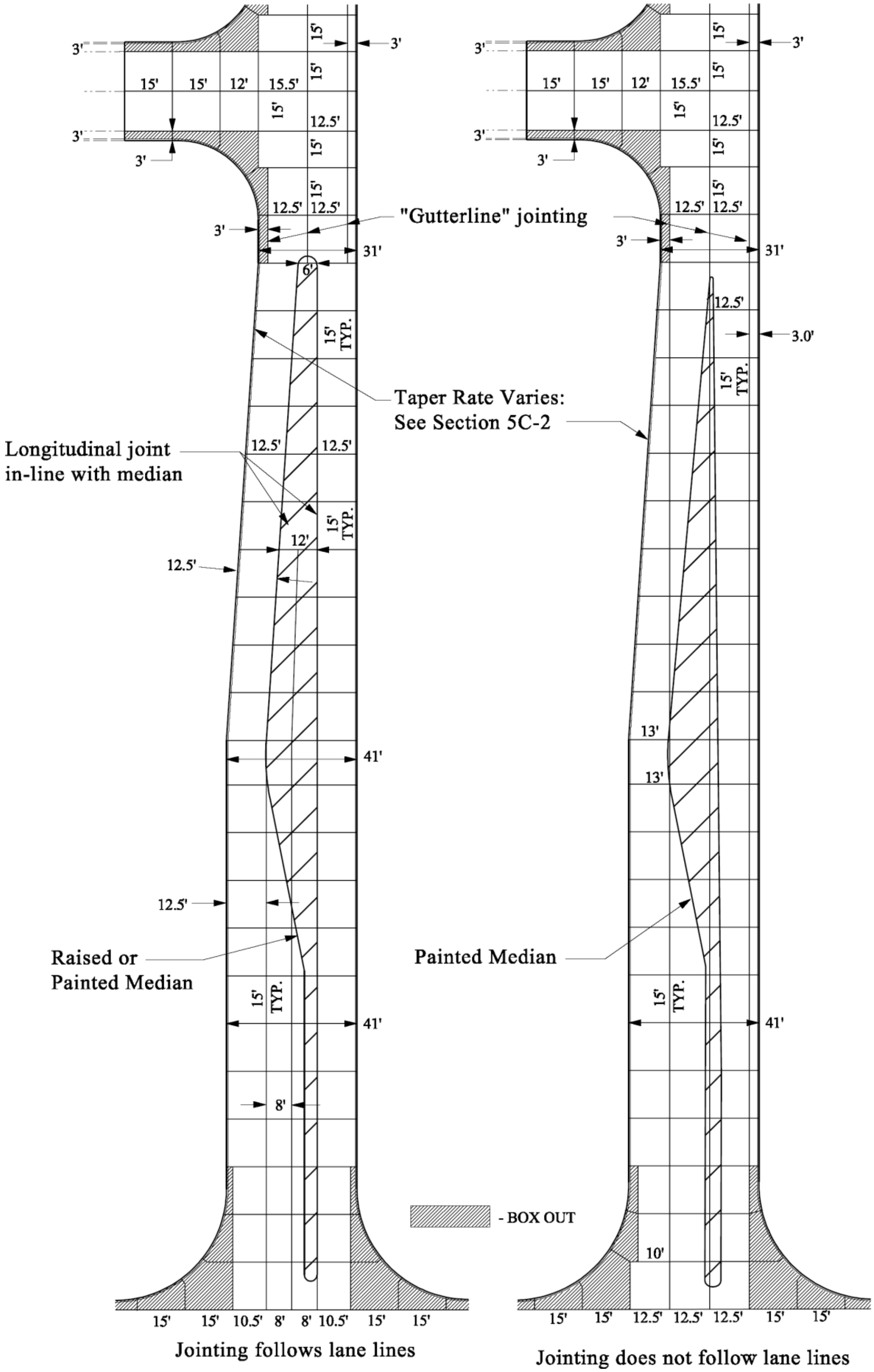


Figure 5G-3.10: Concentric Widening - Four Lane to Five Lane

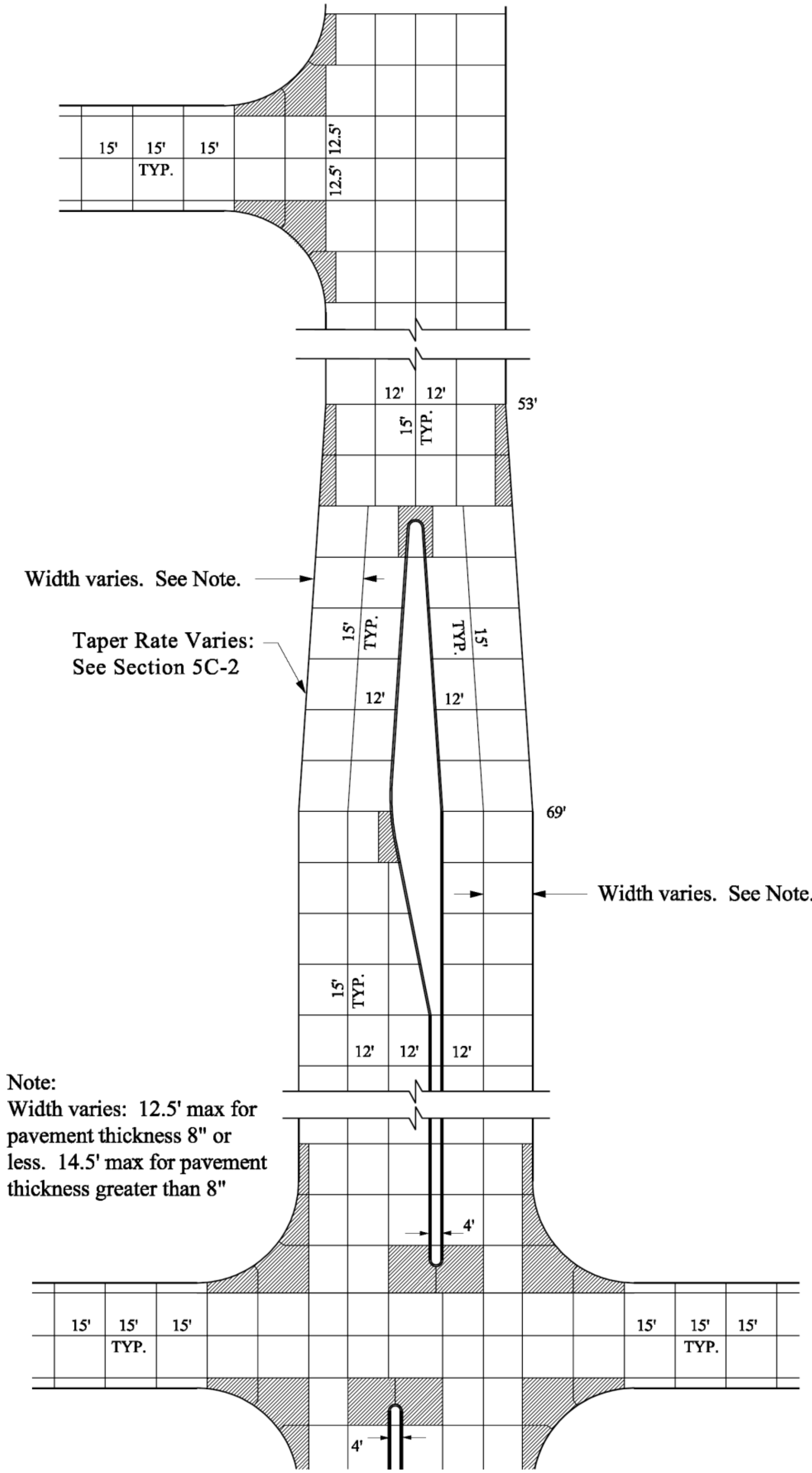
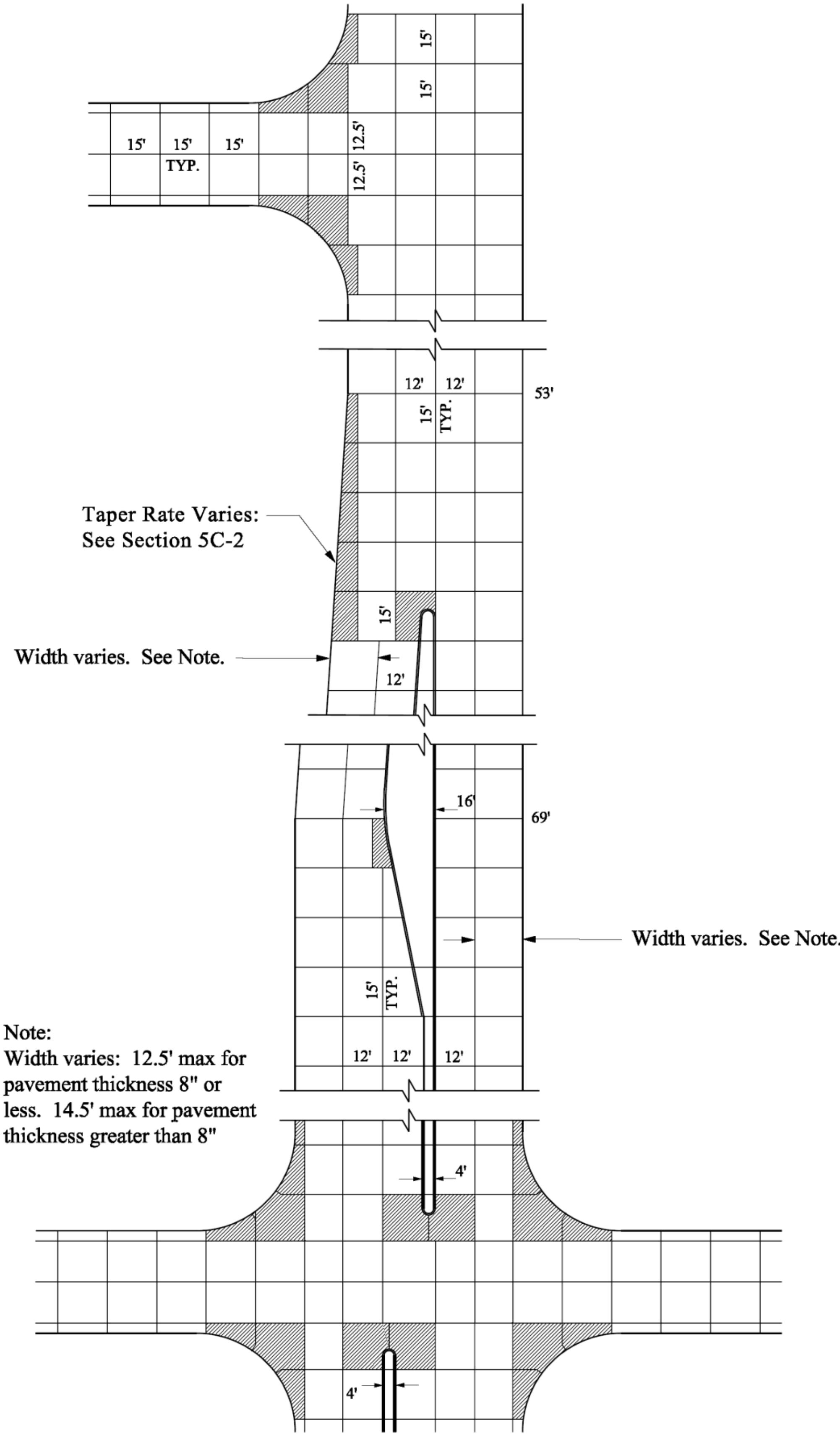


Figure 5G-3.11: Widening One Side - Four Lane to Five Lane



B. Jointing Cul-de-sacs

This section describes how to joint a cul-de-sac. The process is illustrated through an example of a street that is terminated with a cul-de-sac. Assume the pavement thickness is 7 inches.

Step 1: Locate Longitudinal Joints

The longitudinal joints running down the street should be extended into the cul-de-sac. The remaining longitudinal joints in the cul-de-sac should be placed roughly a lane width apart - somewhere in the range of 8 to 12 feet is acceptable.

A BT-1 or L-1 is an appropriate longitudinal joint, since the pavement thickness is less than 8 inches.

Step 2: Locate Transverse Joints

The next step is to place the transverse joints. The maximum spacing for transverse joints is 15 feet and the minimum spacing is 12 feet. Therefore, the joints within the cul-de-sac should be spaced within this range (see Figure 5G-3.12).

A C joint is the appropriate joint to use since the pavement thickness is less than 8 inches.

Step 3: Extend Joints Through the Free Edge of the Pavement

When extending the previously placed joints through the free edge of the pavement, the acute angle between the joint and the pavement edge (and between the joint and other joints) must be greater than or equal to 70 degrees. Also, all joints should be at least two feet long. Details A, B, and C in Figure 5G-3.13 illustrate how this can be accomplished.

- Detail A shows a transverse joint that is extended through the free edge of the pavement unaltered. These are acceptable because all angles between the transverse joint and the longitudinal joints and between the transverse joint and the free edge of the pavement are greater than 70 degrees.
- Detail B uses a dashed line to show the original position of a transverse joint whose angle, with the free edge of the pavement, is less than 70 degrees. This joint should be skewed to make it perpendicular to the free edge of the pavement, as shown by the solid line.
- Detail C illustrates a situation where skewing the joint to make it perpendicular to the free edge of the pavement would cause the angle between the joint and a longitudinal joint to be less than 70 degrees (shown by the dashed line). When this situation occurs, the joint is extended a minimum of two feet beyond the longitudinal joint, and then it is skewed to make it perpendicular to the free edge of the pavement. Both segments of the joint should be at least two feet long.

Step 4: Label Joints

The completed jointing layout for the cul-de-sac is shown in the figures that follow. The L-1 and BT-1 joints may be used interchangeably, at the contractor's discretion, depending on the paving sequence. Therefore, the designer may identify the longitudinal joints as either L-1 or BT-1 on the jointing layout.

Because the majority of the joints are either the C or the BT-1 or L-1, it is not necessary to identify every joint on the jointing layout. A note on the plan describing the transverse joints as C and longitudinal joints as L-1 or BT-1 except as noted otherwise is sufficient, provided that a few key joints on the diagram are identified. Whenever a series of joints changes to a different type of joint, the joint at the location of change is identified. Any joint that may be a source of confusion should also be labeled.

Joint lengths are also shown on the jointing layout, normally rounded to the nearest foot. Similar to labeling joint types, not every length needs to be indicated. However, any length that cannot be inferred from the diagram should be labeled.

Figure 5G-3.12: Placement of Longitudinal and Transverse Joints

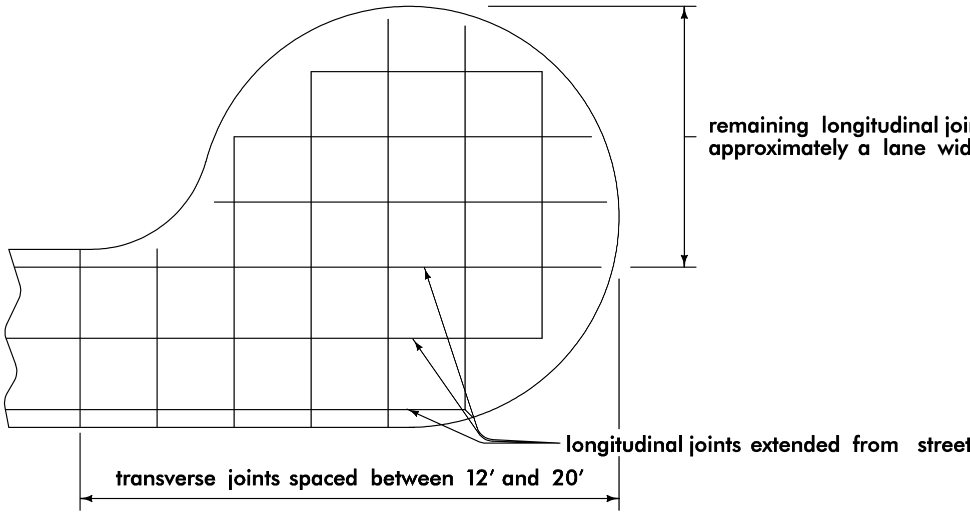


Figure 5G-3.13: Final Jointing Layout - Gutterline Jointing Examples

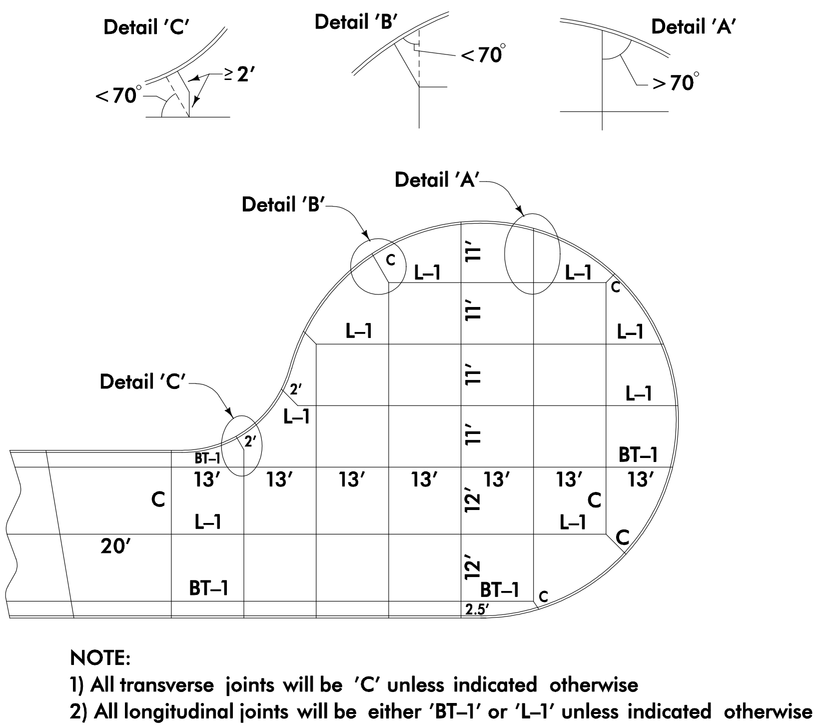


Figure 5G-3.14: Cul-de-sac Joint Locations - Quarter-point Jointing Examples
(SUDAS Specifications Figure 7010.905, sheet 1)

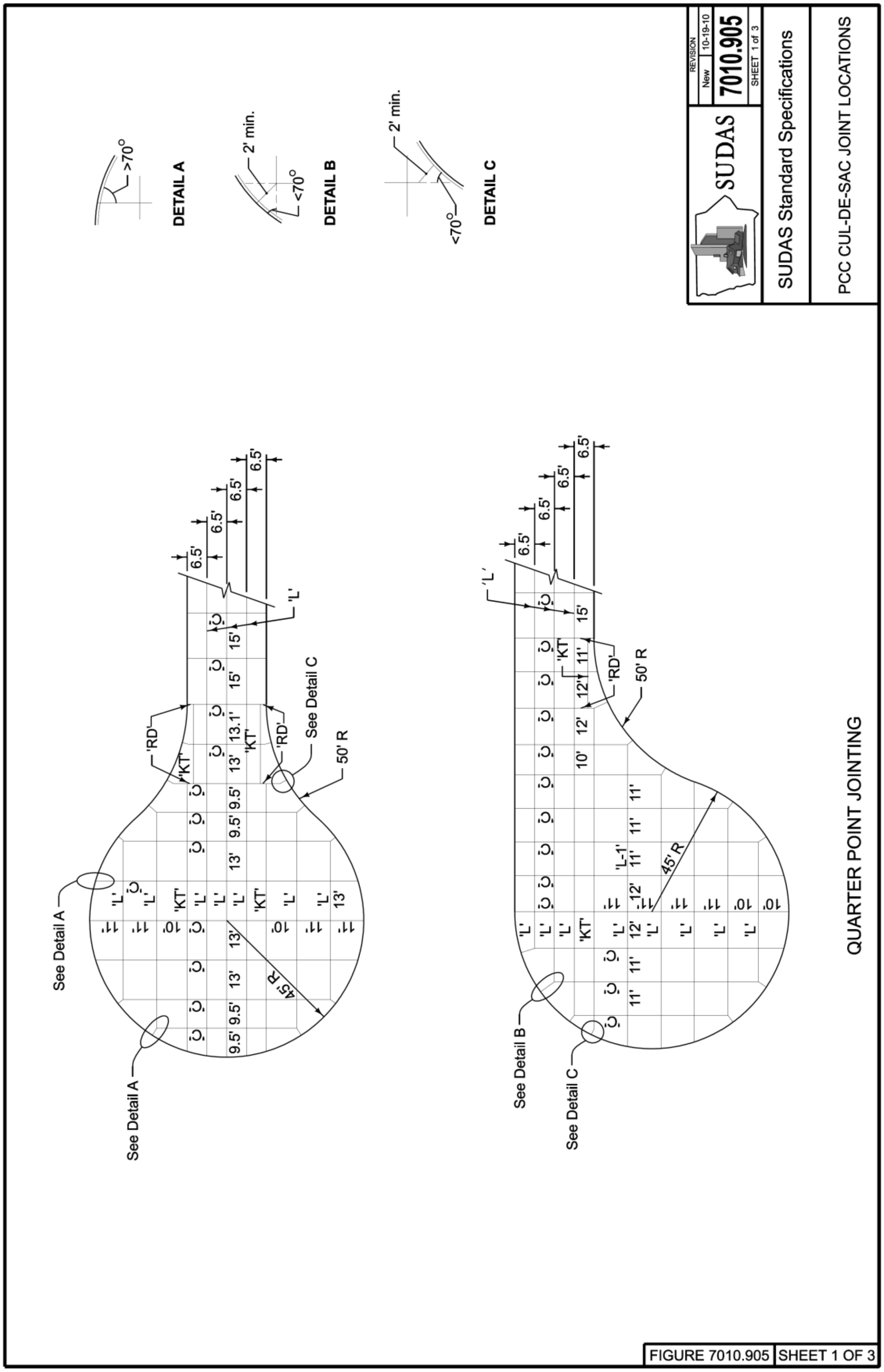


Figure 5G-3.15: Cul-de-sac Joint Locations - Third-point Jointing Examples
(SUDAS Specifications Figure 7010.905, sheet 2)

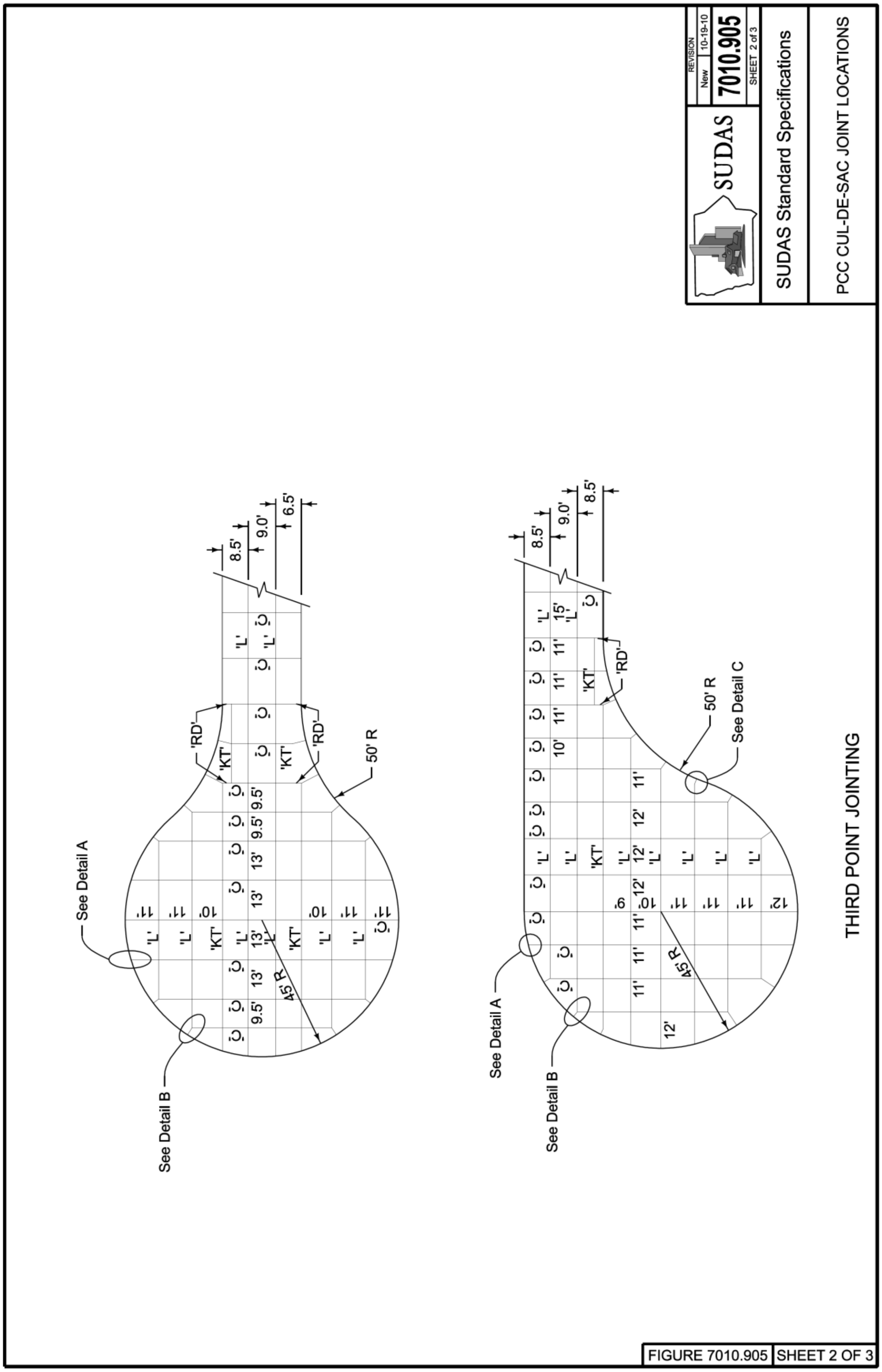
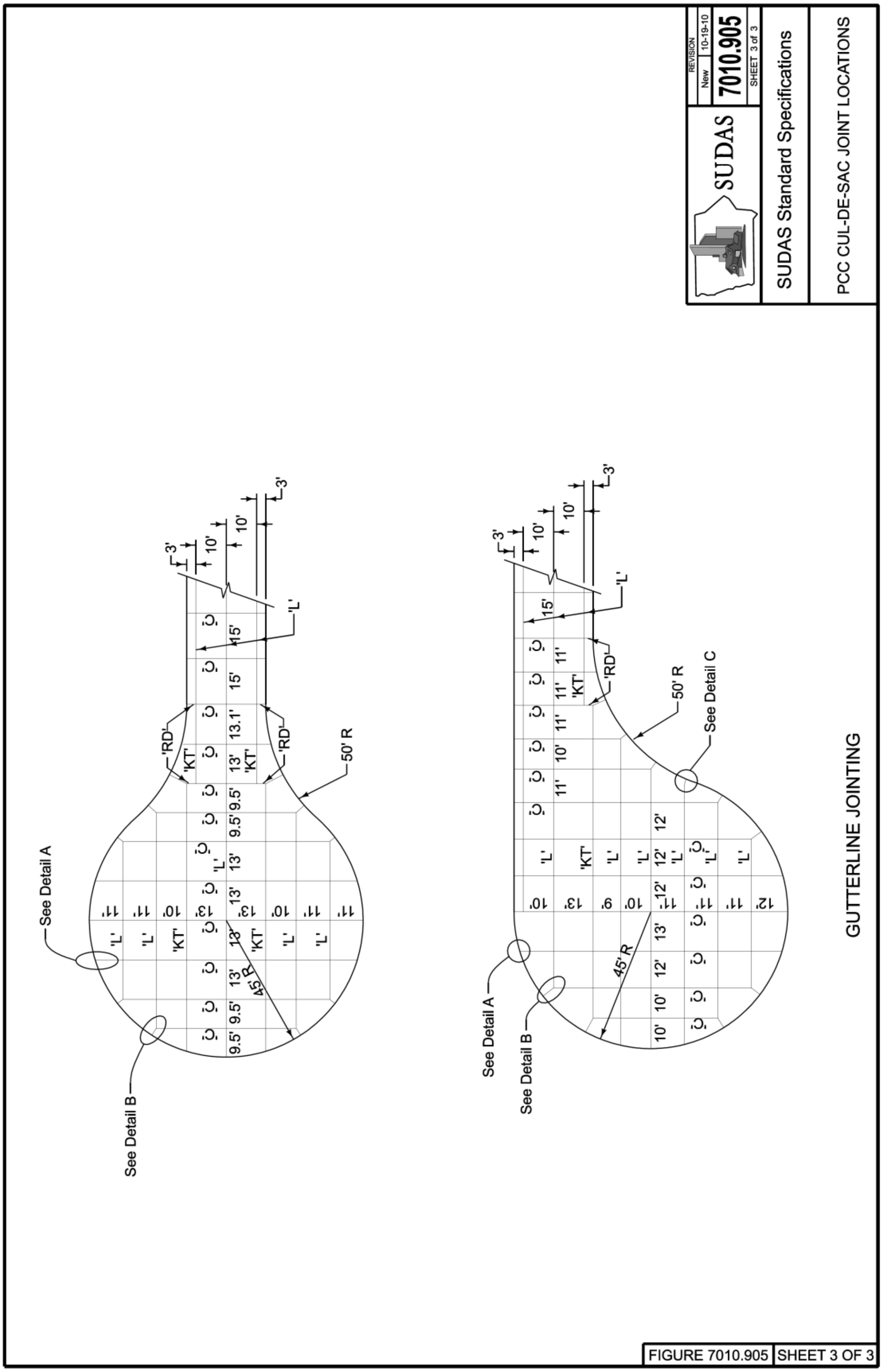


Figure 5G-3.16: Cul-de-sac Joint Locations - Gutterline Jointing Examples
(SUDAS Specifications Figure 7010.905, sheet 3)



Jointing Rural Intersections

This section describes how to joint rural intersections by following the guidelines outlined in Iowa DOT Design Manual Section 7A-3. The first example illustrates a step-by-step process for jointing a T-intersection. The second example discusses the jointing of an intersection at a divided highway. Even though not all rural intersections will be exactly like the ones in these examples, the process described is applicable to other layouts.

A. Example 1: T-Intersection

The first example is a T-intersection of a rural two-lane highway and a paved sideroad. The intersection has returns on each side (see Figure 5G-4.01) and the pavement thickness is 10 inches. The design year truck volume on the sideroad is 250 vpd.

Step 1: Place Joints with Predetermined Locations

1. **Longitudinal Joints:** Because the location of longitudinal joints for both the mainline and the sideroad are predetermined by the lane pavement width, these joints should be placed first. Within the intersection, the road that is paved first, or already exists, determines which joints are longitudinal and which are transverse. In this example, assume that the mainline is paved first. Since the mainline is a rural two lane highway, the longitudinal joints are spaced at the lane pavement width. The longitudinal joints running down the centerline and edges of the sideroad define the locations of the first transverse joints for the mainline (see Figure 5G-4.01).

   To determine an appropriate longitudinal joint to use, refer to SUDAS Specifications Figure 7010.101. Normally, the type of joint used depends on the pavement thickness. Since the pavement thickness is greater than 8 inches in this case, either a KT-2 or an L-2 joint is appropriate.

2. **Joints at End-of-taper:** The only other joints with predetermined locations are the transverse joints that are placed where the end-of-taper sections terminate. End-of-taper sections are 2 foot wide sections placed at the ends of an intersection return (see Figure 5G-4.01). They are used to prevent the return from narrowing to a point as it intersects with the pavement. Concrete less than 2 feet in width is weak and cracks readily.

   As Figure 5G-4.01 shows, normal practice is to place a transverse joint in the mainline or sideroad pavement where the end-of-taper section terminates. Figure 5G-2.02 in Section 5G-2 indicates a CD joint should be used on the mainline if the pavement thickness is greater than or equal to 8 inches. On the sideroad, CD joints are also used since the design year truck volume is greater than 200 vpd (C joints could be used on the sideroad if the design year truck volume was less than 200 vpd).

   Note that the transverse joints within the intersection are not skewed.

Step 2: Locating Difficult Joints

Difficult locations to joint, such as intersection returns and traffic islands, are addressed next. After joints have been placed in these locations, the rest of the joints can be worked in around them.

1. **Intersection Returns:** The two intersection returns are shaded in Figure 5G-4.01. To help vehicles negotiate the turn, a curved longitudinal joint (normally offset 12 feet from the free edge of the pavement) is placed in the intersection return to delineate the turning path. A second curved longitudinal joint (normally offset 24 feet from the free edge of the pavement) is placed if enough area is available.

2. **Traffic Islands:** Joint design at the traffic islands is not an exact process. It is done by trial-and-error until satisfactory results are achieved.

   The first thought may be to place CD transverse joints at every radius point of the island (see Figure 5G-4.01, Detail A). However, with this layout, the 20 foot maximum and 12 foot minimum spacings for a CD joint are violated.

   Detail B shows joints at the desired 20 foot interval. Although the spacing of this placement is correct, an awkward area of pavement is formed and a crack is likely to develop as shown in Detail B.

   Detail C illustrates a combination of the methods used in the first two details. No rules of spacing are violated and no awkward areas of pavement exist.

   The transverse joints attached to the island are extended across the sideroad and mainline pavements and across the intersection return adjacent to the island, as shown in Figure 5G-4.01. The joints used in one area must also be acceptable for any other areas into which they are extended. If the extended joints do not satisfy spacing or other criteria in any adjacent areas, they must be redesigned in the original area.

Step 3: Locating Remaining Joints

After the joints at difficult locations are located, the remaining joints (generally transverse joints) are placed in appropriate locations. As noted in Step 1, the appropriate transverse joint for both the mainline and the sideroad is the CD joint. The maximum spacing for CD joints is 20 feet and the minimum spacing is 12 feet. Therefore, the remaining areas that need transverse joints should have CD joints spaced within this range.

1. **Mainline and Sideroad:** The location of the remaining transverse joints on the mainline and sideroad is largely determined by the location of joints already placed in Steps 1 and 2 (see Figure 5G-4.01). The remaining joints are spaced between 12 and 20 feet between these already-placed joints. However, you must also consider how these joints will be extended into the returns (described below).

2. **Intersection Returns:** After the transverse joints have been located in the mainline and the sideroad, they are extended into the intersection returns to be used as transverse joints for those areas as well. As with other transverse joints, those in intersection returns must intersect with the free edge of the pavement. However, the acute angle between the joint and the pavement edge (and between the joint and other joints) must be greater than or equal to 70 degrees. Details A, B, C, and D in Figure 5G-4.02 illustrate how to intersect joints with the free edge of the pavement (and with other joints) under various conditions.

- Detail A shows a transverse joint that intersects with the free edge of the pavement unaltered. This is acceptable because all angles between the transverse joint and the longitudinal joints and between the transverse joint and the free edge of the pavement are greater than 70 degrees.
- Detail B uses a dashed line to show the original position of a transverse joint whose angle with the free edge of the pavement is less than 70 degrees. This joint should be skewed to make it perpendicular to the free edge of the pavement, as shown by the solid line.
- Detail C illustrates a situation where skewing the joint to make it perpendicular to the free edge of the pavement causes the angle between the joint and the edge of the mainline to be less than 70 degrees. When this situation occurs, the joint is extended a minimum of 2 feet beyond the edge of the mainline or sideroad, and then it is skewed to make it perpendicular to the free edge of the pavement.
- Detail D shows the curved longitudinal joints that were placed in the intersection return in Step 2. Each of these joints terminates at an intersection with a transverse joint. The intersection of these joints is required to be at least 2 feet from the edge of the mainline or sideroad. This requirement determines the appropriate transverse joint at which the longitudinal joint terminates. The dashed line in the detail indicates the position of the longitudinal joint if it is extended too far. Because the intersection with the transverse joint is less than 2 feet from the pavement edge, the longitudinal joint is terminated at the previous transverse joint.

After all joints are placed, the layout should be checked to ensure that all joint spacings and angles are acceptable. If they are not, the spacing of the mainline or sideroad joints may need to be changed, one or more joints may be added, or joints within the returns may be modified. Figure 5G-4.02 shows all of the transverse joints appropriately placed.

Step 4: Label Joints

The completed jointing layout of the T-intersection is shown in Figure 5G-4.03. As stated on SUDAS Specifications Figure 7010.101, the L-2 and KT-2 joints may be used interchangeably at the contractor's discretion, depending on the paving sequence. Therefore, the designer may identify the longitudinal joints as either L-2 or KT-2 on the jointing layout. The transverse joints in the end-of-taper sections are C joints because they are only 2 feet long, which are not long enough to use a doweled transverse joint like the CD. The joints on the right side of the traffic island are also C joints.

It is not necessary to identify every joint on the jointing layout. A few key joints on the diagram should be identified and whenever a series of joints changes to a different type of joint, the joint at the location of the change should be identified. Also, any joint that may be a source of confusion should be identified.

Joint lengths are also shown on the jointing layout, normally rounded to the nearest foot. Similar to labeling joint types, not every length needs to be indicated. However, any length that cannot be inferred from the diagram should be labeled. For example, the distance the mainline or sideroad transverse joints extend into the intersection returns before being skewed perpendicular to the free edge of the pavement, should be dimensioned (see Figure 5G-4.03).

B. Example 2: Intersection at a Divided Highway

The jointing design process for a four-way intersection at a divided highway is basically the same as the T-intersection, except that there is also a paved median opening to deal with.

As with the T-intersection, start out by placing the longitudinal joints that are predetermined by the lane pavement width. After doing this, place longitudinal joints through the opening (see Figure 5G-4.04). The edges of the left-turn lanes define the location of two of these joints. The remaining longitudinal joints in the opening are spaced roughly a lane width apart - somewhere in the range of 10 to 16 feet is acceptable.

After this, the process is basically the same as the T-intersection:

- Place the transverse joints at the end-of-taper sections.
- Place the curved longitudinal joints in the return.
- Place the transverse joints around the islands. Figure 5G-4.04 illustrates the design through this point.
- Place the remaining transverse joints and extend them into the returns and into the median opening. Refer back to the T-intersection example for details on how the joints should intersect with the free edge of the pavement and with other joints.
- Label the joints.

Figure 5G-4.05 illustrates the final jointing layout.

Figure 5G-4.01: Placement of Predetermined and Difficult Joints

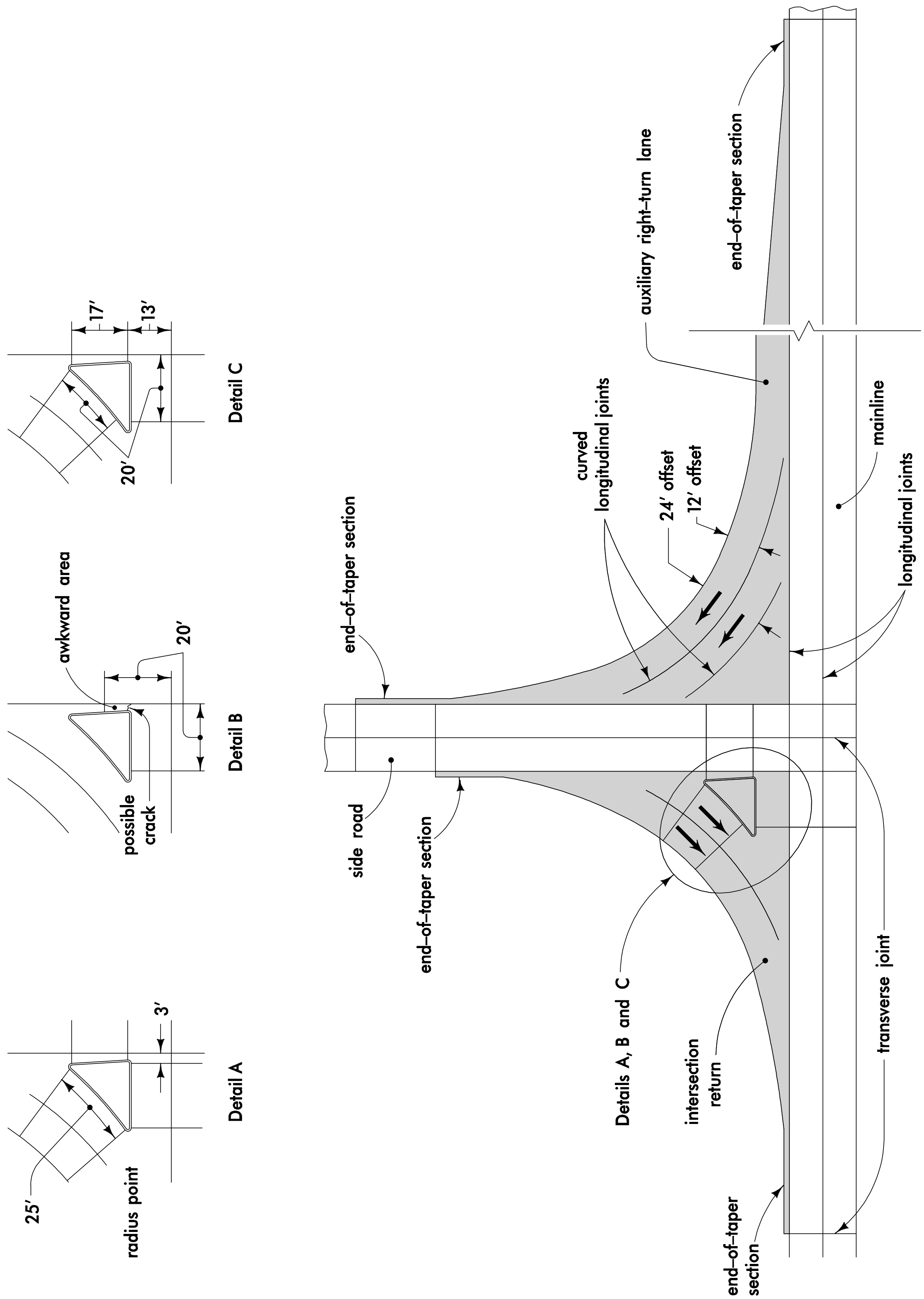


Figure 5G-4.02: Placement of Remaining Joints

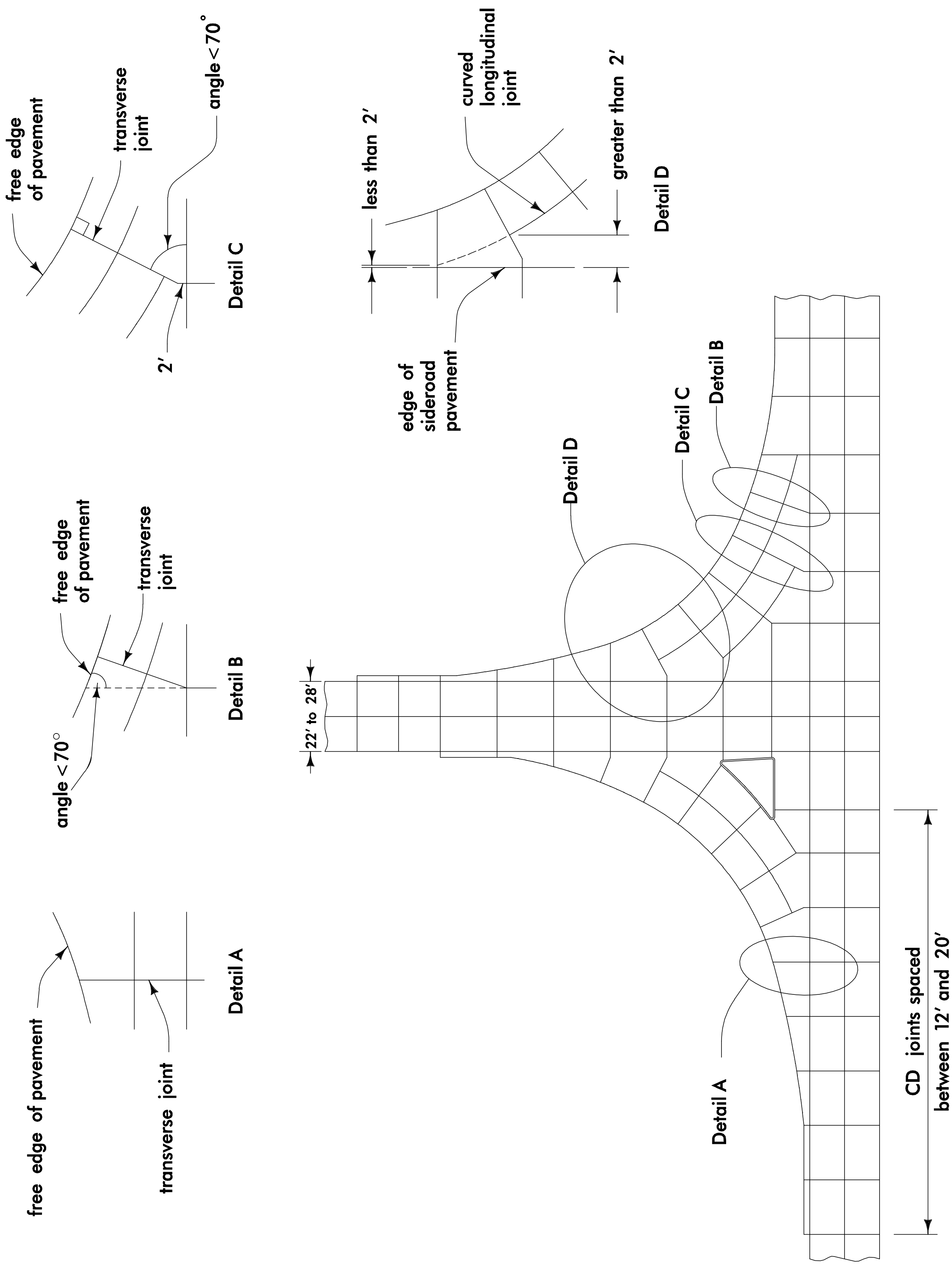


Figure 5G-4.03: Final Joint Layout

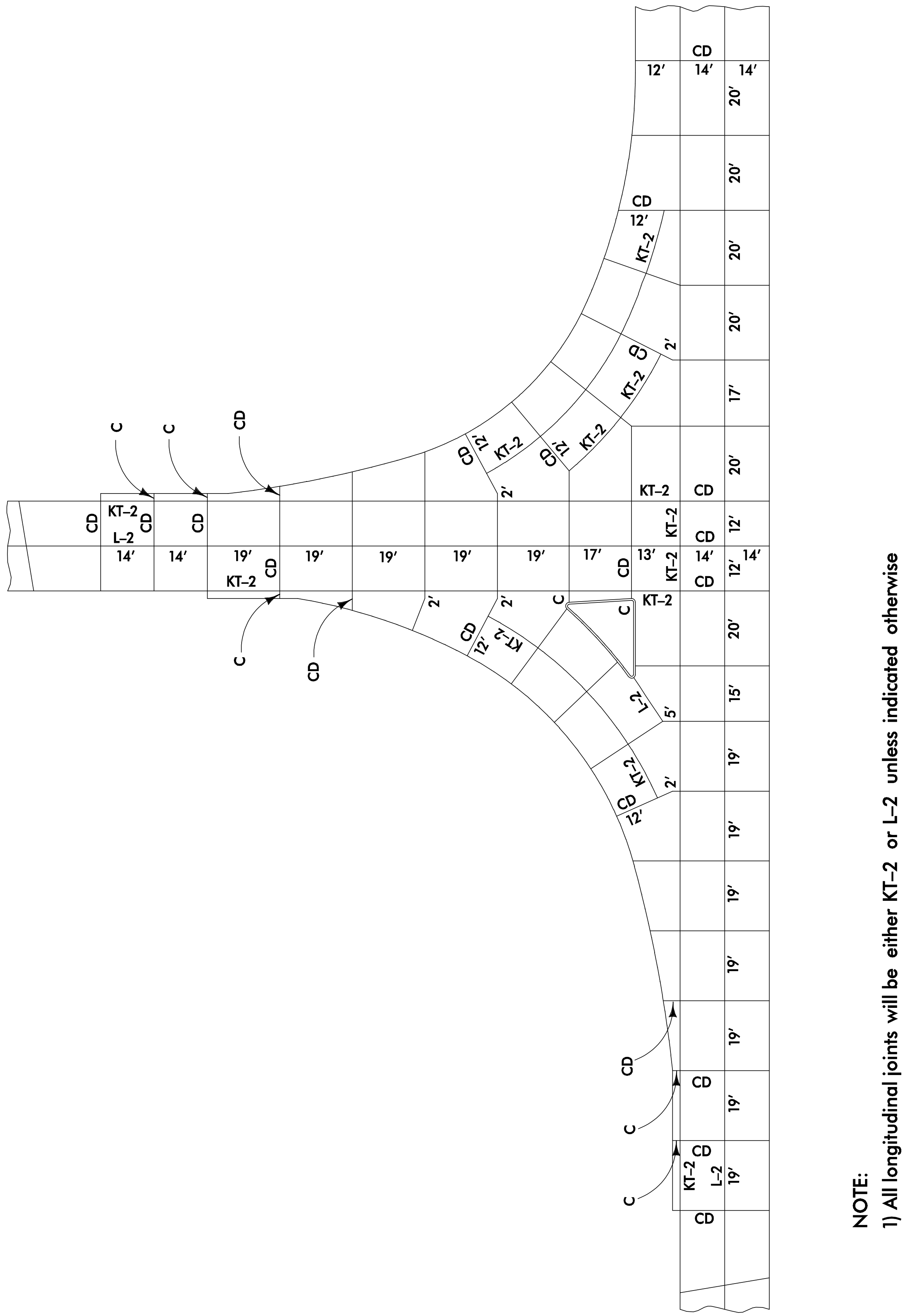


Figure 5G-4.04: Placement of Predetermined and Difficult Joints

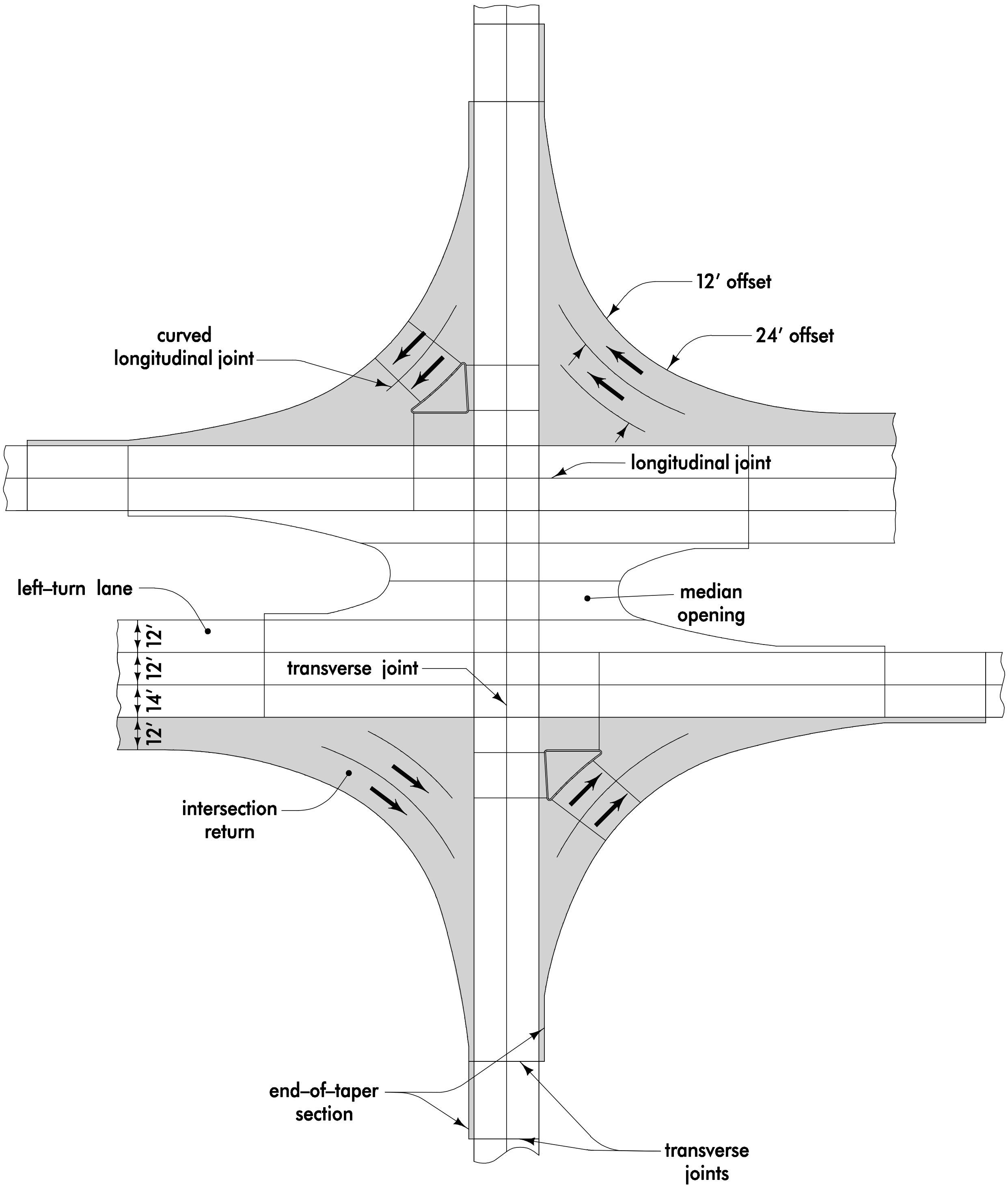
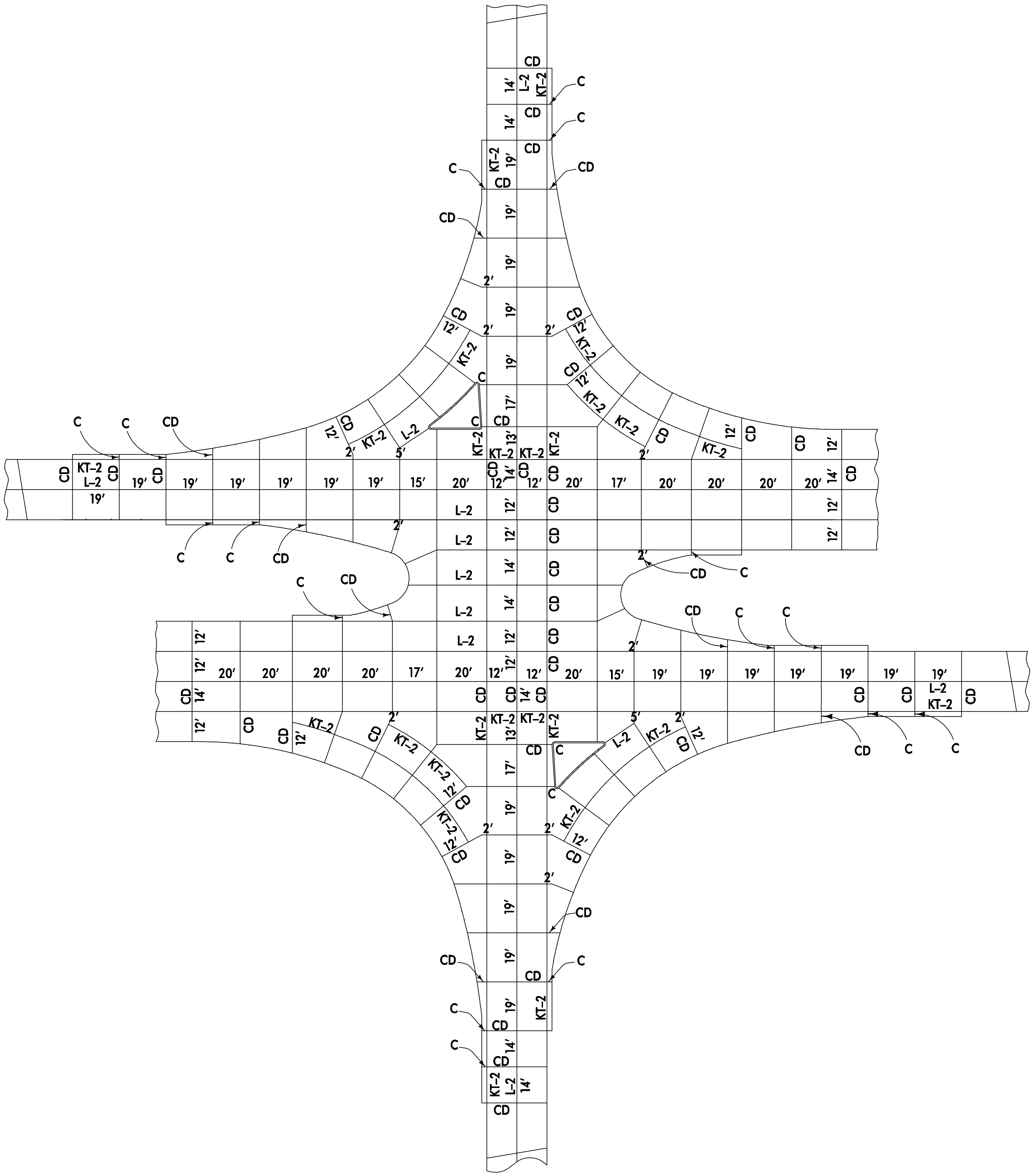


Figure 5G-4.05: Final Jointing Layout



Note:
1) All longitudinal joints will be either KT–2 or L–2 unless indicated otherwise.

Jointing Concrete Overlays

A. General Information

Bonded and unbonded concrete overlays can be placed on existing concrete pavements, asphalt pavements, and composite pavements. Although some normal joint design criteria apply to all concrete overlays, the various overlay options require special design considerations. For the purposes of this section dealing only with jointing guidance, guidelines for concrete overlays over asphalt and composite pavements are combined because of their similarity.

B. Bonded Concrete Overlays

1. Bonded Concrete Overlays of Concrete Pavements:

a. Joint Design: The bonded overlay joint type, location, and width must match those of the existing concrete pavement in order to create a monolithic structure. Matched joints eliminate reflective cracking and ensure that the two layers of the pavement structure move together, helping maintain bonding. To minimize curling and warping stresses, some agencies have successfully created smaller overlay panels by sawing additional transverse and longitudinal joints in the overlay between the matched joints. An important element in transverse joint design is joint dimensions. The depth of transverse joints should be full depth plus 0.50 inch. To prevent debonding, the width of the transverse joints should be equal to or greater than the width of the underlying joint or crack in the existing pavement. If the pavement system experiences expansion and the overlay pushes against itself because the width of the transverse overlay joint is less than the width of the underlying existing pavement crack, debonding may occur. The width of the existing underlying pavement crack may be determined by spot excavating along the pavement edge. Longitudinal joints should be sawed at least one-half the thickness of the overlay. Tie bars, dowel bars, or other embedded steel products are not used in bonded concrete overlays to minimize restraint forces in the bond.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Sawing should begin as soon as the concrete is strong enough that joints can be cut without significant raveling or chipping. Lightweight early-entry saws allow the sawing crew to get on the pavement as soon as possible. To help match transverse joint locations, place guide nails on each edge of the existing pavement at the joints; after the overlay is placed, mark the joint with a chalk line connecting the guide nails.

2. Bonded Concrete Overlays of Asphalt and Composite Pavements:

a. Joint Design: The recommended joint pattern for bonded overlays of asphalt is small square panels, typically in the range of 3 to 8 feet, to reduce curling and warping stresses. It is recommended the length and width of joint squares, in feet, be limited to 1.5 times the overlay thickness in inches. In addition, if possible, longitudinal joints should be arranged so that they are not in the wheel path. The use of tie bars or dowels is not necessary because of the small panel spacings.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Joint sawing should commence as soon as the concrete has developed sufficient strength so that joints can be cut without significant raveling or chipping, typically within 3 to 6 hours of concrete placement. Lightweight early-entry saws with 1/8 inch wide blades may be used to allow the sawing as soon as possible. Transverse joints can be sawed with conventional saws to a depth of T/4. Transverse joint sawcut depths for early-entry sawing should not be less than 1.25 inch. Longitudinal joints should be sawed to a depth of T/3. Joint sealing is not required.

C. Unbonded Concrete Overlays

1. Unbonded Concrete Overlays of Concrete Pavements:

a. Joint Design: Load transfer is better in unbonded overlays of concrete pavements than in new JPCPs because of the load transfer provided by the underlying pavement. Doweled joints are used for unbonded overlays of pavements that will experience significant truck traffic, typically pavements 8 inches and thicker. Joints are typically mismatched to maximize load transfer from the underlying pavement. Shorter joint spacing should be used to reduce the risk of early cracking due to enhanced curling caused by the stiff support provided by the underlying pavement (see Table 5G-5.01). Using lane tie bars may be appropriate in open-ditch (or shoulder) sections of unbonded overlays if the overlay is 5 inches or greater. In this category, a #4 tie bar (0.50 inch) may be appropriate. The use of tie bars in confined curb and gutter sections should be considered if the overlay is 6 inches or greater.

Table 5G-5.01: Typical Transverse Joint Spacing

Unbonded Resurfacing Thickness	Maximum Transverse Joint Spacing
< 5 inches	6 x 6 foot panels
5 to 7 inches	Spacing in feet = 2 times thickness in inches
> 7 inches	15 feet

Source: Harrington, 2008.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Transverse joints can be sawed with conventional saws to a depth of between T/4 (minimum) and T/3 (maximum), but not less than 1.25 inch. Transverse joint sawcut depths for early entry sawing should not be less than 1.25 inch. Saw longitudinal joints to a depth of T/3.

2. Unbonded Concrete Overlays of Asphalt and Composite Pavements:

a. Joint Design: The load transfer design is the same as for new concrete pavements. Doweled joints are used for unbonded overlays of pavements that will experience significant truck traffic, typically pavements 8 inches and thicker. For pavements less than 6 inches thick, the maximum spacing in feet is 1.5 times the slab thickness in inches. For pavements 6 inches thick or greater, a maximum joint spacing in feet of two times the slab thickness in inches is often recommended for unbonded overlays. A 6 inch overlay would thus receive a maximum 12 foot joint spacing. The maximum recommended spacing is typically 15 feet. The use of tie bars for unbonded overlays should follow conventional use for pavements 5 inches thick or more. Using lane tie bars may be appropriate in open-ditch (or shoulder) sections of unbonded overlays if the overlay is 5 inches or greater. In this category, a # 4 tie bar (0.50

inch) may be appropriate. The use of tie bars in confined curb and gutter sections should be considered if the overlay is 6 inches or greater.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Transverse joints can be sawed with conventional saws to a depth of between T/4 (minimum) and T/3 (maximum). When there is evidence of some wheel rutting on the existing asphalt pavement, sawcut depth is of particular concern for unbonded overlays because the distortions in the underlying asphalt pavement can effectively increase the slab thickness. Transverse joint sawcut depths for early-entry sawing should not be less than 1.25 inch. Longitudinal joints should be sawed to a depth of T/3. Always match overlay joints to the joints in any concrete patches in the existing pavement and cut the joints full depth.

Table 5G-5.02: General Jointing Practices for PCC Overlays

Construction Consideration of Joints	Bonded Overlays of Concrete	Bonded Overlays of Asphalt or Composite	Unbonded Overlays of Concrete	Unbonded Overlays of Asphalt or Composite
Typical Thickness:	***3 to 4 inch***	***3 to 4 inch***	***5 to 12 inch***	***5 to 12 inch***
Joint spacing for concrete overlays requires special consideration for each type:				
• Joints are to be matched with underlying concrete to prevent cracking.	X			
• Joints are typically mismatched to maximize load transfer from the underlying pavement.			X	
• Recommended length and width of panels in feet should be limited to 1.5 times the overlay thickness in inches.	X	X		
• Because of the potential for higher curling and warping stress from a rigid underlying pavement, shorter than normal spacing is typical.			X	X
Joint sawing:				
• The timing of sawing is critical. Sawing joints too early can cause excess raveling.	X	X	X	X
• Sawing must be completed before curl stresses exceed the bond strength developed.	X	X		
• Sawing too late can cause excess stresses, leading to uncontrolled random cracking.	X	X	X	X
• Transverse joint saw-cut depth for conventional saws.	Full Depth + 0.50 inch	T/4	T/4 min. - T/3 max.	T/4 min. - T/3 max.
• Transverse joint saw-cut for early-entry saws.	Full Depth + 0.50 inch	Not < 1.25 inch	Not < 1.25 inch	Not < 1.25 inch
• Longitudinal joint saw-cut depth.	T/2 (at least)	T/4 – T/3	T/4 – T/3	T/4 – T/3
• Transverse joint width must be equal to or greater than the underlying crack width at the bottom of the existing transverse joint.	X			
• Joint type, location, and width must match those of the existing pavement to create a monolithic structure.	X			
• Recommended joint pattern is square panels		X		
Sealing:				
• Seal joint using low-modulus hot-pour sealant with narrow joint.	X	**	*	*
Other considerations:				
• Doweled joints are used for pavements that experience heavy truck traffic, 8 inch pavements and thicker.			X	X
• Lane tie bars may be appropriate in open ditch (or shoulder) sections of unbonded overlays if the overlay is 5 inches or greater.			X	X
• Tie bar use in curb and gutter sections should be considered if the overlay is 6 inches or greater.			X	X

* Some states have experienced problems with asphalt stripping of the separation layer, particularly under heavy truck traffic and high speeds. Therefore, sealing is important in these conditions. On lower speed roadways without a heavy traffic loading, some states successfully do not seal.

** Joint between overlay and non-integral curb and gutter.

Source: Harrington, 2008.

D. References

Harrington, D. *Guide to Concrete Overlays: Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements*. Second Ed. National Concrete Pavement Technology Center, Iowa State University. ACPA publication no. TB021.02P. 2008.

Railroad Crossings

A. Railroad Crossing Improvements

Improvements to railroad crossings can take several forms. These include closing of an existing crossing, improvements to the existing crossing, and separating the roadway from the railroad tracks. Potential improvements to existing crossings include installation of adequate signage, signals, and signals with gate arms.

The local jurisdiction must use judgment in the selection process for crossing improvements. Several factors weigh into the selection process including the amount and speed of traffic on the roadway and railroad, available sight distance, and safety benefits. Traffic control systems for railroad-highway grade crossings must comply with the Manual on Uniform Traffic Control Devices (MUTCD).

The Jurisdiction should contact the offices of Rail Transportation and Local Systems at the Iowa DOT for any agreements and requirements that must be followed.

B. Railroad Crossing Construction

When railroad crossings are required on streets subject to heavy loads, an approved high quality grade crossing material should be installed. Some railroads may require an asphalt separation between the header and the crossing to allow for easier railroad maintenance of the crossing. Some railroads may require that the crossing material be installed by their own forces, with the costs borne by or shared with the local jurisdiction. Example railroad crossing approaches are included in Figures 1 and 2. In all cases, the railroad should be contacted for their specific crossing requirements.

C. Working with a Railroad

Working with a railroad company requires coordination at numerous steps along the planning, design, and construction process. A list of potential coordination steps follows; however, these requirements differ for each company and should be verified early in the planning process.

Phase	Possible Coordination Required
Planning	Right of entry permit for survey Coordination regarding potential modifications/improvements
Design	Right of Entry Permit for Survey Utility Accommodation Permit Maintenance Consent Agreement Coordination regarding crossing material and safety elements
Construction	Railroad Protective Liability Insurance Right of Entry for Construction Railroad Flaggers

D. Railroad Related Agencies in Iowa

Two governmental agencies are involved in regulating railroad activities within the State of Iowa. Additional information about these organizations is available at their respective websites:

Iowa DOT, Rail Transportation	http://www.iowadot.gov/iowarail/index.htm 800 Lincoln Way Ames, Iowa 50010 515-239-1140
Federal Railroad Administration Region 6 Office	http://www.fra.dot.gov/ 901 Locust Street – Suite 464 Kansas City, MO 64106 816-328-3840

E. Railroad Companies in Iowa

Currently there are nineteen railroads operating within the State of Iowa. These include three Class I railroad companies, Amtrak, and several regional and local railroads. The Iowa DOT maintains a website with links to the websites of the freight railroads operating within the State (http://www.iowadot.gov/iowarail/railroads/industry/profiles.aspx#).

Figure 5H-1.01: PCC Railroad Crossing Approach
(SUDAS Specifications Figure 7010.903)

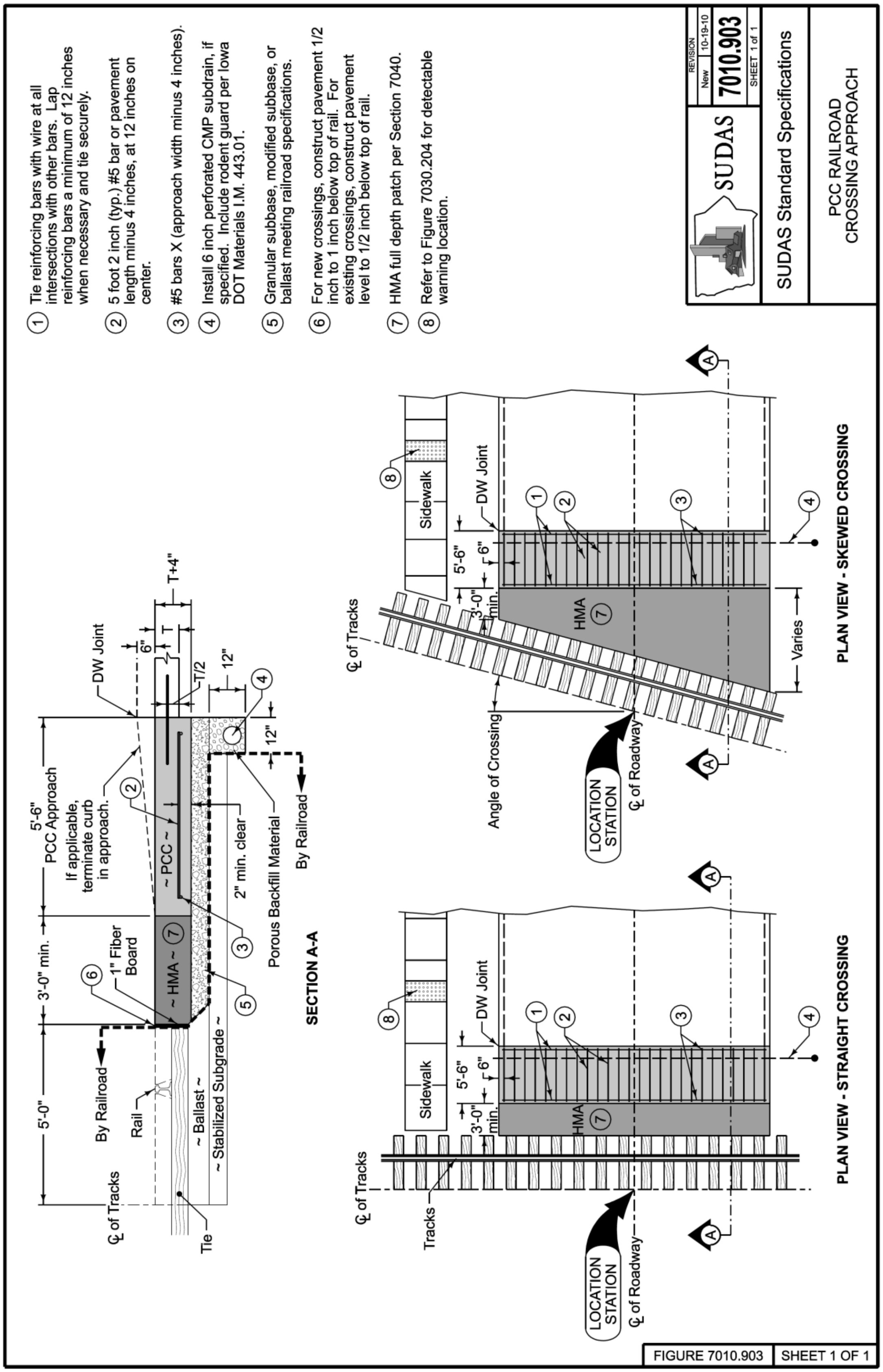
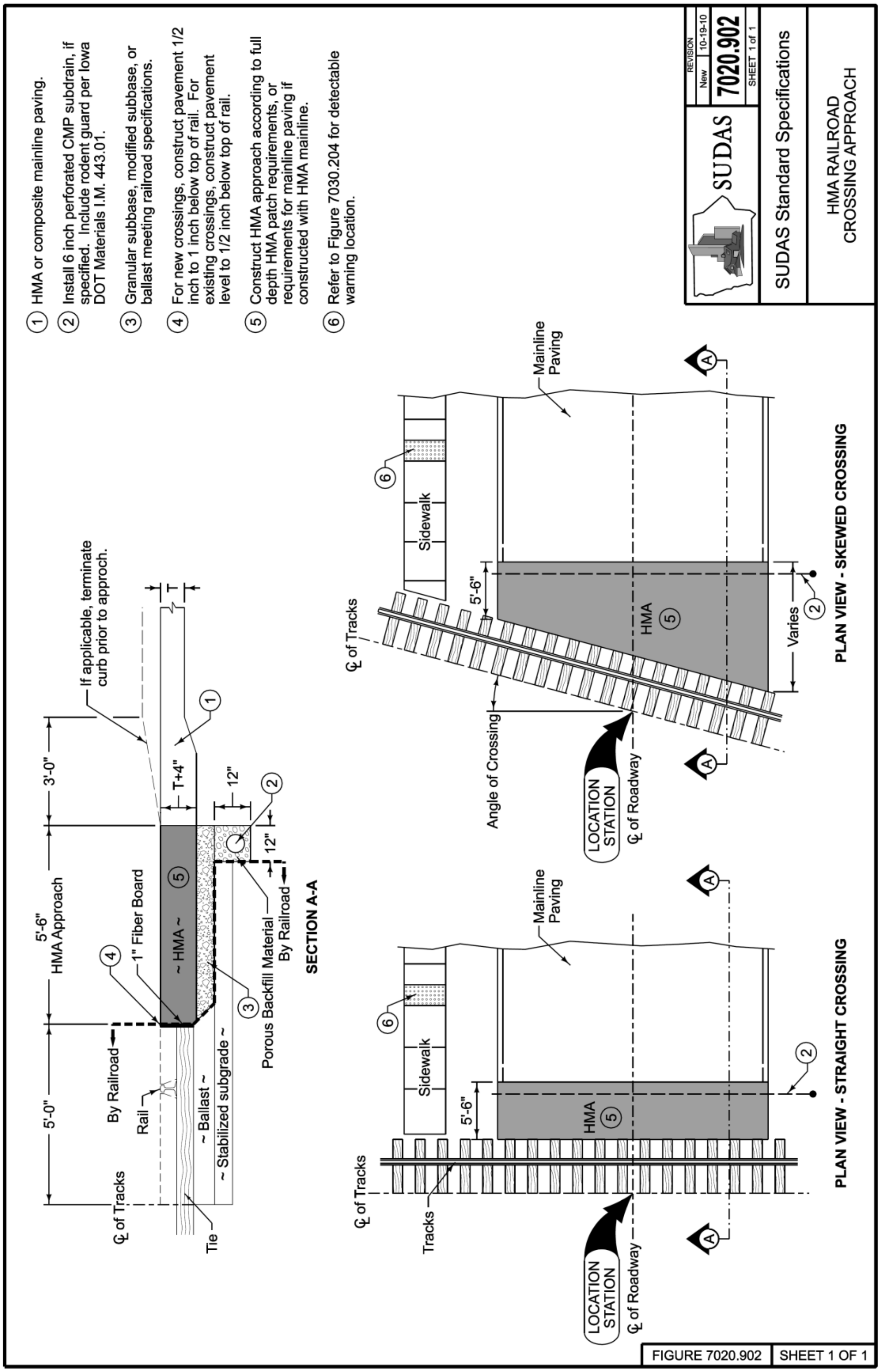


Figure 5H-1.02: HMA Railroad Crossing Approach
(SUDAS Specifications Figure 7020.902)



General Access Management

A. General Information

The efficiency and safety of a street or highway depends largely upon the amount and character of interruptions to the movement of traffic. The primary cause of these interruptions is vehicular movements to and from businesses, residences, and other developments along the street or highway. Regulation and overall control of access is necessary to provide efficient and safe highway operation and to utilize the full potential of the highway investment.

The Jurisdictions reserve the right to make exceptions to the criteria where the exercise of sound and reasonable engineering judgment indicates that the literal enforcement of the criteria would cause an undue hardship to any interested party.

B. Access Permit Procedure

An access permit may be required for any public or private access constructed to a public street. The Jurisdictional Engineer will stipulate the information required and the permit form to use. Access to streets or highways under the jurisdiction of the Iowa DOT will be governed by requirements of the Iowa DOT with Jurisdictional review (See Section 5J-1).

In addition to specific details, the following general criteria will be used by the Jurisdiction when reviewing an access request:

1. Safety to the traveling public
2. Preservation of the traffic-carrying capacity of the highway
3. The impact upon the economy of the area
4. Protection of the rights of the traveling public and of property owners, including the rights of abutting property owners

C. Definitions

Access management definitions can be found in the following resources:

1. Iowa Department of Transportation - “Iowa Primary Road Access Management Policy.”
2. Transportation Research Board - “Access Management Manual.”

D. Entrance Type

1. **Major:** An entrance developed to carry sporadic or continuous heavy concentrations of traffic. Generally, a major entrance carries in excess of 150 vehicles per hour. An entrance of this type would normally consist of multiple approach lanes and may incorporate a median. Possible examples include racetracks, large industrial plants, shopping centers, subdivisions, or amusement parks.

2. **Commercial/Industrial:** An entrance developed to serve moderate traffic volumes. Generally, a commercial/industrial entrance carries at least 20 vehicles per hour but less than 150 vehicles per hour. An entrance of this type would normally consist of one inbound and one outbound traffic lane. Possible examples include service stations, small businesses, drive-in banks, or light industrial plants.

3. **Residential:** An entrance developed to serve light traffic volumes. Generally, a residential entrance carries less than 20 vehicles per hour. An entrance of this type would not normally accommodate simultaneous inbound and outbound vehicles. Possible examples include single-family residence, farm, or field entrances.

E. Access Management Principles

A variety of access management, location, and design practices and policies can be used to improve the safety and operations of the roadway within a state's, city's, or county's jurisdiction.

Following are the 10 Principles of Access Management identified by the TRB:

1. **Provide a Specialized Roadway System:** Different types of roadways serve different functions. It is important to design and manage roadways according to the primary functions that they are expected to serve.

2. **Limit Direct Access to Major Roadways:** Roadways the serve higher volumes of regional through traffic need more access control to preserve their traffic function. Frequent and direct property access is more compatible with the function of local and collector roadways.

3. **Promote Intersection Hierarchy:** An efficient transportation network provides appropriate transitions from one classification of roadway to another.

4. **Locate Signals to Favor through Movements:** Long uniform spacing of intersections and signals on major roadways enhances the ability to coordinate signals and ensure continuous movement of traffic at the desired speed.

5. **Preserve the Functional Area of Intersections and Interchanges:** The functional area of an intersection or interchange is the area that is critical to its safe and efficient operation. This is the area where motorists are responding to the intersection or interchange, decelerating, and maneuvering into the appropriate lane to stop or complete a turn.

6. **Limit the Number of Conflict Points:** Drivers make more mistakes and are more likely to have collisions when they are presented with the complex driving situations created by numerous conflict points.

7. **Separate Conflict Areas:** Drivers need sufficient time to address one potential set of conflicts before facing another. The necessary spacing between conflict areas increases as travel speed increases, to provide drivers adequate perception and reaction time.

8. **Remove Turning Vehicles from Through-traffic Lanes:** Turning lanes allow drivers to decelerate gradually out of the through lane and wait in a protected area for an opportunity to complete a turn. This reduces the severity and duration of conflict between turning vehicles and through traffic, and improves the safety and efficiency of roadway intersections.

9. **Use Nontraversable Medians to Manage Left Turn Movements:** Medians channel turning movements on major roadways to controlled locations. Nontraversable medians and other techniques that minimize left turns or reduce driver workload can be especially effective in improving roadway safety.

10. **Provide a Supporting Street and Circulation System:** Provide a supporting network of local and collector streets to accommodate development, as well as unified property access and circulation systems.

F. References

American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*. 2004.

Institute of Transportation Engineers. *Transportation and Land Development*. 2002.

Iowa Department of Transportation. *Iowa Primary Road Access Management Policy.* 2012.

Transportation Research Board. *Access Management Manual*. 2003.

Transportation Research Board. *NCHRP Report 420: Impacts of Access Management Techniques.* 1999.

Transportation Research Board. *NCHRP Report 457: Evaluating Intersection Improvements: An Engineering Study Guide*. 2001.

Transportation Research Board. *NCHRP Report 659: Guide for the Geometric Design of Driveways.* 2010.

Transportation System Considerations

This section addresses transportation system considerations in access management, including TRB Principles of Access Management 1 through 4 and 10:

A. Provide a Specialized Roadway System (Principle 1)

The primary function of major arterial roadways is to safely and efficiently accommodate through traffic. The primary function of local streets is to provide access to adjacent properties. Minor arterials and collectors provide a blend of the mobility and access functions. Design and management of transportation facilities, including access management, must consider the classification and intended function of roadways.

B. Limit Direct Access to Major Roadways (Principle 2)

Providing direct property access to major roadways can significantly affect corridor operations and safety, and is not consistent with the function of the major roadway. Higher levels of access control become more necessary as major road through traffic volumes and speeds increase.

C. Promote Intersection Hierarchy (Principle 3)

Provide appropriate transitions from one roadway classification to the next.

- Freeways intersect arterials with interchanges.
- Arterials intersect collectors.
- Collectors intersect local streets.
- Local streets provide connections to private accesses.

D. Locate Signals to Favor through Movements (Principle 4)

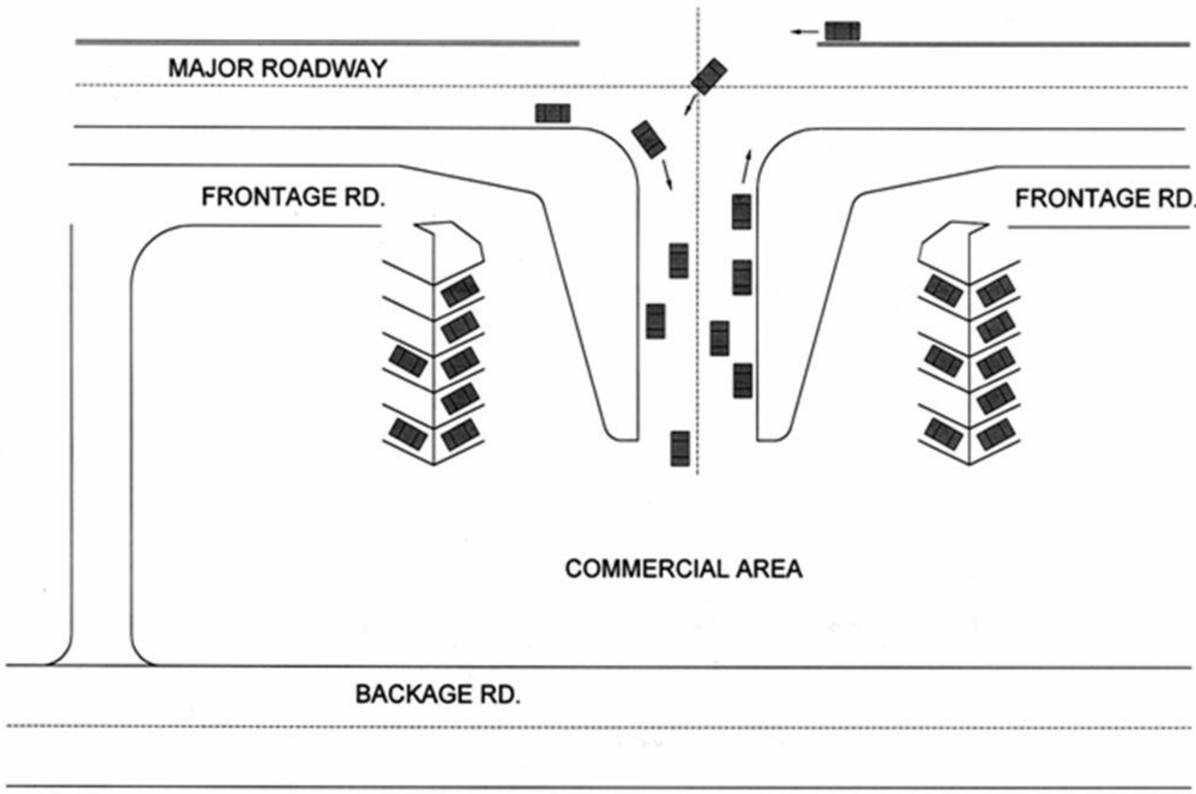
All major arterials, minor arterials, and major collectors within urbanized areas, the urban fringe or areas that may ultimately be subject to urban growth should have long, uniform traffic signal spacing.

- Provides the flexibility to use timing plans that can provide efficient traffic progression over a wide range of speeds and cycle lengths.
- Use a minimum of 1/2 mile spacings on major suburban/urban arterials.
- Use a minimum of 1/4 mile spacings on minor arterials and major collectors where traffic progression is less important than on major arterials.
- Locate cross-roads and full median openings only at locations that conform to the selected spacing interval so that the intersection may be signalized when conditions warrant.
- Where signal location does not conform to recommended spacing, reduce the cross-street green and increase the major street green so as to maintain progression on the major street.

E. Provide a Supporting Street and Circulation System (Principle 10)

- Provide local and collector streets to accommodate access to development.
- Provide access connections between adjacent parcels.
- Require adequate internal circulation for development.
- Provide alternate access from minor roads.
- Provide frontage and backage roads (see Figure 5I-2.01).

Figure 5I-2.01: Frontage and Backage Roads with Adequate Vehicle Queue Storage



Access Location, Spacing, Turn Lanes, and Medians

This section addresses access location, spacing, turn lane and median needs, including TRB Principles of Access Management 5-9:

A. Preserve the Functional Area of Intersections and Interchanges (Principle 5)

AASHTO states, "Ideally, driveways should not be located within the functional area of an intersection or in the influence area of an adjacent driveway. The functional area extends both upstream and downstream from the physical intersection area and includes the longitudinal limits of auxiliary lanes."

1. **Upstream Functional Distance:** The upstream functional distance of the intersection can be further defined as the approach distance to an intersection that is required for the driver to change speeds in order to complete a movement, such as entering an auxiliary lane or slowing down for a turn or signal. The upstream functional distance includes the sum of:

   d_1, distance traveled during driver's perception - reaction time

   d_2, deceleration distance while the driver maneuvers to a stop

   d_3, queue storage length required (50 foot minimum)

Table 5I-3.01: Distance Traveled During Driver's Perception-reaction, (d_1)

Speed (mph)	Rural (feet)	Urban/ Suburban (feet)
20	75	45
30	110	65
40	145	90
50	185	110
60	220	135
70	255	155

Source: TRB Access Management Manual

Table 5I-3.02: Desirable Maneuver Distances, (d_2)

Speed (mph)	Distance (feet)
20	70
30	160
40	275
50	425
60	605
70	820

Source: TRB Access Management Manual

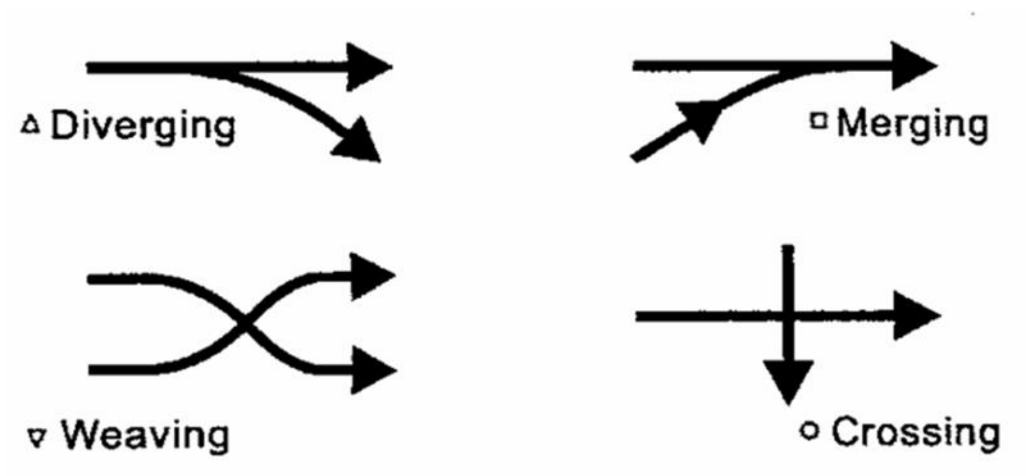
For example, at an urban intersection approach with a 30 mph speed and minimal queuing, the upstream functional distance would be 275 feet (65 feet + 160 feet + 50 feet).

2. **Downstream Functional Distance:** The downstream functional distance from an intersection should be based on upstream functional distance for the proposed adjacent access point. Minimum separation should be no less than the AASHTO stopping sight distance.

B. Limit the Number of Conflict Points (Principle 6)

Traffic conflicts occur where the paths of traffic movements cross. Eliminating or reducing conflict points will simplify the driving task, contributing to improved traffic operations and fewer collisions.

Figure 5I-3.01: Types of Vehicular Conflicts



C. Separate Conflict Areas (Principle 7)

Separating conflict areas allows drivers to address one potential set of conflicts at a time. The higher the speed, the longer the distance a vehicle will travel during a given perception-reaction time. Also, drivers need more time to react to complex conflict areas. Hence minimum separation distances are a function of both the speed of traffic on a given section of roadway and the complexity of the decision with which the driver may be presented. The complexity of the problem, in turn, increases with both the number and type of conflicts and the volume of traffic.

Various methods that can be utilized to separate conflict areas include the following:
- Minimum access spacing
- Minimum corner clearance
- Minimum property line clearance
- Limit the number of accesses per property
- Designate the access for each property

Figure 5I-3.02: Two Lane Undivided Roadway (Single Entrance)

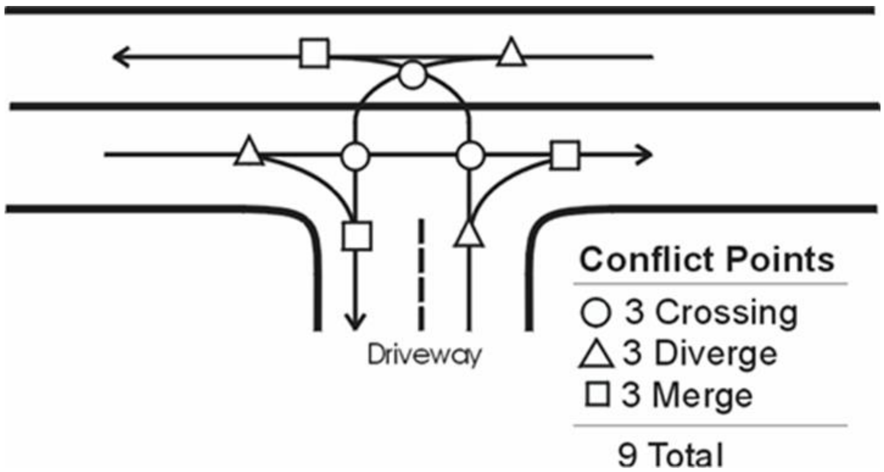
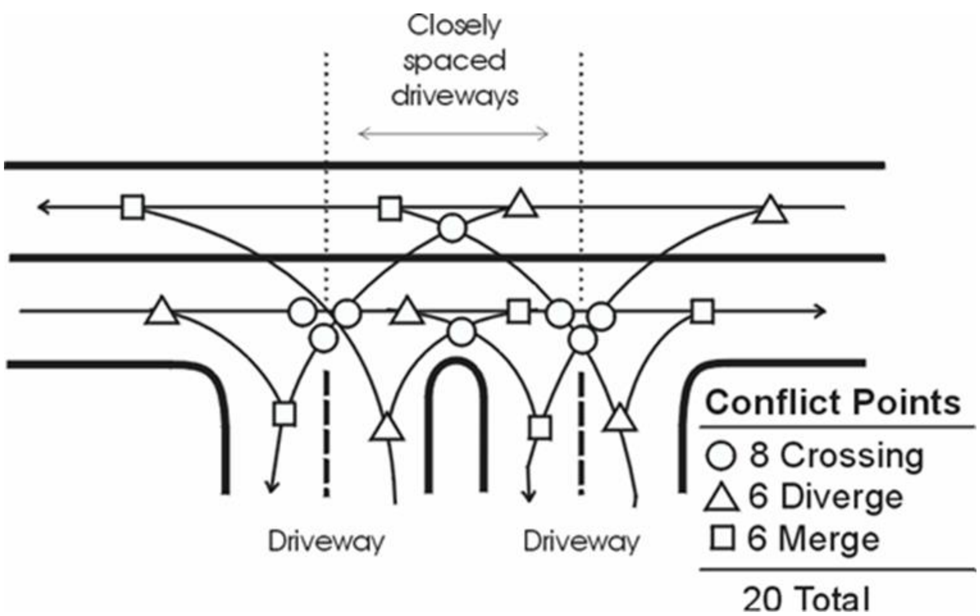


Figure 5I-3.03: Two Lane Undivided Roadway (Closely Spaced Entrances)



1. **Driveway Density:** The number of driveways per block or per mile significantly affects the safety of the corridor. Crash rates increase very quickly as the number of access points increases on arterial and collector roadways.

Table 5I-3.03: Crash Rates (crashes per million vehicle miles traveled) vs. Access Point Density

Access Points per Mile	Approximate Accesses per 500 feet	Representative Crash Rate for an Undivided Roadway	Increase in Crashes Associated with More Access Density
Under 20	Under 2	3.8	----
20 to 40	2 to 4	7.3	+92%
40 to 60	4 to 6	9.4	+147%
Over 60	Over 6	10.6	+179%

Source: National Cooperative Highway Research Program Report 420.

2. **Access Spacing for Major Arterials:** Provide separation between access connections so that drivers can assess potential conflict locations one-at-a-time. Applicable spacing criteria may include:
   - Functional area (Section 5I-2)
   - AASHTO stopping sight distance
   - Preventing right turn overlap (see below)
   - Other criteria as established by the Jurisdiction

   Right turn overlap occurs when a through vehicle must monitor two egress right turning vehicles at once while still performing other driving tasks. By separating access points a proper distance, the overlap does not occur, and the through driver has only one egress right turning vehicle to monitor. Recommended minimum access spacings to avoid right turn overlap shown in Table 5I-3.04 are comparable to AASHTO stopping sight distances.

Table 5I-3.04: Minimum Access Spacing to Prevent Right Turn Overlap

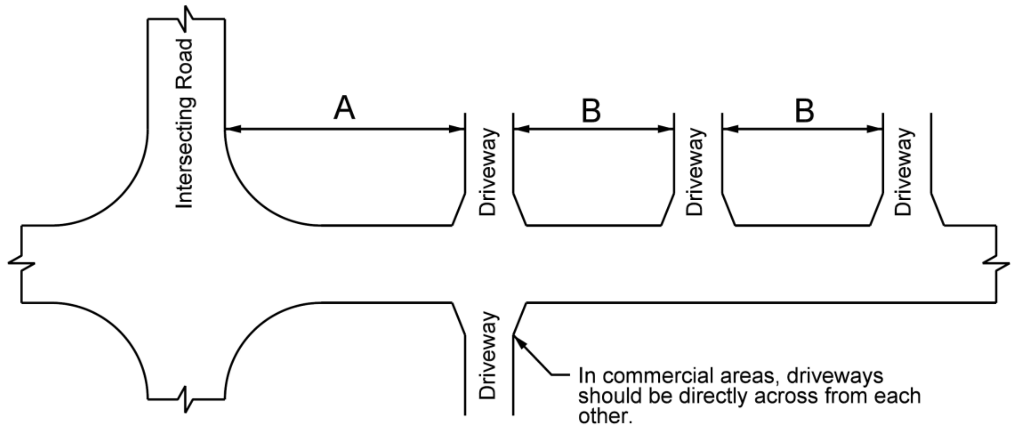
Speed (mph)	Recommended Minimum (feet)[1]
25	120
30	185
35	245
40	300
45	350

[1] Intersection clearance should be the same as driveway spacings or at least as long as stopping sight distance.

Source: Transportation Research Board Record 644, 1977.

3. **Access Spacing for Minor Arterials, Collectors, and Local Streets in Urban/Suburban Areas:** For minor arterials and major collectors, direct access from individual properties should be avoided wherever possible. Property access should be provided from minor collectors, local streets, frontage roads and backage roads. Major arterial access spacing criteria should be used for minor arterials and major collectors when possible.

Table 5I-3.05: Minimum Distance between Driveways or from Intersecting Streets



	Minor Arterial			Collector			Local		
	Res. Area	***C/ I Area***	***Ag Area***	***Res. Area***[3]	***C/I Area***	***Ag Area***	***Res. Area***[3]	***C/I Area***	***Ag Area***
A. Minimum intersection clearance[1]	145’	170’	300’	100’	100’	300’	75’	75’	150’
B. Minimum driveway spacing[2]	100’	200’	300’	75’	100’	300’	---[4]	---[4]	150’

Res = Residential, C/I = Commercial/Industrial
[1] Values are measured from the back of the curb, intersecting road to the adjacent driveway near edge.
[2] Values are measured between driveway edges.
[3] One access drive allowed per lot. Depending on lot size, an additional drive may be allowed upon approval of the Jurisdiction.
[4] See Jurisdictional Engineer for local requirements.

4. **Access Spacing for State Primary Roads:** In rural areas, travel speeds are usually 55 mile per hour and above. This means that driveway spacing in rural areas must be longer to provide for a safe driving environment. On state highways, spacing is also longer because the routes are primarily designed to carry through traffic rather than to serve as property access routes. The more important a route is for through traffic and commerce, the longer the spacing between driveways. The following table shows the State of Iowa's standards for its highway system.

Table 5I-3.06: Iowa DOT Access Control - Minimum Spacings

State Highway Priority	Minimum Spacing Between Driveways	Number of Driveways Per Mile
Priority I (Full Access Control)	Interchanges at roads	N/A
Priority II (Four Lane Divided)	2,640' (minimum)[1] 5,280' (preferred)[1]	2 2
Priority III	1,000' rural (minimum)[1] 1,320' rural (preferred)[1]	4 4
Priority IV(a) Priority IV(b)	600' rural (≥ 45 mph) 300' urban (≤ 40 mph)	8 16
Priority V (Access Right Acquired Between 1956 to 1966)	1 access per 1,000' of frontage not exceeding 2,000'	2 to 5
Priority VI	Safety and need	Varies

[1] Access allowed only at interchanges and selected at-grade locations

5. **Access Spacing for County Roads:** On county roads, the spacing standard should also depend on the nature of the road, e.g. how important the road is for through traffic. Even on the lowest functional levels, some sort of driveway spacing standard is important for traffic safety.

Table 5I-3.07: County Road Minimum Access Spacings

County Road Route Type	Minimum Spacing Between Driveways	Number of Driveways Per Mile
Minor arterials	600'	9
Collectors	300'	18
Local traffic service	150'	36

6. **Additional Access Spacing Considerations:**
   - At a minimum, the upstream corner clearance should be longer than the longest expected queue at the adjacent intersection.
   - High speed, high volume roadways need longer corner clearances whereas the corner clearance on a local street can be much shorter.
   - Residential streets - driveways on corner lots should be located on the lesser street and near the property line most distant from the intersection.
   - Typically, all elements of an access drive, including the radii should be within a property frontage.
   - At a minimum, all driveway geometrics should be along the frontage of the property served by the driveway.
   - On major roadways, the corner clearance should be at least as long as the stopping sight distance so that vehicles turning corners can make safe stops when encountering entering traffic.
   - Encourage owners of adjacent properties to construct joint-use driveways in lieu of separate driveways.
   - Encourage a property owner to replace two or more driveways with a single driveway (or fewer driveways).
   - For adjacent properties, locate joint access on the property line. Reciprocal easements must be executed.

D. Remove Turning Traffic from Through-traffic Lanes (Principle 8)

All driveway and intersection geometrics require that turns be made at very slow speeds and hence result in high speed differentials. Providing auxiliary lanes (left-turn and right-turn bays) is the most effective means of limiting the speed differential. This is especially important on high volume and high speed roadways.

The several methods by which turning vehicles can be removed from through traffic lanes are:

- Install isolated left-turn bay
- Install a nontraversable median with left-turn bays
- Install right-turn deceleration bay
- Install right-turn lane
- Install a continuous two-way left-turn lane (TWLTL)

1. **Turn Lane Warrants for Urban/Suburban Areas (Unsignalized):** Providing left and/or right turn lanes can significantly improve the operation and safety of an intersection. They allow turning vehicles to exit the through traffic lane with reduced speed differential and provide queue storage without interference with through traffic. Rear-end and side-swipe collisions are greatly reduced. Capacity is increased and delay decreased.

   General information regarding improvements for intersections, including guidelines for including left and right turn lanes, can be found in NCHRP Report 457. More specific information and warrants for installation of left turn lanes is presented in NCHRP Report 745.

   In general, the decision to provide turn lanes should be based on safety rather than just capacity. Where practical, left turn lanes should be provided at median openings on divided roads, regardless of projected traffic volumes.

2. **Rural Turn Lane Warrants and Right Turn Deceleration Length (Unsignalized):** See Iowa Department of Transportation’s Design Manual, Chapter 6 - Geometric Design.

3. **Three Lanes with TWLTL:** Three lane roadway designs can be effectively used in situations where there are low to moderate levels of through traffic, yet there are concerns about conflict points and crashes caused by left-turning traffic. The upper limit for using a three lane design is about 17,000 vehicles per day of traffic. Three lane designs are ideal where right-of-way width is limited due to existing land development or other constraints. Three lane roads can either be designed that way originally or can be created by widening an existing two lane route or by modifying an existing four lane undivided route.

4. **Five lanes with TWLTL:** When the average daily traffic (ADT) on a street exceeds about 17,000 vehicles per day, four lane roadways with raised medians or five lane roadways with TWLTL are more appropriate designs. The limit for five lane roadway (with TWLTL) is approximately 24,000 ADT. TWLTL should generally not be used in situations where there are more than four total through lanes.

E. Use Nontraversable Medians to Manage Left Turn Movements (Principle 9)

The majority of access-related crashes involve left turns. Providing nontraversable medians limits and defines locations of left turns, thereby improving safety. Full access median openings that allow left turns from all directions are best provided at signalized intersections and unsignalized junctions of arterial and collector streets. Providing median closures or partial access medians at other intersections and access points reduces the number and types of conflicts.

1. **Median Closures:** Median openings should be considered for closure where:
   - A safety or operational problem is evident and an appropriate retrofit cannot be made.
   - Median width is less than 11 feet, thereby not allowing for construction of left turn lanes.
   - The left-turn bay of a nearby signalized intersection needs to be extended.
   - A pattern of left-turn crashes is evident.
   - Heavy pedestrian use is predicted or crashes involving pedestrians have occurred at the intersection.

   Implementation of a median closure involves providing a section of median of the same design as existing on either side of the opening. The following should be considered during design:
   - Tree lines, building lines, and lighting may head drivers into believing the median can be crossed.
   - Visual cues should be provided to clearly inform drivers that the opening has been closed.
   - The need for visual cues is especially critical during nighttime hours where a four way intersection previously existed or there are access drives directly opposite each other.
   - Minimum 4 feet median width face-to-face of curbs is recommended.
   - Select and locate landscaping materials to delineate the median while considering potential sight distance obstructions.

Figure 5I-3.04: Two Lane Roadway Conflict Points at Typical Three Way Intersection or Driveway

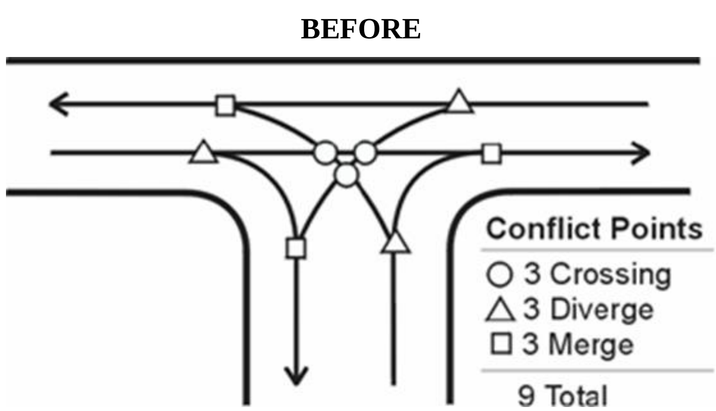


Figure 5I-3.05: Two Lane Roadway with Raised Median Closure (right-in/right-out only access)

AFTER

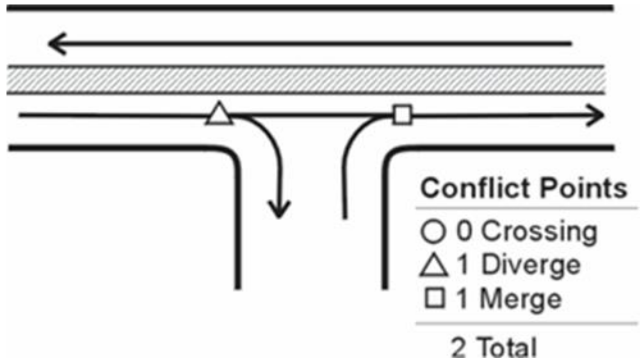
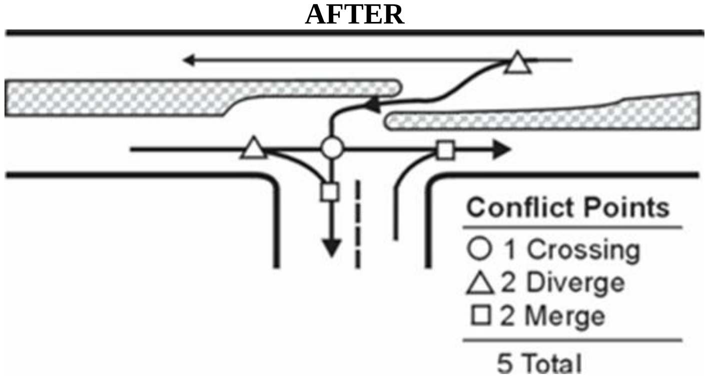


Figure 5I-3.06: Two Lane Roadway with Raised Median (left turn ingress allowed into driveway)

AFTER



2. **Raised Medians vs. Two Way Left Turn Lanes:**
   - Because they are the most restrictive access management treatment, constructing raised center medians along arterials is often very controversial among business and property owners. Two way left turn lanes (TWLTL) are usually much less controversial. Business persons and property owners feel that installation of raised medians will have a large, negative impact on their customers, sales, and property values. Therefore, TWLTL are often suggested as compromise solutions.
   - Arterial roadways with raised medians are statistically safer and operate better than any other configuration. Research indicates that raised median roadways are significantly safer than undivided roadways in urban areas. When traffic volume on an arterial roadway is projected to exceed about 24,000 average annual daily traffic (AADT) during the next 20 years, including a raised median is prudent.
   - In general, TWLTL projects function well when traffic levels are moderate, when the percentage of vehicles turning as opposed to traveling through is high, and when the density of commercial driveways is low. TWLTL will function very well on most arterials where AADT is in the range of 10,000 to 24,000 AADT (five lane TWLTL).
   - TWLTL projects can also work very well in places where the number of driveways per block or mile is high, but the land use is such that not many turning movements are generated per hour. An example would be an arterial street passing through a predominately residential area.

- TWLTL are much less effective in situations where commercial driveway densities are high and these driveways are spaced close together. In such a situation, the number of conflict points is high, and this will be reflected in crash experience. Research from many states indicates that raised median roadways are always safer than TWLTL roadways. If TWLTL are considered, driveway density and driveway spacing must be managed very aggressively.

Table 5I-3.08: Crash Rates (crashes per million vehicle miles traveled) vs. Median Type

Access Points Per Mile	Undivided (Painted Centerline) Crash Rate	TWLTL Crash Rate	Raised Median Crash Rate	Rate Reduction Raised Median Versus TWLTL
Less than 20	3.8	3.4	2.9	-0.5 (15%)
20 to 40	7.3	5.9	5.1	-0.8 (14%)
40 to 60	9.4	7.4	6.5	-0.9 (12%)
Over 60	10.6	9.2	8.2	-1.0 (11%)

Source: National Cooperative Highway Research Program Report 420

F. References

Transportation Research Board - National Cooperative Highway Research Program (NCHRP). *NCHRP Report 420: Impacts of Access Management Techniques*. National Academy Press. Washington, DC. 1999.

Transportation Research Board - National Cooperative Highway Research Program (NCHRP). *NCHRP Report 644: Guidelines for Conducting a Disparity and Availability Study for Federal DBE Program*. National Academy Press. Washington, DC. 1977.

Driveway Design Criteria

A. General

For efficient and safe operations, access drives and minor public street intersections can be improved by the following:

- Smooth vertical geometrics
- Adequate driveway throat width and curb return radii
- Provide adequate sight distance
- Additional egress lane
- Quality driveway construction
- Define the ingress and egress sides of the access drive

Refer to NCHRP Report 659 - Guide for the Geometric Design of Driveways for supplemental information.

B. Width Measurement

1. The width of an entrance with a radius return or with a flared taper that connects to a curb and gutter roadway is measured at a point 10 feet back from the roadway curb. The curb opening may exceed the maximum allowable width of the entrance to accommodate the allowable radius or taper.

2. The width of an entrance that connects to a rural roadway (no curb and gutter) is measured across the top of the entrance at the culvert line or at the location where a culvert would normally be placed.

C. Dimensions

Figure 5I-4.01: Entrance Dimensions

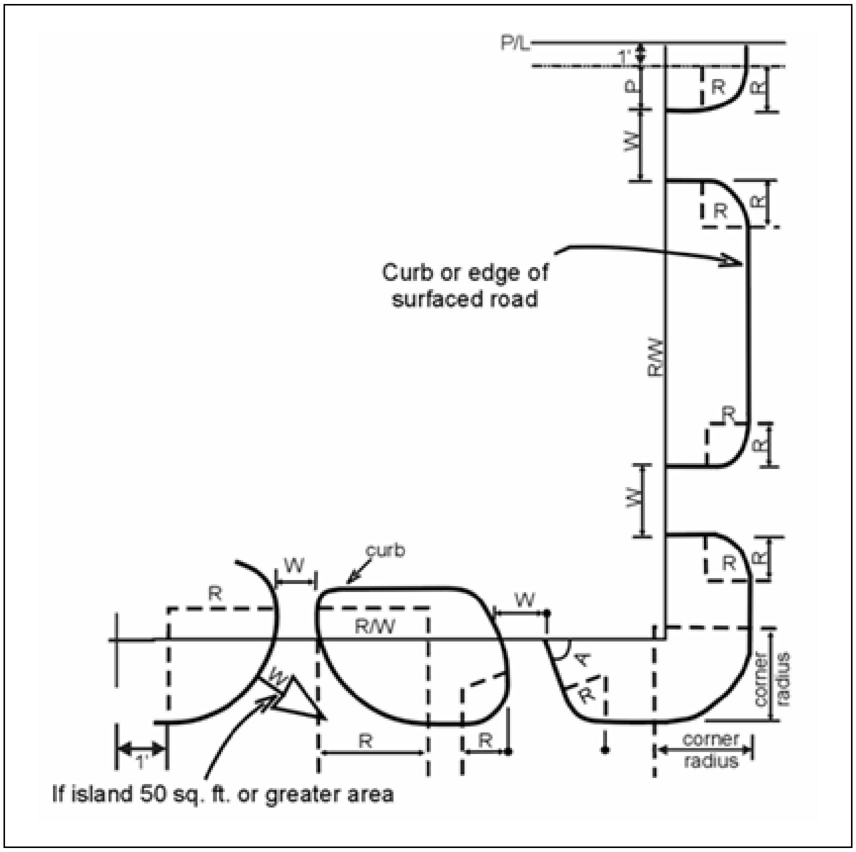


Table 5I-4.01: Driveway Dimensions[1]
(all dimensions are in feet)

Dimension Reference (See Figure 5I-4.01)		Major Arterial Street				Minor Arterial Street				Collector (Major and Minor)				Local Street			
		Residential	Commercial	Industrial	Agricultural	Residential	Commercial	Industrial	Agricultural	Residential	Commercial	Industrial	Agricultural	Residential	Commercial	Industrial	Agricultural
Width Minimum Maximum	W	15 30	24 45	24 45	20 30	15 30	24 45	24 45	20 30	10 24	24 40	24 45	20 30	10 24	24 32	24 40	20 30
Right-turn Radius[2] Minimum Maximum	R	10 25	10 35	25 50	25 35	10 25	10 35	25 50	25 35	10 25	10 35	25 50	25 35	10 15	10 20	10 30	20 35
Min. Acute Angle[3] Pref. Acute Angle	A	60° 90°	70° 90°	70° 90°	70° 90°	60° 90°	70° 90°	70° 90°	70° 90°	60° 90°	70° 90°	70° 90°	70° 90°	60° 90°	70° 90°	70° 90°	70° 90°
Min. Pavement Thickness (inches)	T	6/8	7/9	*	6	6	7	*	6	6	7	*	6	6	7	*	6

[1] Major entrances require special design.
[2] 3 foot flares (F) may be used for residential and agricultural entrances.
[3] Any variation from 90° will be evaluated on a case by case basis. The minimum acute angle (measured from the edge of the pavement) is 60°.
* Requires special design.

1. The width (W) shown applies to rural routes and city streets including neighborhood business, residential, and industrial streets. For joint entrances centered on property lines, the entrance width may increase 5 feet rounded to the nearest 5 foot interval but should not exceed 45 feet. In rural areas (open ditch roadways) widths for paved entrances should include an additional 4 feet for shoulders (Minimum 2 feet shoulders each side).

2. The radius (R) for agricultural uses will vary according to the following intersecting acute angles:

Table 5I-4.02: Agricultural Acute Angle and Radius

Acute Angle	Acute Radius Decrease (feet)	Obtuse Radius Increase (feet)
85° to 90°	0	0
75° to 85°	5 feet	5 feet
65° to 75°	5 feet	10 feet
60° to 65°	10 feet	15 feet

Where the entrance radius specified is greater than the distance between the back of curb and the front edge of the sidewalk the radius may be reduced to meet the available space but should be no less than 10 feet. An option to the radius under this condition is the use of flared entrances. When a flare is used, it should be 3 feet wide and should be constructed from the back of curb to the sidewalk. If no sidewalk exists, flares should be 10 feet long.

3. For individual properties, the number of entrances should be as follows:

   a. Single Family (SF) Residential: Each SF residential property is limited to one access point. However, where houses are located on corner lots, have extra wide frontage, or on heavy traveled roadway more than one access point may be allowed to eliminate backing out on a heavily traveled roadway.

   b. Multi-family (MF) Residential: Access is determined by information provided by the Owner/Developer in a Traffic Impact Report and by comments generated during the Jurisdiction Engineer's review and acceptance of that report.

   c. Commercial: Commercial property having less than 150 feet of frontage and located mid-block is limited to one access point to the street. An exception to this rule may be where a building is constructed in the middle of a lot and parking is provided for each side of the building. A second access point may be allowed for commercial property having more than 150 feet of frontage. For commercial property located on a corner, one access to each street may be allowed, provided dimensions are adequate from the intersecting street to the proposed entrance. (See Section 5I-3 - Access Location, Spacing, Turn Lanes, and Medians).

   d. Industrial: Access is determined on a case-by-case basis. The Jurisdiction will consider good traffic engineering practice and may require information to be provided by the applicant in a Traffic Impact Report. (See Section 5I-3 - Access Location, Spacing, Turn Lanes, and Medians).

   e. Agricultural: Access with adequate frontage may be authorized with more than two accesses at not less than 300 feet intervals provided a minimum distance of 30 feet is maintained from the inlet and outlet of two adjacent culverts.

   In all cases, the location of the access will be such that the taper or radius does not extend beyond the extension of the property line. In general, all construction must occur only on the property owner's frontage.

4. Minimum acute angle (A) is measured from the edge of pavement and is generally based on one-way operation. For two-way driveways, and in high pedestrian activity areas, the minimum angle should be 70 degrees. Entrances should be placed at 90 degrees whenever possible.

5. The entrance pavement thickness (T) is based on the following:

   PCC - Class "A" or "C" - 4,000 psi
   HMA - Greater than or equal to 100K ESAL (optional for rural area).

   For those entrances not paved, 6 inches (min.) of Class "A" gravel should be required.

D. Sight Distance

1. Sight distance is based upon AASHTO stopping sight distance criteria. However, the height of an object is increased from 2.0 feet to 3.5 feet to acknowledge an approaching vehicle as the "object" of concern. Therefore, sight distance at an access location is measured from the driver's height of eye (3.5 feet) to the height of approaching vehicle (3.5 feet).

2. An access location should be established where desirable sight distance is available, as shown below.

Table 5I-4.03: Desirable Sight Distances

Design Speed (mph)	**Intersection Sight Distance (feet)**	
	Left Turn from Stop	***Right Turn from Stop and Crossing Maneuver***
55	610	530
50	555	480
45	500	430
40	445	385
35	390	335
30	335	290
25	280	240

Note: the sight distances shown above are for a stopped passenger car to turn onto or cross a two lane roadway with no median and grades of 3% or less. For conditions other than those stated, refer to the 2004 AASHTO "Green Book" for additional information.

Source: Based on Exhibit 9-55 and Exhibit 9-58 of the 2004 AASHTO "Green Book."

3. On a four lane divided primary highway where access is proposed at a location not to be served by a median crossover, sight distance is required only in the direction of the flow of traffic.

E. Driveway Grades

1. **Slopes vs. Speed Differential:** Driveway slope is important due to speed differential. Turning vehicles must slow appreciably to enter a driveway. The steeper the driveway, the more vehicles must slow in order to prevent "bottoming out", increasing the speed differential with through traffic and increasing the possibility of rear-end collisions.

Table 5I-4.04: Driveway Slope and Entry Speed

Driveway Slope	Typical Driveway Entry Speed
Greater than 15%	Less than 8 mph
14 to 15%	8 mph
12 to 13%	9 mph
10 to 11%	10 mph
8 to 9%	11 mph
6 to 7%	12 mph
4 to 5%	13 mph
2 to 3%	14 mph
0 to 2%	About 15 mph

Source: Oregon State University, 1998

A speed differential much above 20 miles per hour begins to present safety concerns. When the speed differential becomes very large (say, 30 to 35 miles per hour), the likelihood of traffic crashes involving fast-moving through vehicles colliding with turning vehicles increases very quickly. Rear-end collisions are very common on roads and streets when large speed differentials exist and the density of commercial driveways is high. When the speed differential is high, it is also more likely that when crashes do occur they will be more severe, causing greater property damage and a greater chance of injury or fatalities. Keeping the speed differential low is very important for safety reasons, as the table below indicates.

Table 5I-4.05: Speed Differential and Crashes

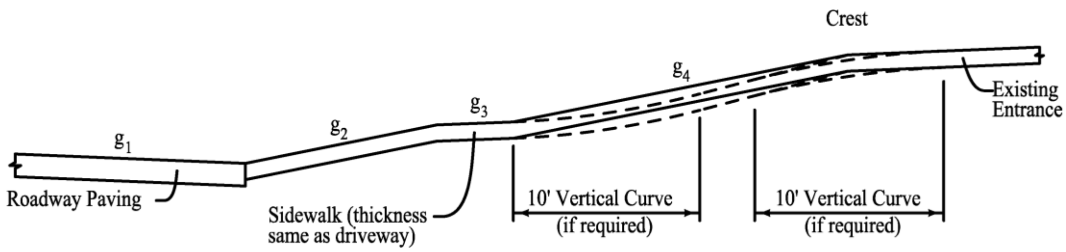
When the Speed Differential Between Turning and Through Traffic Is:	The Likelihood of Crashes Is:
10 mph	Low
20 mph	3 times greater than at 10 mph
30 mph	23 times greater than at 10 mph
35 mph	90 times greater than at 10 mph

Source: Oregon State University, 1998

2. **Vertical Profile:** A driveway's vertical profile should allow a smooth transition to and from the roadway. The National Highway Institute's course workbook on Access Management recommends the following maximum driveway slopes for urban/suburban streets:
   - Arterial 3 to 4%
   - Collector 5 to 6%
   - Local Less than 8% (may use 9% in special areas)

   These slopes were chosen to keep the speed differential at or below 20 miles per hour. See Figures 5I-4.02A and 5I-4.02B.

Figure 5I-4.02A: Typical Section - Commercial/Industrial and Residential Entrance



1. Algebraic Difference Between g1 and g2:
   a. Commercial/Industrial: Not to exceed 9%
   b. Residential: Not to exceed 12%

2. Algebraic Difference Between g2 and g3:
   a. Commercial/Industrial: Not to exceed 6%
   b. Residential: Not to exceed 8%

3. Maximum Slope of g3 = 2% (ADA compliance)

4. Algebraic Difference g3 to g4:
   a. Commercial/Industrial: Not to exceed 5%
   b. Residential: Not to exceed 8%
   c. 10 foot vertical curve required for change in grade exceeding 5%

5. Maximum Slope of g4:
   a. Commercial/Industrial: 7%
   b. Residential: 10%

6. 10 foot vertical curve required for change in grade from g4 to existing exceeding 5%

7. If the above grade restrictions require a depressed sidewalk through the driveway, a transition section should be provided between the normal sidewalk grade and the depressed section. As a general rule, use the following transition lengths:

Elevation Difference from Normal Sidewalk Grade (inches)	Transition Distance (feet)
1 to 2	8
2 to 4	12
4 to 6	16
Greater than 6	Desirable max. slope is 16:1 Absolute max. slope is 12:1

3. Non-curb and Gutter Roadways:

a. Private drive access to local, collector, or arterial streets that have no curb and/or gutter improvements should be constructed with grades and dimensions as shown in Figure 5I-4.03. Heavily used driveways connected to existing gravel roadways may require an 8 inch deep compacted Class “A” crushed stone base material. The driveway pavement should be extended to the proposed roadway pavement width, if known, or 15.5 feet from the centerline, if not known. A culvert properly sized for the ditch flow should be installed at the established roadside ditch flowline beneath the private drive access. Culvert should be 15 inches minimum and 18 inches desirable. The culvert should be either corrugated metal or reinforced concrete pipe with minimum of 1 foot of cover over the pipe per the Jurisdiction’s requirements.

b. For Farm to Market (FM) roads, when grading on new construction, or complete reconstruction projects on paved (or to be paved) FM roads, the following will apply:
   1) When a culvert is not required, the following slopes will apply.
      - 10:1 slope of flatter from shoulder line to ditch bottom in clear zone area.
      - 6:1 slope or flatter from clear zone area to the right-of-way line.
      - 10:1 to 6:1 transition zone.
   2) When a culvert is required, the following slopes will apply.
      - 8:1 slope or flatter from shoulder line to normal placement of a culvert.
      - 6:1 slope or flatter from culvert area to the right-of-way line.
      - 8:1 to 6:1 transition zone.

   For remaining open ditch roadways (paved or non-paved), the sideslopes will be 6:1 for posted speeds of 40 mph or greater, and 4:1 for posted speeds of less than 40 mph.

F. Other Criteria

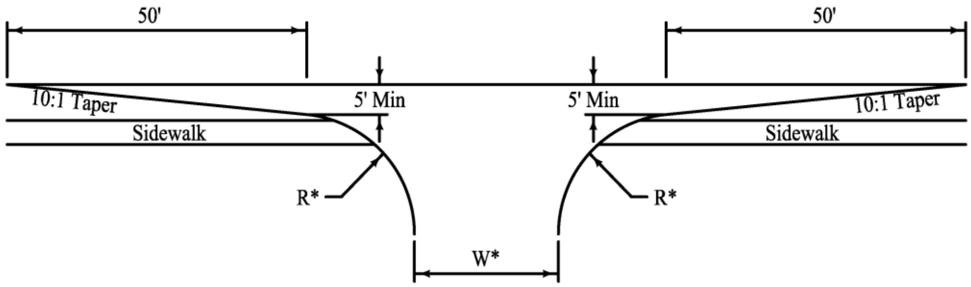
1. **Utility Conflicts:** Any adjustments made to utility poles, street light standards, fire hydrants, catch basins or intakes, traffic signs and signals, or other public improvements or installations, which are necessary as the result of the curb openings or driveways, should be accomplished with no additional cost to the Jurisdiction.

2. **Access Signs:** Driveway approaches, whereby the driveway is to serve as an entrance only or as an exit only, should be appropriately signed by, and at the expense of, the property owner subject to approval of the Jurisdiction Engineer.

3. **Abandoned Driveways:** Any curb opening or driveway that has been abandoned should be restored by the property owner.

4. **Offset Radius and Driveway Tapers:** For driveways without a right turn lane on the street approach, providing an offset radius and driveway taper can help reduce speed differential between turning and through traffic, reducing the possibility of rear-end crashes. Figure 5I-4.03 shows a typical taper system that can be effectively used. The downstream taper for right turns from the driveway may be considered optional. Right-of-way restrictions may limit the use of this method.

Figure 5I-4.03: Offset Radius and Driveway Tapers



*Driveway radii and widths vary depending on entrance type, street classification, and zoning.

5. **Sidewalks:** For driveways that intersect pedestrian circulation paths and pedestrian access routes (sidewalks and shared use paths), all ADA requirements must be met. See Chapter 12 - Sidewalks and Bicycle Facilities.

G. References

Institute of Traffic Engineers. *Transportation and Land Development*. 1988.

Oregon Department of Transportation. *Driveway Profile Study - Summary of Results*. 1998.

Transportation Research Board - National Cooperative Highway Research Program (NCHRP). *NCHRP Report 659: Guide for the Geometric Design of Driveways for Supplemental Information*. National Academy Press. Washington, DC. 2010.

Traffic Impact Studies

A. General

A traffic impact study may be required for commercial, industrial, or residential developments in obtaining site plan, rezoning, or access permit approval. The Jurisdictional Engineer must be contacted to determine if a traffic impact study is required. If a study is required, the study scope (study limits, analysis years, scenarios, etc.) should be determined through discussion with the Jurisdictional Engineer.

B. Study Process

Traffic impact studies typically include the following elements. Specific tasks, level of analysis, and documentation requirements will depend on the specific needs of the study and Jurisdictional requirements.

1. **Data Collection:** Gather and review needed information regarding existing and proposed conditions, possibly including:
   - Current and historic daily and hourly traffic volume counts.
   - Recent intersection turning movement counts.
   - Projected volumes from previous studies, travel demand models, or area transportation plans.
   - Current land uses, densities, and occupancy near the site.
   - Preliminary site plan for proposed development with land uses, building areas, phasing and completion dates, and proposed access locations identified.
   - Other approved projects and anticipated development near the site.
   - Land use and zoning plans near the site.
   - Current street system information (functional classifications, lane configurations, speed limits, access locations, traffic control, parking)
   - Traffic signal locations, phasing, timing, and coordination.
   - Planned or proposed transportation improvement projects in the area.
   - Crash history (3 to 5 years), if safety concerns have been identified.

2. **Background Traffic:** Determine estimated background traffic for analysis years and scenarios. For simple studies with a short-term analysis year, this may simply be current traffic count data. For more complex studies or longer-range analysis years, background traffic may also include trip generation from proposed area development or land uses, annual traffic growth rates, and/or area travel demand model traffic forecasts.

3. **Site Traffic:** If available, local data should be used in determining estimated daily and peak hour trip generation for the site. If local data is not available, the latest edition of *ITE Trip Generation* or other national data should be used as a basis for estimating trip generation for the site. Sound judgment must be used in reviewing, adjusting, and applying published trip generation data. Depending on specific site characteristics, generated trips may need to be adjusted for mixed-use developments (internal or multi-purpose trips) or unique pedestrian, bicycle, or transit usage. After site-generated trips are prepared, they are distributed and assigned to the study area roadway system, considering the following:

- Type of proposed development and area from which trips will be attracted
- Size of proposed development
- Surrounding land uses and population density
- Proposed site access locations and configurations
- Proposed or anticipated traffic control at access points
- Conditions of surrounding street system
- Competing developments, where applicable

Site traffic is normally distributed in terms of a percentage of inbound or outbound traffic at each study access point, intersection, or ramp junction for each analysis period. These distributions are then used to calculate assigned peak hour traffic turning movement volumes. Assigned traffic is combined with background traffic to determine total traffic for each analysis scenario to be analyzed. Depending on the type of development, the total traffic is often adjusted for pass-by trips. Pass-by trips (or diverted trips) are those trips already on the adjacent street network (background traffic) that will enter and exit the site.

4. **Analysis:** Total peak hour traffic for each study access point, intersection, and ramp junction is analyzed for each analysis scenario according to current *Highway Capacity Manual* (HCM) procedures. Analyses will determine projected vehicle delays, volume/capacity ratios, levels of service, and vehicle queuing. Analysis software such as Highway Capacity Software (HCS) or *Synchro* is typically used. For complicated roadway systems or conditions, additional simulation analysis may also be necessary. In addition to capacity analysis results, several other factors should be considered in evaluating traffic operations for the study, including the following:
   - Crash history, crash rates, predominate crash types, and safety concerns
   - Traffic control needs, including MUTCD traffic signal warrant analysis
   - Anticipated impacts and vehicle queuing and access point/intersection spacing
   - On-site parking, circulation, and potential impacts on adjacent street system
   - Pedestrian, bicycle, and transit needs
   - Service and delivery vehicle access

5. **Improvement Needs:** Based on analysis results, identify access and/or street network improvement needs necessary to provide acceptable operations. Perform capacity analyses with proposed improvements to evaluate expected operations. Typical improvement needs may include:
   - Adding or lengthening intersection turn lanes
   - Widening, reconstructing, or reconfiguring streets to provide needed lanes and geometry
   - Constructing new street connections for access or through traffic
   - Interchange modifications
   - Changes to traffic control or intersection type (such as all-way STOP, signalization, right-in/right-out only access, or roundabout)
   - Changes to traffic signal phasing, timing, and coordination
   - Access management (combining, eliminating, adding, or improving spacing of access points)
   - Revising site circulation or on-site queue storage
   - Signing or pavement marking modifications

6. **Report:** Prepare and submit to Jurisdictional Engineer a draft traffic impact study report summarizing data collected, analyses performed, and recommendations. Include appropriate tables and graphics. Finalize the report based on comments or concurrence received from the Jurisdictional Engineer.

C. Iowa DOT Access Permits

If a new or modified access is proposed from a highway under the jurisdiction of the Iowa DOT, the applicable District office should be contacted early in the project development to determine access requirements, limitations, and documentation needed. Guidance is provided in the Iowa DOT *Iowa Primary Road Access Management Policy*. Analyses and documentation required will depend on the proposed type and size of development, current access provided, and priority type of the highway. For proposed Type "A" entrances, detailed geometric plans, opening year and full-build year traffic data, and proposed site data are required. Capacity analysis and MUTCD traffic signal warrant analysis may also be required.

D. References

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General Information for Pavement Preservation Program

A. Concept

As effective financial resources for management of infrastructure systems continue to decline, it is necessary to explore different techniques to meet the needs of the public. This is particularly the case for pavements. One such technique is to develop a program of pavement preservation. Pavement preservation techniques have been in use for many years, but most transportation agencies have not developed a complete long-term pavement preservation program that is a part of an overall asset management program. The implementation of a pavement preservation program focuses on maximizing the condition and life of a network of pavements while minimizing the network's life-cycle cost.

The concept of a long-term pavement preservation program is a different approach than has been done in the past; the biggest difference is preservation focuses on being proactive as opposed to reactive. The concept of adopting a proactive preservation approach enables agencies to reduce the frequency of costly, time-consuming rehabilitation and reconstruction projects. It is important to understand, that pavement preservation activities do not include any structural upgrades to the pavement. All treatments are non-structural in nature.

The process to establish a long-term pavement preservation program involves data gathering for each particular pavement section concerning its construction and previous maintenance history, examining the current condition of the pavement, and then determining the appropriate treatment technique to preserve and extend the pavement life.

Customer satisfaction is the greatest benefit of a successful pavement preservation program. From project selection to treatment selection to construction, a good pavement preservation program will benefit users. Safer roads, faster repairs, and a pavement network in better condition that needs fewer repairs are the logical outcomes of a preventative maintenance program.

Transitioning from the traditional rehabilitation and reconstruction activities to a greater emphasis on pavement preservation can be difficult and will require an active educational program. Government officials and the public must be convinced that it is best to provide for continuation of proactive activities to maintain a roadway in good condition as opposed to delaying preservation until major work is required. NCHRP Repot 742 *Communicating the Value of Preservation: A Playbook* is a good source of educational program examples.

B. Definitions

Minor Rehabilitation: Non-structural enhancements made to existing pavements to eliminate age related top-down surface deterioration.

Pavement Preservation: A program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extends pavement life, improves safety, and meets motorist expectations (FHWA, 2005). A pavement preservation program is achieved through the application of routine maintenance, preventative maintenance, and minor non-structural rehabilitation.

Preventative Maintenance: A planned strategy of cost effective treatments to an existing roadway system that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without significantly increasing the structural capacity. (AASHTO, 1997) Preventive maintenance treatments are applied to pavements in good condition and that have significant remaining service life. Preventative maintenance activities are often undertaken by contract, although in-house staff can accomplish them as well.

Routine Maintenance: Day-to-day activities that are scheduled by maintenance personnel to maintain and preserve the condition of the roadways at a satisfactory level of service. Routine maintenance activities are traditionally completed by in-house forces and not contracted out.

C. Benefits

Preventive maintenance activities are the backbone of a pavement preservation program. Simply constructing a roadway and allowing it to deteriorate over its design life is not acceptable. Some of the benefits of a strong preventative maintenance program, besides higher customer satisfaction, include:
- Better informed decisions
- Improved pavement condition
- Cost savings
- Improved strategies and techniques
- Improved safety
- Extended pavement life

An essential part of the pavement preservation process is utilizing all forms of information about a pavement section as the appropriate treatment type is selected. Once the history of a pavement section and the current condition are known, the type of treatment can be selected and the timing of the treatment can be established.

Through the implementation of a pavement preservation program, pavement conditions stabilize because pavements in good condition are maintained in good condition longer. Although it is difficult to prove, especially in the early stages of a pavement preservation program when there are a number of pavements that require major rehabilitation or reconstruction, many agencies have shown that the cost to maintain their roadway system is reduced because of less expensive treatments and extended service lives.

As pavement preservation decisions are being made, agencies are examining different materials and processes that will improve performance of the treatments. The use of high quality materials and quality control are increasingly important.

To users, safety is one of the fundamental expectations they have as they travel. Most pavement preservation treatments will improve the surface characteristics of smoothness and friction. Other surface defects are also addressed. In addition, pavement preservation treatments are less disruptive to traffic movements and extend the time when reconstruction with its extensive impact to traffic flow is needed.

Pavement Preservation Process

A. Pavement Deterioration

The concept behind pavement preservation is to treat pavements while they are still in good condition and without serious structural damage. Successive, systematic treatments will extend the service life and delay the more expensive major reconstruction project. In order to apply the appropriate preventative maintenance treatment at the optimum time, the history and the current condition of a pavement section must be evaluated. Preventative maintenance techniques address the pavement surface condition and do not impact the structural capacity of the pavement. If the structural carrying capacity is affecting the condition of the roadway, it is probably not a candidate for preventative maintenance and it is best to program as a reconstruction project. The causes of any pavement deterioration for various types of pavements must be accurately determined. Typical causes of deterioration for each pavement type include the following:

1. **Flexible Pavements:** Flexible pavements, hot mix asphalt, or other bituminous pavements are affected by traffic, environmental/aging, material problems, and moisture intrusion. These elements impact the pavement in different ways:

   a. **Traffic:** Traffic can lead to load related distresses, such as rutting or fatigue cracking in the wheel paths. Fatigue can lead to development of potholes. Also polishing of the surface leads to friction loss.

   b. **Environmental/Aging:** The environment and aging can lead to oxidation of the asphalt, block cracking, and raveling. Environmental elements can also cause the development of thermal cracks, which are seen as regularly spaced transverse cracks.

   c. **Material Problems:** Material problems include bleeding, shoving, stripping, and surface deformation.

   d. **Moisture Infiltration:** Moisture infiltration can cause further breakdown of existing cracks and thus increased roughness.

2. **Rigid Pavements:** For rigid PCC pavements, the general causes of deterioration include traffic loading, environmental factors, material problems, construction problems, joint deterioration, and moisture infiltration.

   a. **Traffic:** Traffic can lead to load related distress, such as mid-slab cracking, pumping, faulting, and corner breaks. Polishing and the subsequent loss of friction is also traffic related.

   b. **Environmental and Materials:** D-cracking and alkali-silica reactivity (ASR) are material problems. Freeze-thaw action and poor entrained air can affect joint stability.

   c. **Construction Problems:** Construction quality can cause cracking and surface defects in the form of map cracking and spalls.

d. **Joint Deterioration:** Incompressible materials in the joint from poor joint seal maintenance can cause joint spalls.

e. **Moisture Infiltration:** Moisture can lead to further breakdown of cracks and spalls and increased roughness. It can also contribute to pumping, transverse joint faulting, and corner breaks.

B. Evaluating Pavement Conditions

Numerous pieces of information need to be examined in order to determine if the pavement section is a candidate for preventative maintenance and the selection of the type of treatment that best meets that pavement section's needs. The extent of the evaluation process will vary depending on the roadway classification and the type of project. In each case, once the information is compiled, engineering judgment must be applied to determine the correct treatment to use to address the distresses exhibited by each section of pavement.

1. **Background Data:** Obtain data from project files, such as original design parameters, construction information regarding materials, subgrade/subbase information, current traffic data, and maintenance activities undertaken on that roadway section. This information can sometimes be difficult to locate if a good record system has not been established, but as much information as possible should be compiled. Discussions with agency engineering and maintenance staff members can potentially fill in gaps in records.

2. **Existing Condition:** Undertake a visual site inspection to obtain information about the condition of the pavement. Ascertain information on what types of distress are exhibited by the pavement section. Note any restrictions such as right-of-way limitations, presences of bridges, drainage problems, and obstructions.

   The specific severity and extent of each type of pavement distress should be examined closely. Additional field testing such as falling weight deflectometer (FWD), pavement cores, friction testing, splash and spray, and materials testing may be necessary. The extent of the additional testing may be dependent on the roadway classification. Much of this information should be contained in a pavement management system (PMS). The PMS can be in many forms including a sophisticated computer program, a relatively simple spreadsheet, or notes accumulated by maintenance personnel.

   The Iowa DOT has a program of data collection on all roads in the state and is one source of pavement condition information. The program in administered by the CTRE program at The Institute for Transportation at Iowa State University. Information can be accessed at http://www.ctre.iastate.edu/ipmp/services/. It is necessary to undertake additional effort to convert the raw data to useable information.

3. **Future Projections:** It is also important to evaluate any future changes that may be expected for each roadway section. Changes in adjacent land use or improvements to area roadways could impact the traffic volume and the vehicle mix of a roadway section. Long-range transportation planning documents should provide this information. This information is critical to understanding the service life expectancy of the existing pavement and then the subsequent preventative maintenance treatments to match that service life.

Preventative Maintenance Treatment Type Selection

A. Introduction

Once all of the background, existing pavement condition, and future changes have been determined for a pavement section, the appropriate preventative maintenance treatment or treatments can be selected. Professional engineering judgment is critical in order to analyze the available data and select the most effective treatment. The selection of the most appropriate treatment must also take into consideration the availability of qualified contractors and the availability of quality materials to accomplish the work. In some instances, a combination of treatments may be needed to maintain the pavement in good condition.

In addition to the technical analysis, it is important to complete a financial review that will compare the various treatment types, their expected service life, and the associated costs. Comparisons can be made by calculating a simplified annualized cost through dividing the estimated cost of the treatment by the expected service life of each treatment type.

B. Flexible Pavement Treatment Types

Several traditional preventative maintenance treatments are available for flexible pavements. These include:

- Crack filling
- Crack sealing
- Full/partial depth patches
- Fog seals
- Slurry seals
- Microsurfacing
- Bituminous seal coats
- Milling
- Thin overlays

The above treatments will be described in greater detail. Additional treatments are available, but generally involve use of proprietary materials or processes or are not included in this manual. If appropriate, designers should include some of these other treatments in their analyses. These treatments are only effective if there are no structural problems with the pavement or the supporting subbase/subgrade.

1. **Crack Filling:** Crack filling is a good treatment method for reducing intrusion of moisture through the pavement slab. It will assist in reducing further crack deterioration, associated roughness, and rutting. Crack filling will traditionally involve minimal preparation and use of lower quality bituminous materials. Treatment should occur during cool, dry weather, which will provide for wider crack widths. Proper cleaning and a dry condition are the key to achieving good performance and maximizing service life. Cracks should be cleaned to a depth of 3 inches. Crack filling material is generally an asphalt emulsion since actually sealing of the crack is not

expected. Crack filling is appropriate for non-working cracks between 1/4 inch and 1 inch wide. The potential exists for increased roughness and loss of surface friction if the joint is overfilled. See SUDAS Specifications Section 7040, 3.07.Service life is from 2 to 4 years.

2. **Crack Sealing:** Crack sealing is effective at reducing moisture intrusion in the pavement as well as minimizing the amount of incompressible materials in the cracks. It differs from crack filling in that it is used on working cracks and involves crack routing, substantial crack preparation, and higher quality sealant material. Crack sealing is appropriate for cracks between 1/4 inch and 3/4 inch wide. Use on longitudinal or transverse cracks with little or no secondary cracking or raveling at the crack face. Proper crack preparation and cleaning are essential to optimal performance. Saw or rout cracks to a minimum 3/8 inch width and a depth of 1/2 inch. The width and depth may be adjusted depending on the sealant to be used. Clean cracks of existing joint filler material, vegetation, dirt, or other foreign material. See SUDAS Specifications Section 7040, 3.06. Service life is from 2 to 8 years.

3. **Full/Partial Depth Patches:** Patches restore a pavement's structural integrity and improve its ride. Partial depth patches address distress in the upper one-third of the pavement slab. Slab removal may be accomplished by sawing and jackhammer or by milling. Minimum partial patch depth is 2 inches and maximum depth is 1/2 of the slab thickness. Prior to placement of patch material, clean partial depth patch area and ensure it is dry. Cover entire patch area with tack coat. Lifts should not exceed 3 inches in thickness with the top lift 2 inches or less. Ensure the final compacted surface is level with or not more than 1/8 inch above the surrounding pavement. Full depth patches will address various types of more structural distress, such as broken down thermal cracks. Apply tack coat to all vertical edges. Maximum lift thickness is 3 inches with the top lift being 2 inches or less. Compact intermediate lifts with a roller or vibratory compactor, depending on patch size. Compact final lift with steel-wheeled finish roller. Ensure final compacted surface is level with or not more than 1/8 inch above the surrounding pavement. See SUDAS Specifications Section 7040, 3.02 and 3.03. Patches are often completed in advance of a surface treatment. Service life is from 3 to 15 years.

4. **Fog Seals:** Fog seals are applications of diluted emulsion without a cover aggregate and are used to seal the pavement, inhibit raveling, and slightly enrich hardened or oxidized asphalt. Application rates vary from 0.05 to 0.10 gallons per square yard. If necessary, vegetation control should be completed in advance of the treatment. Ensure pavement is clean and dry prior to application. See Section 5K-4 for additional information. Fog seals can have a negative effect on friction and stripping in susceptible asphalts. Service life is from 1 to 3 years.

5. **Slurry Seals:** Slurry seals are effective at sealing low-severity cracks, waterproofing the pavement, and restoring friction. Slurry seals also address raveling, oxidation, and hardening of asphalt. They are a mixture of crushed, well-graded aggregate, a mineral filler, and asphalt emulsion that is spread across the full width of the pavement or it can be used as a strip treatment for low areas and cracks. Thickness is generally less than 1/2 inch. The slurry is basically placed one aggregate layer thick. Allow a minimum of 7 days cure time before applying permanent pavement markings. See Section 5K-4 and SUDAS Specifications Section 7070. Service life is 3 to 6 years.

6. **Microsurfacing:** Microsurfacing corrects or inhibits raveling and oxidation of the pavement, improves surface friction, reduces moisture infiltration, addresses low to medium severity bleeding, and can be used to fill surface irregularities and ruts up to 1 1/4 inch deep. Microsurfacing materials are similar to slurry seals except that microsurfacing uses latex modified asphalts versus an emulsified asphalt. Application of the microsurfacing is by specialized equipment using an augured screed. Microsurfacing typically breaks within a few

minutes of placement and can carry traffic after about an hour. See Section 5K-4. Service life is 4 to 7 years.

7. **Bituminous Seals Coats:** Seal coats, also sometimes known as chip seals, are effective at improving surface friction, inhibiting raveling, correcting minor roughness and bleeding, and sealing the pavement surface. Bituminous seal coats are also used to address longitudinal, transverse, and block cracking, as well as sealing medium severity fatigue cracks. Seal coats can be applied in multiple layers to address more serious problems. Asphalt emulsion is applied directly to the pavement surface and is followed by the application of aggregate chips that are immediately rolled to embed them into the emulsion. Application rates depend upon the aggregate gradation and maximum size. Loose chips may be a problem on higher speed roadways. Fog seals may be used in conjunction with seal coats to provide a greater degree of binding for the aggregates. See Section 5K-4 and SUDAS Specifications Section 7060. Single layer service life is 4 to 6 years.

8. **Milling:** Milling is used to reduce pavement irregularities and to produce a uniform surface. Milling should be considered if rutting is at a level of 1/4 inch or more. Milling is used in conjunction with other surface treatments, such as slurry seals and microsurfacing in addition to thin asphalt overlays, and is not suggested to be used as a final stand-alone treatment. It can be used to restore proper grades and pavement cross-slopes. For best results, the milling depth should match the lift thickness of the exiting pavement. See Section 5K-4 and SUDAS Specifications Section 7040, 3.05.

9. **Thin Overlays:** Thin overlays are placed in a single lift less than 1 1/2 inches thick. The overlay is expected to improve rideability, surface friction, profile, crown, and cross slope. In addition, specific distress types of low severity cracking, raveling, roughness, low severity bleeding, and low severity block cracking are improved. Thin overlays dissipate heat rapidly and rely on timely compaction to be successful. Dense-graded, open-graded, and stone-matrix mixes may be used. See SUDAS Specifications Section 7020. Service life is 7 to 10 years.

C. Rigid Pavement Treatment Types

Several preventative maintenance treatment types are available to address pavement distresses in PCC pavements. These include:
- Crack sealing
- Joint resealing
- Partial depth patches
- Full depth patches
- Dowel bar retrofit
- Diamond grinding
- Pavement undersealing/stabilization
- Pavement slab jacking
- Concrete overlays

These are the traditional preventative maintenance treatment types. Other less frequently used treatments are available to address specific distress needs.

1. **Crack Sealing:** Crack sealing is accomplished to reduce moisture intrusion and retard the rate of deterioration of the cracks. It is accomplished by thorough preparation and placement of high quality materials. It is used on random transverse and longitudinal cracks of low to medium severity where the crack width is less than 1/2 inch. Proper preparation of the crack and placement of the sealing material are critical for attainment of the expected 4 to 8 year service

life. The sealant material is critical to the success of the operation. Thermoplastic (rubberized asphalt) and thermosetting (silicone) sealants are the usual materials. The crack should be routed to 3/8 inch wide and 1/2 inch deep. The crack should be thoroughly cleaned and dried prior to application of the sealant. Refacing the sides of the crack with sandblasting is recommended. See SUDAS Specifications Section 7040, 3.06.

2. **Joint Resealing:** Joint resealing is important to minimize moisture in the joint and the subgrade/subbase, in addition to minimizing the intrusion of incompressible materials into the joint. Proper resealing of joints will reduce faulting, pumping, and spalling. Removal of the old sealant material and cleaning of the joint prior to resealing are critical. Removal of the old joint material can be accomplished by using a rectangular joint plow, diamond saw, or high-pressure water blast. Following refacing of the joint with a diamond bladed saw, the joint should be cleaned with high pressure air or water. Immediately prior to sealant application, the joint should be blown again with high pressure air to remove any sand, dust, or other incompressible that may remain in the joint. The joint must be dry and clean as joint sealant material is applied. See SUDAS Specifications Section 7040, 3.06. Service life is 4 to 8 years.

3. **Partial Depth Patches:** Partial depth patches are used to address spalling and surface scaling, as well as other problems in the top one-third of the pavement slab. Repair materials are selected based on available curing time, ambient temperature, size and depth of the repair, and cost. The materials are generally classified as cementitious, polymers, or bituminous. Rapid cure and high strength proprietary products are also available. It is critical to identify the limits of the weakened concrete so the patch can connect to sound concrete. The actual extent of the deterioration is often greater than what is visible at the surface. The removal area should extend a minimum of 3 inches beyond the deteriorated area in all directions. The patch area can be prepared by chipping with a lightweight jackhammer, milling with a carbon tipped milling machine, and sawing the edges of the patch and removal with a lightweight jackhammer. The patch area should be square or rectangular in shape and in line with existing joint patterns. The repair area must be swept, sandblasted, and air blasted to ensure a clean, dry patch area. Sandblasting is very effective at removing any dirt, oil, thin layers of unsound concrete, and laitance. Bonding agents are generally required for the patch materials. Sand-cements grouts consisting of one part sand and one part Type III cement with sufficient water to create a thick, creamy consistency have proven successful. Epoxy bonding agents can also be used with PCC and proprietary patching materials. Compressible joint materials must be used against the adjoining slab or to extend an existing joint through the patch area. The compressible material should extend 1 inch below and 3 inches beyond the repair boundaries. It may be possible to saw the joint through the patch, but timing is very critical. Since partial depth patches have large surface areas compared to their volume, it is very important to apply a curing compound as soon as the water has evaporated from the surface. The curing compound should be applied at 1.5 to 2 times the normal rate. The final step is resealing of the joint. See SUDAS Specifications Section 7040, 3.03. Service life of a well done partial depth patch is 5 to 15 years.

4. **Full Depth Patches:** Typical PCC pavement distresses that can be addressed by full depth repairs include transverse cracking, corner breaks, deteriorated joints, and blowups. Full depth repairs are an effective means for restoring the rideability and structural integrity of deteriorated PCC pavements. Long lasting full depth repairs are dependent upon selecting appropriate locations, effective load transfer design, and correct construction procedures, including finishing, texturing, and curing the patch. If the pavement exhibits a materials related deficiency, such as D-cracking, the service life of the patch will be short. Sizing the patch is critical to its success. Distressed areas should be identified and marked. Extent of the patch area may have to be adjusted if a period of time passes between initial identification and actual work activity. It may be necessary to do coring and deflection studies to identify the extent of deterioration below the slab surface. Full depth patches should be a minimum of 6 feet long and a full lane wide. All

joints through or adjacent to full depth patches must be re-established. Connect patches to make one large patch if the patches are 8 to 10 feet from each other in a single lane. The load transfer technique used in the patch should match the load transfer technique in the existing slab. Full depth repairs could be used in conjunction with diamond grinding to correct any roughness problems. See SUDAS Specifications Section 7040, 3.02. Service life is expected to be from 10 to 15 years.

5. **Dowel Bar Retrofit:** Dowel bar retrofit (DBR) is a method of load transfer restoration. It is used on non-doweled plain jointed concrete pavements. A successful dowel bar retrofit project will enhance pavement performance by reducing pumping, faulting, and corner breaks. Pavements with structurally adequate slab thickness, but exhibiting significant loss of load transfer due to poor aggregate interlock or base/subbase/subgrade erosion, are good candidates for DBR. It will also retard deterioration of transverse joints and cracks. Typical design includes three or four dowels inserted into the pavement at joints in each wheel path. The size of the dowel bar varies from 1 inch to 1 1/2 inches in diameter according to the slab thickness. See SUDAS Figure 7010.101. The slots are generally 3 feet long, centered on the joint or crack. The slot must be long enough to allow the dowel to lie flat in the slot without hitting the curve of the saw cut. The width of the slot should be 2.5 inches and the depth sufficient to position the center of the dowel at the mid-depth of the slab. The slot must be parallel to the centerline of the pavement slab so the dowels do not lock up pavement movements. The dowel assembly will have end caps to facilitate movement and a compressible insert to form the joint across the slot. The slot filler materials are the critical element to a successful installation. Desirable properties include little or no shrinkage, similar coefficient of thermal expansion as the existing concrete, good bond strength, and the ability to gain strength rapidly. Concrete with Type III cement, sand, and 3/8 inch maximum sized aggregate can be used or there are proprietary products available. Dowel bar retrofit projects often include following up with diamond grinding. All transverse joints should be re-established by sawing over the joint and through the fill board. The joint should then be prepared and sealed. Dowel bar retrofit projects will allow the original service life of the pavement to be restored.

6. **Diamond Grinding:** Diamond grinding is the removal of a thin layer of pavement surface using closely spaced diamond saw blades. It is used to improve ride quality by eliminating joint and crack faulting. In addition, surface friction, transverse cross slope, and tire/pavement noise are improved. It does not address structural problems or material related distress. Structural problems, such as pumping, corner breaks, and working transverse cracks, must be addressed before grinding. If joint/crack faulting exceeds 1/4 inch, the project may not be a candidate for diamond grinding. The blade spacing and width of groove are dependent on the hardness of the aggregate. As the aggregates get softer, the width of the land area and groove get larger. The depth of cut should be set so that 95% of the area is ground. The surface distresses will redevelop if the root cause of the distress is not corrected prior to diamond grinding. Thus, it may be necessary to complete full and partial depth patches, load transfer restoration, and slab stabilization prior to grinding. See SUDAS Specifications Section 7040, 3.04. Service life varies from 5 to 15 years, depending on the hardness of the aggregates and the level of structural distress correction completed prior to grinding.

7. **Pavement Undersealing/Stabilization:** Slab stabilization is pressure insertion of a flowable material to restore support beneath PCC slabs. It fills existing voids but does not lift the slab. Pavement stabilization restores pavement support, reduces pavement deflections, and reduces progression of pumping, faulting, and corner breaks. Slab stabilization must be completed prior to significant pavement damage. The main issue with slab stabilization is identifying where the voids are located and the extent of the voids. Distress surveys and deflection testing are necessary. Deflections may be measured using a FWD or by using a loaded truck with gauges placed at the corners of the slab. Other methods, such as ground penetrating radar or

thermography, are also available. Pozzolan-cement grout and polyurethane are the most common materials used for slab stabilization. Other proprietary products are available. It is important to only apply the material at locations where voids exist. If it is placed in areas without voids, the material can induce pressure points and actually increase the pavement deterioration. Once the area of the void is determined, the grout insertion holes can be drilled. Holes should be placed as far as possible from cracks and joints. Holes should be placed close enough to achieve flow from one insertion hole to another. Service life is from 5 to 10 years, depending on the level of truck traffic.

8. **Pavement Slab Jacking:** Slab jacking consists of the pressure insertion of a grout or polyurethane material beneath the PCC slab as a means of raising the slab to a smoother profile. Slab jacking is normally used to correct localized settlement areas, such as over culverts or at bridge approaches. It should not be used to correct faulted joints. Grout insertion holes should be a minimum of 12 inches from a transverse joint or the edge of the slab. Holes should be spaced 6 feet or less center-to-center. It is critical to monitor the amount of lift performed at each location. The slab should not be lifted more than 1/4 inch at a time so that excessive stresses are prevented and slab cracking minimized. Uniform positioning of the grout holes is also important. Work should start from the lowest point of the section being raised and proceed out to the edges of the settled area in a repeating pattern. Materials for slab jacking are typically stiffer than those used for slab stabilization. Cement grout and polyurethanes are typically used.

9. **Overlay:** Concrete overlays exist for all types of pavements, including concrete, asphalt, and composite. Thickness for preservation projects are generally between 3 to 4 inches. Similar to other concrete pavements, overlays require uniform support and effective management of movement. The overlay type can be bonded or unbonded. Bonded overlays are used to eliminate surface distresses when the existing pavement is in good structural conditions. Bonded overlays utilize the existing pavement as an integral part of the new monolithic system and thus thorough surface preparation is critical. Unbonded overlays are essentially a new pavement over a stabilized base (the old pavement). A bond breaker, such as a thin asphalt layer or a layer of non-woven geotextile, is needed between the existing pavement and the overlay. Typically, overlays are constructed using standard concrete mixes and standard construction techniques. Fibers may be added to the concrete mix for additional strength. Joints in bonded concrete overlays must match those in the existing pavement. Service life of concrete overlays is 15 to 20 years. Visit the National Concrete Pavement Technology Center's website (www.cptechcenter.org) for more publications on concrete overlays.

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Thin Maintenance Surfaces

A. General

Seal coats, slurry seals, microsurfacing, and fog seals are termed thin maintenance surfaces or TMS. These thin maintenance surfaces can be a cost effective approach to maintaining flexible pavements. Studies have shown that agencies can maintain a city street or county road network in better condition at lower costs through the use of TMS. Project selection, treatment selection, and timing are critical to the use of TMS.

Since TMS do not involve increasing the structural carrying capacity of a street, it is vitally important to apply the appropriate treatment prior to the start of pavement deterioration. Pavement condition, traffic volumes, materials availability, roadway classification, and local preference must be evaluated before determining the type of TMS to use. General uses for TMS are noted in the following table:

Criteria	Seal Coat	Slurry Seal	Microsurfacing
Traffic Volume:			
Low (< 2,000 vpd)	Recommended	Recommended	Recommended
Medium (2,000 to 5,000 vpd)	Marginal	Marginal	Recommended
High (> 5,000 vpd)	Not Recommended	Not Recommended	Recommended
Bleeding	Recommended	Recommended	Recommended
Rutting	Not Recommended	Recommended	Recommended
Raveling	Recommended	Recommended	Recommended
Cracking			
Slight	Recommended	Recommended	Recommended
Moderate	Recommended	Not Recommended	Not Recommended
Low Friction	May improve	May Improve	May Improve
Snowplow Damage	Most susceptible	Moderately susceptible	Least susceptible

Source: Jahren, 2003

Design of these TMS treatments must take into account the type of pavement distress that is being addressed with the proposed project. It may be necessary to complete crack filling, patching, or other maintenance activities prior to implementing the TMS.

B. Seal Coat

A seal coat is a single layer of asphalt binder that is covered by embedded aggregate with its primary purpose to seal fine cracks in the underlying pavement and retard water intrusion into the pavement and subgrade/subbase. The aggregate protects the asphalt binder layer and provides macrotexture for improved skid resistance. Seal coating is also a cost effective way to address bleeding and raveling. Most often, the asphalt binder is an emulsion. Cutback asphalts may be used as well. Emulsified asphalt is a mixture of liquid asphalt and water. A cutback is a mixture of liquid asphalt and a distillate, such as kerosene or fuel oil. The aggregates are typically less than 1/2 inch in size.

One of the most critical factors in the design is to determine the quantities of asphalt binder and aggregate. The goal should be to have the single layer of stone 70% into the asphalt binder layer with little or no stones to clean up. In order to attain that goal, the designer must take into account the traffic volume; the absorption of the binder into the cover aggregate; the texture of the existing pavement; and size, shape, and gradation of the aggregate. Seal coat projects have an expected life span of 4 to 6 years.

Seal coating is recommended for low and medium volume roadways with low speeds due to the increased chance for insurance claims for vehicle damage from the loose rock as traffic volumes and speed increases. In addition, the impact to the public is compounded on high volume roadways due to the time the facility is out of service, generally 24 hours. As traffic volumes increase, it becomes more critical to include very high quality, durable aggregates in the mix design.

Selection of the asphalt binder is important to the success of the project. Although cutback asphalts can be used, their use has rapidly declined over the years due to the costly and harmful solvents used. Typically, asphalt emulsions are used. They are made up of asphalt cement, water, and an emulsifying agent (surfactant). The asphalt cement is typically in the same range as is used for hot mix production and makes up about 2/3 of the volume of the binder. Water provides the medium to keep the asphalt in suspension. The surfactant (usually soap) causes the asphalt particles to form tiny droplets that remain in suspension in the water, and it determines the electrical charge of the emulsion. It is important that the emulsion and the aggregate have opposite electrical charges in order to maximize the bond between the emulsion and the aggregate. Since most aggregates have a negative charge, emulsions such as CRS-2P with a positive (cationic) charge are used.

Cover aggregate should be clean and dust free to maximize adherence. A uniform gradation of hard, durable aggregate will increase the resistance to impact from traffic and snowplows. Aggregate application needs to follow binder application very closely. The cover aggregate should be applied so it is only one layer thick. Excess aggregate increases the chance for dislodging properly embedded aggregate during the cleanup operation, as well as increasing the potential for vehicle damage. The aggregate may be gravel, crushed stone, or a mixture. Cubical shaped aggregate is preferable to flat aggregate. Flat and elongated aggregates can be susceptible to bleeding due to traffic causing the flat chips to lie on their flattest side. If flat aggregate is used and the binder is applied too thick, the pavement will bleed; if it is too thin, the pavement will ravel. Angular aggregate is preferable to round aggregate because angular aggregate chips tend to lock together.

One of the problems with seal coats is the generation of dust from the aggregate. One way to address the dust problem is to pre-coat the aggregate. Pre-coating involves applying either a film of paving grade asphalt or a specially formulated pre-coating bitumen to the aggregate. The use of pre-coated aggregate improves aggregate bonding properties, as well as reducing dust. It also shortens the required curing time and vehicle damage from loose aggregate. Fog seals may also be used to address dust problems and to cover the "gravel road" appearance of seal coat. Fog seals are generally a 50-50 mix of emulsion and water. It is important to recognize that skid resistance may be compromised with the use of fog seals.

Many design tools are available. One of the most often used is the Minnesota Seal Coat Handbook. It can be found at: http://www.lrrb.org/media/reports/200634.pdf. Another source is the Thin Maintenance Surfaces Manual developed by the Institute for Transportation at Iowa State University. It can be found at: http://www.intrans.iastate.edu/publications/_documents/handbooks-manuals/thin-maintenance-surfaces/thin_maint_surf.pdf.

C. Slurry Seal

Slurry seal is a mixture of emulsified asphalt oil, aggregates, water, and additives. It is pre-mixed and placed as a slurry onto the pavement. Slurry seals are commonly recommended for use on low and medium volume roadways. They are used to treat low to medium levels of raveling, oxidation, and rutting. Applications of slurry seals will improve skid resistance. Slurry seals are often described as the most economical, versatile TMS for low to medium volume roadways.

Aggregates commonly used for slurry seal applications consist of a combination of crushed stone and additives, such as Portland cement, lime, and aluminum sulfate. The additives are used to modify curing time. Aluminum sulfate retards curing time and Portland cement and lime shortens curing time. The aggregate gradations are described as fine and coarse. Coarse gradations have greater stability and are preferred for rut filling and scratch (bottom) courses. The additives make up less than 2% of the mixture and the aggregates are about 75% to 80%. Higher quality aggregates such as granite and quartzite will provide for a more durable application of slurry seal. Smaller aggregate gradations are used for maximum crack sealing, while coarse gradations are used when the project goal is to improve skid resistance.

The asphalt binder is an asphalt emulsion. The usual grades are CSS-1h or SS-1h, which are cationic and anionic slow setting emulsions, respectively. The emulsion is formulated with relatively stiff base asphalt (the suffix h = hard) for use in warm climates. The emulsion will make up about 7% to 14% of the mixture. Water is the remaining element of the mixture.

Temperature and humidity are critical to the cure time of the slurry seal. Temperatures must be 50°F and rising before application can begin. Slurry seals should not be placed at night. Slurry seals can be used to address slight rutting distress as well as to fill open joints by a strip treatment.

Mix design is generally completed by a laboratory certified by the International Slurry Seal Association (ISSA). Compatibility of the emulsion, aggregates, water mineral filler additives, and any other elements needs to be checked using materials that will be incorporated into the project.

D. Microsurfacing

Microsurfacing is a mixture of polymer-modified asphalt emulsion, graded aggregates, mineral filler, water, and other additives. It is mixed in a pug mill and evenly spread over the pavement. It is used to address oxidation, raveling, rutting, and skid resistance problems. Microsurfacing can be applied to higher speed, higher volume roadways than slurry seals and it can be used on both asphalt and concrete roadways. It can be placed at a thickness that is two or three times the size of the largest aggregate; however, trying to lay it too thick may result in rippling, displacement, and segregation. Multiple lifts are used if thicker application rates are needed.

Microsurfacing differs from slurry sealing in four main areas. They are:

- Microsurfacing can be placed in layers thicker than a single aggregate size.
- Microsurfacing always contains polymer modifiers.
- It cures through a chemical reaction versus evaporation.
- Higher quality aggregates are used.

Microsurfacing can also be accomplished at night to potentially minimize traffic impacts. If specifically designed for rapid opening, a microsurfacing project may be returned to traffic in as little as 1 hour.

Design of the microsurfacing mix is generally included in the contract to be completed by the contractor and/or the emulsion supplier. It is critical that all elements of the mix be compatible with each other in order to develop a mix that will address the project conditions. Publication A143 from the ISSA is used as a guide for development of the mix design.

Generally, a single emulsion that works for the climatic conditions and traffic volumes is selected. Cationic emulsions, such as CSS-1h, are typically used, but CQS-1h can be used if it is important to minimize traffic delays due to construction activities. Polymers are added to the emulsions to reduce thermal susceptibility, improve thermal crack resistance, and improve aggregate retention.

Aggregates must be high quality with fractured faces to form higher bonds with the emulsions. Freshly crushed aggregates, as opposed to weathered aggregates, have a higher electrical charge and improve the bond as well. Washing the aggregate to remove clay, silt, and dust is important to ensure proper cohesion.

Mineral fillers are used to aid in the mixing process and spreading of the mixture. Typical mineral fillers are Portland cement, hydrated lime, fly ash, kiln dust, limestone dust, and baghouse fines.

Additives may also be included in the mix. Aluminum sulfate, aluminum chloride, and borax are typically used. These additives allow the contractor to control breaking and curing times.

Properly designed and applied microsurfacing projects have a service life of up to 7 years.

E. Fog Seal

A fog seal is an application of diluted asphalt emulsion without a cover aggregate. It is used to seal and enrich the asphalt surface, seal minor cracks, and provide shoulder delineation. Fog seals are used on low and high volume roads. Its primary use on high volume roads has been to prevent raveling of open-graded friction courses in addition to delineating between the mainline and shoulder.

A fog seal is designed to coat, protect, and/or rejuvenate the existing asphalt binder. Fog seal use on mainline pavements should generally be restricted to only those locations having an open surface texture. This includes chip seals, heavily aged dense graded pavements, and open graded pavements. The fog seal emulsion must fill the voids in the surface of the pavement. A slow setting emulsion such as CSS-1 or SS-1, diluted to one part asphalt emulsion to four parts water is used. Emulsions that are not correctly diluted may not properly penetrate the surface voids and a slippery surface may be the result.

Before placing the fog seal, the pavement must be dry and clean and all pavement repairs accomplished. The diluted asphalt emulsion should be applied at 0.12 gallons per square yard. Success of application is impacted by temperature so summertime application is required. Generally, no application past August 31 is allowed. Pavement and air temperatures must be greater than 60°F to apply the fog seal.

The service life of a fog seal is fairly short, ranging from 1 to 2 years.

Permeable Interlocking Pavers

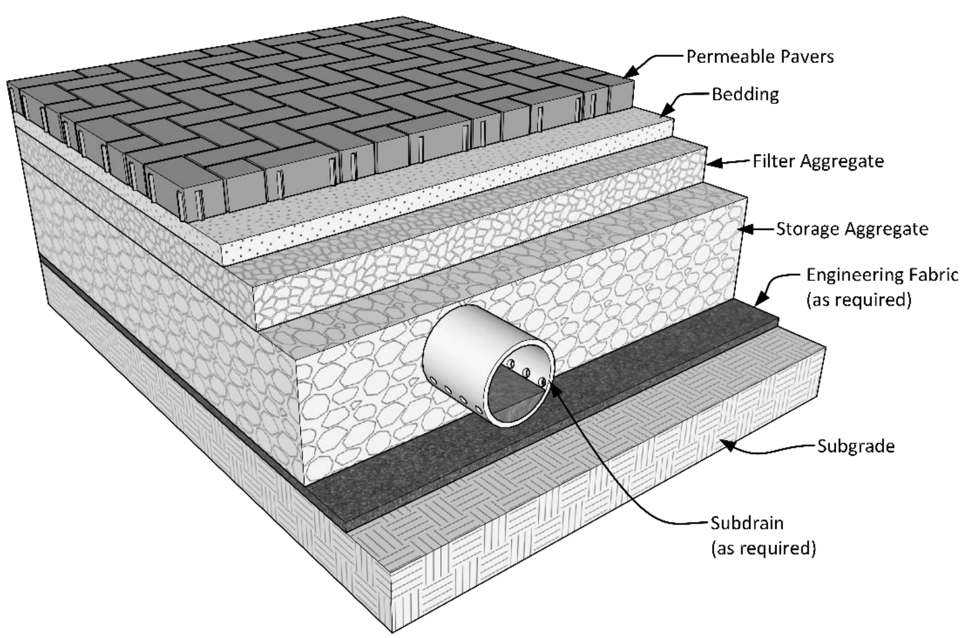
A. General

Permeable pavements are designed to infiltrate runoff, whereas runoff sheds off the surface of conventional pavements. In permeable pavements, runoff passes through the surface and is stored in the aggregate base. In pervious soils, the runoff infiltrates the soil; in less permeable soils, a subdrain system is placed to slowly discharge the runoff. Runoff volume reduction is achieved as the water is infiltrated into the underlying soils. The peak runoff rate is reduced due to the stormwater being stored in the aggregate subbase and slowly released to the downstream piping systems. Traditionally, at a minimum, the depth of the aggregate subbase is designed to meet the storage needs for the Water Quality Volume (WQv), which is 1.25 inches of rainfall in Iowa.

Permeable pavements can dramatically reduce the surface runoff from most rainfall events by disconnecting and distributing runoff through filtration and detention. The use of permeable pavements can result in stormwater runoff conditions that approximate the predevelopment site conditions for the immediate area covered by the pavers.

The design of permeable interlocking pavements (PIP) involves both structural and hydrological analyses. Figure 5L-1.01 illustrates a typical cross-section of a PIP. These two design elements are typically not interconnected and in reality are often in conflict. This is particularly the case with the subgrade treatment and volume of aggregate subbase. Structural design requires a compacted subgrade and the hydrologic design desires an uncompacted subgrade to allow as much infiltration as possible. In most instances, the hydrologic requirements for filter and storage aggregate exceed the structural needs for the unbound aggregate subbase.

Figure 5L-1.01: Permeable Interlocking Paver Cross-section



PIP are used for low speed/low volume streets, alleys, parking lots, and driveways. The design and operating speed of the facility should be below 35 mph. Permeable paver projects should only be developed in areas dominated by impermeable surfaces or surfaces that are fully vegetated so that sediment runoff is minimized and life of the pavement is maximized. PIP are capable of handling truck traffic.

The following elements should be reviewed prior to undertaking a detailed design process:

- Underlying geology and soils
- NRCS hydrologic soils groups
- History of fill, disturbance, or compaction of underlying soils
- Current drainage patterns and volume of runoff
- Local and downstream drainage facilities
- Distances to potable water supply wells
- Elevation of the static water table
- Traffic volumes, including percent trucks

Because water is stored in the subbase rock, it may be necessary to protect structures that are adjacent to the permeable paver project by sealing the foundation walls. The PIP must be a minimum of 100 feet from a municipal water supply well.

There are two types of permeable interlocking pavers. One type is concrete pavers that are 3 1/8 inches thick; the other type is clay brick pavers that are at least 2 5/8 inches thick. The concrete pavers must comply with ASTM C 936. There are two ASTM standards for brick pavers, depending on the traffic loading. ASTM C 902 is for pedestrian and light vehicular traffic locations. ASTM C 1272 is for heavier vehicular traffic and will be the type listed in the SUDAS Specifications. The clay pavers should be 2 3/4 inches thick, Type F brick for PX applications according to ASTM C 1272.

B. Structural Design

The design procedure for permeable interlocking pavers is the same as for flexible pavements. Research has shown that the load distribution and failure modes of PIP are similar to other flexible pavements. Because the designs are the same as for flexible pavements, the AASHTO *Guide for Design of Pavement Structures* (AASHTO, 1993) can be used. The paver used in design for concrete pavers is a 3 1/8 inch thick paver with a minimum 1 inch bedding layer. The structural coefficient is 0.44 per inch. This provides a structural number of 1.82. The clay brick paver is 2 3/4 inches thick, which has a corresponding structural number of 1.21. The remaining structural support comes from the aggregate layers and the soil subgrade.

The American Society of Civil Engineers has developed a design standard called *Structural Design of Interlocking Concrete Pavement for Municipal Streets and Roadways* (ASCE/T&DI/ICPI 58-10). The structural design for clay brick pavers is the same as for concrete pavers. The engineer will need to determine or select the following:

- Design traffic loading (ESALS)
- Design life (40 years minimum)
- Design reliability (usually 75% to 80%)
- Overall standard deviation (0.45)
- Required structural number to meet traffic loading
- Initial serviceability (flexible pavements = 4.2)
- Terminal serviceability (local streets = 2.0)
- Subgrade resilient modulus based on saturated soil characteristics, including seasonal variability

- Drainage conditions

Once these elements are determined, the design thickness of the unbound aggregate subbase can be determined. The ASCE design standard has tables showing thickness of the layers that were developed using the AASHTO 1993 Guide. Thickness is selected based on the ESALS, the soil category, and the drainage.

Three types of interlock are critical to achieve: vertical, rotational, and horizontal. Vertical interlock is achieved by the shear transfer of loads to surrounding pavers through the material in the joints. Rotational interlock is maintained by the pavers being of sufficient thickness and aspect ratio (3:1 minimum), being placed close together, and restrained by a curb from lateral forces of vehicle tires. Rotational interlock can be further enhanced if there is a slight crown to the pavement cross-section. Horizontal interlock is primarily achieved through the use of laying patterns that disperse forces from braking, turning, and accelerating vehicles. Herringbone patterns, either 45° or 90°, are the most effective patterns for maintaining interlock. A string or soldier course should be used at the interface between the pavers and the edge restraint.

A PCC edge restraint is typically used for street and alley projects. The edge restraint may be a standard curb and gutter section, a vertical curb section, or a narrow concrete slab, and should be placed on the subbase aggregates.

After placement, the pavers are compacted with a high frequency plate compactor, which forces the joint material into the joints and begins compaction of the paver into the bedding layer. The pavement is transformed from a loose collection of pavers into an interlocked system capable of spreading vertical loads horizontally through the shear forces in the joints.

One of the direct conflicts with the hydrologic design of PIP is the compaction of the subgrade soils. The structural design calls for subgrades compacted to 95% Modified Proctor Density according to AASHTO T 180. The effective compaction depth should be 12 inches minimum. This compaction requirement will prevent efficient infiltration of water through the subgrade and thus will likely necessitate a piping design to handle the stormwater that accumulates in the storage aggregate (unbound subbase).

The engineer should provide a geotextile between the subgrade and the storage aggregate (subbase) as a means of preventing mixing of the materials. The geotextile should comply with Iowa DOT Section 4196 for subsurface drainage.

C. Hydrologic Design

The design process follows traditional storm sewer procedures for pavements. The initial step in the hydrologic design is the determination of the design storm event. Some agencies may establish the storm return period and the rainfall intensity. Information on intensity-duration-frequency for various return periods can be found in Chapter 2. In addition, the contributing area must be determined. The runoff volume should be determined according to the methods described in Chapter 2 using a design rainfall depth of 1.25 inches as a minimum, unless the jurisdiction has a different policy.

The next step involves establishing the drainage area. The storm event is then applied to the drainage area and the volume of runoff is determined.

The permeability of the subgrade soil is a critical design element. If the subgrade soil permeability is less than 1/2 inch per hour, a subdrain piping network will be needed. Soil compaction to support vehicular traffic will decrease permeability. Good design practice for vehicular traffic loads is to provide a minimum CBR of 5. Thus as the soil permeability is determined it should be assessed at the density required to realize a CBR of 5 under soaked conditions.

To maximize the effectiveness of the PIP, the pavement grade should be as flat as possible, although steeper grades can be used. The general guideline is that the longitudinal grade should be greater than 1% and less than 12%. Three design alternatives exist for the PIP. They are:

- Full infiltration: All of the stormwater runoff from the design storm is infiltrated into the subgrade soils. See Figure 5L-1.02.A.
- Partial exfiltration: Some of the design storm runoff is infiltrated and the remainder is collected in the subdrain system and slowly discharged into the downstream systems. This is accomplished by setting the subdrain pipe above the top of the subgrade. See Figure 5L-1.02.B.
- Full Exfiltration: Soil permeability is limited and thus all of the runoff volume is carried away through the subdrain piping. See Figure 5L-1.02.C.

Designers must also evaluate and provide for larger storm events. One way to provide for the larger storms but still provide for infiltration of the water quality storms is to raise the elevation of the intakes above the pavers so the small storms are infiltrated and the large storms are handled by the intakes and pipe network.

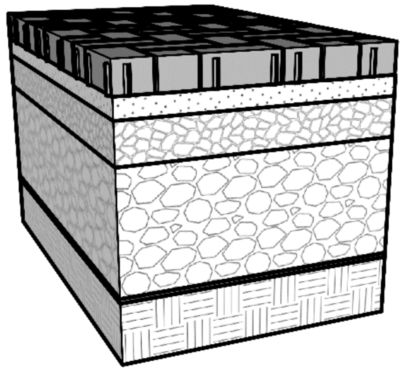
Once the volume of runoff and the soil permeability are known, the thickness of the storage aggregate layer (Iowa DOT Gradation No. 13/ASTM Gradation No. 2) can be determined. The void space (volume of voids/volume of aggregate) for Iowa DOT Gradation No. 13 is 40%. A 40% void space provides 0.4 cubic feet of stormwater storage for each cubic foot of aggregate. Thus, the volume of the storage aggregate will need to be 2.5 times the volume of water to be stored.

Due to the need to compact the subgrade soil to handle vehicles, it is very likely that subdrains will be needed to discharge at least a portion of the runoff. The elevation and sizing of the subdrains should be set to provide for full discharge of the design storm within 72 hours either through infiltration into the subgrade soil or through subdrain pipe discharge.

In order to prevent absorption of the bedding stone into the storage aggregate layer, a layer of filter aggregate (Iowa DOT Gradation No. 3/ ASTM Gradation No. 57) is needed. This layer is typically 4 inches thick. The bedding aggregate (Iowa DOT Gradation No. 29/ASTM Gradation No. 8) is then placed 2 inches thick, compacted, and leveled. Fine graded sand should not be used as the bedding and for filling of voids due to the increased clogging potential.

The pavers are placed, additional bedding stone is added to fill the voids in between the pavers, the area is swept, and finally the pavers are compacted. Sweeping prior to compaction is important to prevent stones on the surface from marring or cracking the pavers. That process may need to be repeated to entirely fill the voids. The final step is to sweep and remove any excess void filler stone.

Figure 5L-1.02: Permeable Interlocking Paver Design Alternatives



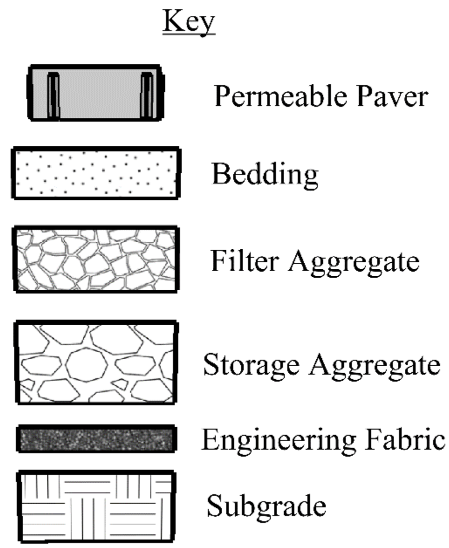


Figure 2.A
Full Infiltration
(Requires soils with infiltration rates greater than 0.5" / hour)

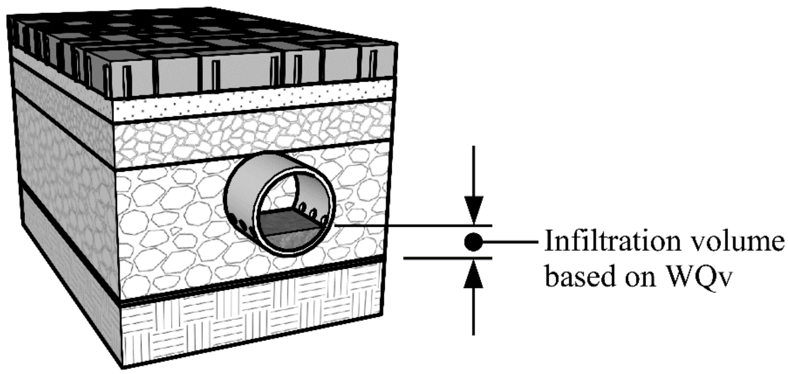


Figure 2.B
Partial Infiltration

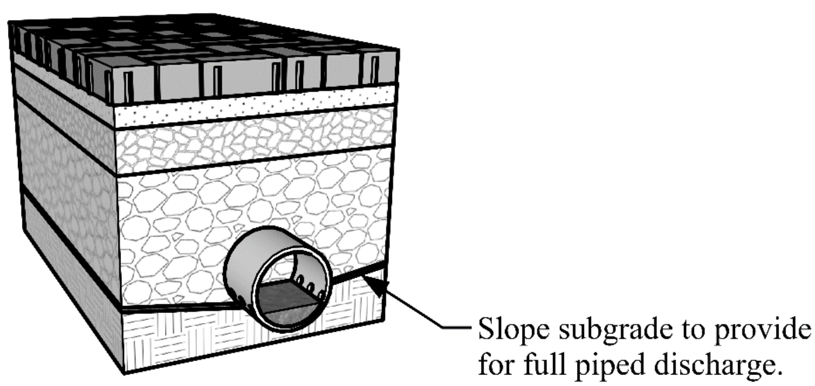


Figure 2.C
Full Exfiltration

D. Construction Elements

Monitoring and controlling the construction activities of a permeable interlocking paver project are critical to the long-term performance of the permeable pavement. Preventing and diverting sediment from entering the aggregates and pavement during construction must be of the highest priority. Aggregate stockpiles must be isolated to prevent contamination by sediment. Erosion and sediment control devices must be placed and maintained throughout the project until vegetation is fully established. All unnecessary vehicle and pedestrian traffic should be restricted once the aggregate placement has initiated. It may be necessary to wash vehicle and equipment tires to prevent tracking dirt and mud onto the aggregate layers.

A test section (approximately 5 feet by 5 feet) should be constructed to provide a basis for construction monitoring. The test section should be placed on the prepared subgrade to illustrate the processes used to place the pavers and illustrate the paver pattern and the edge details.

Restrict all equipment and workers from the paver placement area once the bedding stone has been placed, leveled, and compacted. Pavers may be placed by hand or mechanically. Placement should proceed from one end or side and continue work from the completed placement areas. An important consideration with mechanically placed pavers for large projects is to ensure the wear on the paver molds does not change the size of the pavers and thus impact the ability to correctly place the pavers.

E. Maintenance

As with any pavement, particularly permeable pavements, specific maintenance activities are necessary to achieve the design life of the pavement. PIP can become clogged with sediment that affects its infiltration rate. The rate of sedimentation can depend on the number and type of vehicles using the pavement, as well as the control of erosive soils adjacent to the pavement. The most important element of maintenance is keeping the sediment out of the pavement by vacuum sweeping. Regular vacuum street sweeping will maintain a high infiltration rate and keep out vegetation. Calibration of the vacuum force may be necessary to remove the sediment but minimize removal of the filler material from the joints. Over time, it may be necessary to add additional joint filler material to prevent intrusion by sediment.

Winter maintenance involves plowing snow and applications of de-icing chemicals. Although not required, snowplows can be equipped with rubber edged blades to minimize chipping of the pavers. Use of de-icing chemicals is often not necessary because the PIP remains warmer throughout the winter. Sand should not be used as an abrasive for traction. The sand will clog the filler material in the pavement joints.

Complete Streets

A. Background

Design professionals face an increasingly complex set of competing demands in development and delivery of street projects involving public rights-of-way. Designing a safe facility, completing construction, and installing various traffic control measures are only a part of a much larger picture. Street projects today also need to meet the objectives of regulatory, policy, and community requirements aimed at integrating the roadway into the existing natural and built environments. Among the many factors influencing the planning, design, and operation of today's streets are concerns about minimizing transportation costs; improving public health, creating and maintaining vibrant neighborhoods; accommodating the needs of the young, the physically challenged, as well as an aging population; and adopting greener and more sustainable lifestyles.

In the past, street design was focused on the need to move motor vehicles. The number and width of lanes was determined based on future projected traffic volumes or a set of standards based on the functional classification of the street. The functional classification and the adjacent land use also determined the general operating speed that was to be used for the design. Integration of facilities for pedestrians and bicyclists was not always a high priority. Some observers claim if you do not design for all modes of travel, then you preclude them.

Citizens within some cities are asking agencies to change the way they look at streets and the street function within each community. These agencies are looking to make their streets more "complete." Complete streets are designed and operated to enable safe access to all motorists, pedestrians, bicyclists, and transit users, regardless of age and ability. According to the National Complete Streets Coalition, there are in excess of 600 agencies that have adopted some form of a complete streets policy. Nineteen Iowa agencies, both small communities and larger cities, have adopted complete streets policies. Many other Iowa communities are looking into the concepts of complete streets. Complete streets also complement the principles of context sensitive design by ensuring that streets are sensitive to the needs of all users for the land use within the area. Proponents of complete streets note that by rethinking the design to include all users, the "balance of power" is altered by indicating that streets have many purposes and are not exclusively for motor vehicle traffic. The objectives of the complete streets philosophy are met by slowing vehicles down and providing better facilities for transit, pedestrians, and bicyclists. It is important to understand that safe and convenient walking and bicycling facilities may look different depending on the context. Appropriate facilities in a rural area will be different from facilities in a dense urban area.

There is no one size fits all design for complete streets. While the ultimate design goal for a complete street is a street that is safe and convenient for all users, every design should take into account a number of factors, some of which may be in conflict with each other. The factors include such elements as:

- Number and types of users - vehicles, trucks, transit buses, pedestrians, bicyclists
- Available right-of-way
- Existing improvements
- Land use
- Available budget
- Parking needs
- Community desires

In larger communities where the traffic volumes are heavy and land use density is greater, all of the above elements may be factors to consider. However, in smaller communities with lower traffic volumes and less dense developments, only a few may be important. The application of complete streets principles is most effective when neighborhoods are compact, complete, and connected to encourage walking and biking comfortable distances to everyday destinations such as work, schools, and retail shops. Past land use practices of large tracts for single use development are less effective in encouraging short walking or biking trips.

Complete streets are designed to respect the context of their location. For example, downtown locations may involve greater emphasis on pedestrians, bicyclists, and transit users than single family neighborhoods. Additionally context includes social and demographic factors that influences who is likely to use the street. For example, low income families and those without their own vehicle have the need for an interconnected pedestrian, bicycle, and transit network serving important destinations in the community.

The U.S. DOT adopted a policy statement regarding bicycle and pedestrian accommodations in March of 2010. It states:

> "The U.S. DOT policy is to incorporate safe and convenient walking and bicycling facilities into transportation projects. Every transportation agency has the responsibility to improve conditions and opportunities for walking and bicycling and to integrate walking and biking into their transportation systems. Because of the numerous individual and community benefits that walking and bicycling provide – including health, safety, environmental, transportation, and quality of life – transportation agencies are encouraged to go beyond minimum standards to provide safe and convenient facilities for these modes."

In addition to the U.S. DOT policy, members from the U.S. House of Representatives and the U.S. Senate have introduced a bill entitled "Safe Streets Act of 2014" that calls for all state DOTs and TMAs/MPOs to adopt a complete streets policy for all federally funded projects.

B. Design Guidance

There are a myriad of ways to address the development of complete streets in terms of a planning function, but there are not specific complete streets design elements identified for engineers to use to develop construction or reconstruction projects. The concept of complete streets goes beyond safety, tying in issues of health, livability, economic development, sustainability, and aesthetics.

Applying flexibility in street design to address the complete streets philosophy requires an understanding of each street's functional basis. It also requires understanding how adding, altering, or eliminating any design element will impact different users. For instance, large radii may make it easier for trucks to navigate the street, but they create wider streets for pedestrians to cross. Designers of complete streets should understand the relationship between each criterion and its impact on the safety and mobility of all users.

Various manuals are available to provide design guidance including:
- AASHTO's A Policy on Geometric Design of Highways and Streets (the Green Book)
- The Manual on Uniform Traffic Control Devices (MUTCD)
- The Highway Capacity Manual (HCM)
- AASHTO Guide for the Development of Bicycle Facilities
- ITE Traffic Engineering Manual
- NFPA Fire Code
- Local design ordinances
- The Access Board's PROWAG

Some elements within these manuals are specific standards and some are guidelines with ranges of acceptable values. The MUTCD has been adopted as law; therefore the standards within it need to be met. In addition, there may be different standards for facilities that are under the Iowa DOT's jurisdiction than those for local control. If federal or state funding is being used to assist in a project's financing, the standards may be different yet. Local jurisdictions utilize the above manuals for design as a means of protection from lawsuits. Thus from a liability standpoint, it is very important that the design guidance meet the standards or fall within the range of acceptable guidelines provided by the above manuals.

As always, functional classification, traffic volumes, and level of service are factors to consider in any street design, and may be the highest priority for certain facilities. Through stakeholder input, it is important to identify the core issues, develop a spectrum of alternatives, and reach a design decision considering the needs of all of the users. The project development process may determine vehicular level of service is not the critical element and improved service for the other travel modes for pedestrians, bicyclists, and transit users is equal or more important.

C. Design Elements

If a complete streets design is contemplated, many elements must be determined during the design process. Traditionally designers have focused on those related to motor vehicles. With a complete streets design, other elements are also addressed. Each of those elements will be discussed and design guidance presented.

1. **Land Use:** The type of adjacent land use provides insight into several factors. For instance, in industrial areas, the expectation is that truck volumes will be higher. Also in commercial/retail areas, there is an expectation that pedestrians, transit, and bicyclists will have a greater impact. In residential land use areas, the street and right-of-way should accommodate pedestrians of all ages and abilities, and shared use of the street by motorists and bicyclists should be expected.

   Land use will influence speed, curb radii, lane width, on-street parking, transit stops, sidewalks, and bicycle facilities.

2. **Functional Classification:** Most jurisdictions classify their streets as a means of identifying how they serve traffic. Streets are generally classified as arterial, collector, or local facilities. Complete streets projects must take into consideration each street classification because it helps determine how the street and network needs to be treated to handle traffic volumes and other conflicts that may arise if design changes are made.

   Street classifications and the functions of each type are explained in detail in Section 5B-1. It is important to note that all jurisdictions, regardless of size have at least one street in each category. That means that in a larger community an arterial street may carry 20,000 vehicles per day, but in a smaller city the volume on their arterial street might be 2,000 vehicles per day. Similar differences exist in the collector classifications. Generally arterial streets are designated because their primary purpose is to move traffic. Collectors serve the traffic mobility function, but also provide access to adjacent property. Local streets are primarily there to serve adjacent property and should not have through traffic. Designs appropriate for low density residential areas are not likely to fit in the downtown commercial areas due to the likelihood of more pedestrians, bicyclists, trucks, and buses.

3. **Speed:** Because of the differences from community to community in functional classifications, a better criteria to use for design is speed. There are two types of speed to consider in design. The first is operating speed and the other is design speed. Operating speed is typically the posted speed limit and the design speed is often set at 5 miles per hour greater as a factor of safety. It is

also permissible to set the design speed and the posted speed the same. The design speed determines various geometric requirements for safe operations at that speed. These include stopping sight distance, passing sight distance, intersection sight distance, and horizontal and vertical curve elements. These standards are from the AASHTO Green Book and are outlined in Tables 5C-1.01 and 5C-1.02 and for liability reasons should be met at all times, especially for new streets. If it is not possible for any design element to meet the geometric standards on existing streets, warning signs and other safety treatments must be used.

It has been past practice to set the design speed at the highest level that will meet the safety and mobility needs of motor vehicles using the street. One of the principles of complete streets provides for slowing vehicles down to improve safety for all users, especially pedestrians and bicyclists. In general, the maximum speed chosen for design should reflect the network needs and the adjacent land use. The speed limit should not be artificially set low to accomplish complete streets objectives if the roadway environment does not create the driver expectation that they should slow down.

The maximum speed for arterial streets should be 45 miles per hour (mph), but only in rural sections or situations where access control is established and free flowing traffic is the normal situation. A maximum of 35 mph is more typical for most arterial streets in urban developed areas.

Collector streets serve both a mobility and property access function and thus the maximum speed is generally 30 mph. In some cases, 35 mph could be used but only when property access is very limited.

Local streets should be designed at 25 mph since their primary function is for property access.

4. **Design Vehicle:** The selection of the design vehicle is an important element in complete streets design. Lane width and curb radii are directly influenced by the design vehicle. It is not always practical to select the largest vehicle that may occasionally use a street as the design vehicle. In contrast, selection of a smaller vehicle if a street is regularly used by larger vehicles can invite serious operational and safety problems for all types of users.

   When selecting a design vehicle, the designer should consider the largest vehicle that will frequently use the street and must be accommodated without encroaching into opposing traffic lanes during turns. It is generally acceptable to have encroachment during turns into multiple same-direction lanes on the receiving street but not opposing lanes. The choice of a design vehicle is particularly important in intersection design where pedestrians, bicyclists, and vehicles routinely share the same space.

   All street designs must meet the minimum standards for fire departments and other emergency vehicle access and must consider the needs of garbage trucks and street cleaning equipment.

5. **Lane Width:** The AASHTO Green Book provides for lane widths from 9 to 12 feet wide. Narrower lanes force drivers to operate their vehicles closer to each other than they would normally desire. The drivers then slow down and potentially stagger themselves so they are not as close. The actual lane widths for any given street are subject to professional engineering judgment as well as applicable design standards and design criteria. The width of traffic lanes sends a specific message about the type of vehicles expected on the street, as well as indicating how fast drivers should travel. With painted lane lines being 4 to 6 inches wide, the actual "feel" to the driver will be about 1 foot narrower than the design lane width. Wider lanes are generally expected on arterial and collector streets due to truck traffic and higher operating speeds. Snow plowing and removal practices must also be considered as lane width decisions are being made,

especially for the curb lane. Narrower curb lane widths may necessitate different handling of snow because no space is available to plow the snow and it may require loading and removing on a more frequent basis.

It is preferred that arterial streets with 3 to 5% trucks or buses or operating speeds of 35 mph or greater have lanes that are 12 feet wide. That is especially important on the outside lane of multi-lane facilities. It is acceptable to have 11 foot wide lanes on arterial streets when speeds are 30 mph or less, but the entire street context, such as the presence of on-street parking, bike lanes, buffer areas, turn lanes, and volume of trucks and buses, needs to be considered before lane widths are chosen.

Collector streets can have 11 foot wide lanes if the number of trucks and buses is low. Collector street speeds should not exceed 35 mph.

Local commercial and industrial streets should be no narrower than 11 feet due to the larger volume of trucks expected with that land use. Local streets can have lane widths down to 10 foot wide in residential areas. For low volume local residential streets, two free flowing lanes are generally not required. This creates a yield situation when two vehicles meet.

The designer should recognize that there is an impact to the capacity of a street as the lanes are narrowed. According to the Highway Capacity Manual, capacity is lowered by 3% if lane widths are narrowed from 12 feet to 11 feet and 7% if lanes are narrowed to 10 feet.

6. **Curb Radii:** The curb radius of intersection corners impacts turning vehicles and pedestrian crossing distances. Larger radii allow larger vehicles, such as trucks and buses, to make turns without encroaching on opposing travel lanes or the sidewalk, but increase the crossing distance for pedestrians and allows smaller vehicles to turn at faster speeds. Shorter curb radii slow turning traffic and create shorter crossing distances, but make it difficult for larger vehicles to safely navigate the intersection. The curb radii that is chosen by the designer should reflect the number of pedestrians, the number of right turns by larger vehicles, length of the pedestrian crossing, and the width of intersecting streets.

   The curb radii must meet the AASHTO Green Book turning templates for the design vehicle selected. The curb radii may be modified if parking lanes and or bike lanes are present. It is acceptable to have encroachment into same-direction lanes on the receiving street. It is not acceptable to design a curb radius that calls for turning vehicles to encroach upon the opposing traffic lanes. The minimum curb radii in all cases should be 15 feet.

7. **Curb Extensions or Bump-outs:** Curb extensions or bump-outs are expansion of the curb line into the adjacent street. They are traditionally found at intersections where on-street parking exists, but may be located mid-block. Bump-outs narrow the street both physically and visually, slow turning vehicles, shorten pedestrian crossing distances, make pedestrians more visible to drivers, and provide space for street furniture. Use of curb extensions does not preclude the necessity to meet the turning radii needs of the selected design vehicle.

8. **Bicycle Facilities:** Bicycle facilities provide opportunities for a range of users and are a fundamental element of complete streets design. In Iowa, bicycles are legally considered a vehicle and thus have legal rights to use any street facility unless specifically prohibited. They also have legal responsibilities to obey all traffic regulations as a vehicle. Bicycle facilities generally are one of the following three types:
   a. **Shared Use Paths:** Separate travel ways for non-motorized uses. Bicycles, pedestrians, skaters, and others use these paths for commuting and recreation. Generally used by less experienced bicyclists.

b. **Shared Lanes:** These are lanes shared by vehicles and bikes without sufficient width or demand for separate bike lanes. They may be marked or unmarked. Low speed, low volume residential streets generally will not have pavement markings. For higher speed facilities, sharrow pavement markings and signage are used to remind drivers of the presence of bicyclists in the travel lane. Placing the sharrow markings between vehicle wheel tracks increases the life of the marking. These types of shares lanes are used more for commuting than recreation.
c. **Bike Lanes:** Dedicated lanes used on higher speed, higher volume streets separated from vehicle lanes or on-street parking spaces by pavement markings. No specific standards for when to use bike lanes exist, but conflicts between bikes and vehicles in shared lanes generally become problematic when vehicular volumes exceed 3,000 to 5,000 ADT and operating speeds are 30 mph or greater. Bicycle lanes should be a minimum of 5 feet wide on curbed pavements and 4 feet wide on rural cross-sections. If possible, a buffer zone of 3 feet should be provided between the bike lane and the on-street parking area to minimize conflicts with bikes and opening vehicle doors. These lanes are generally used by experienced bicyclists for commuting.

Snow and ice control activities impact vehicular lanes and bike lanes differently. Generally plows will leave some snow on the pavement. Vehicles are able to travel through this material but bicyclists may have more difficulty. In addition, the material may refreeze and make bike use more treacherous.

Design information for each bicycle facility type is detailed in Sections 12B -1 through 12B -3. Bicycle parking facilities at destination points will assist in encouraging bike usage.

9. **On-Street Parking:** On-street parking can be an important element for complete street design by calming traffic, providing a buffer for pedestrians if the sidewalk is at the back of curb, in addition to benefiting adjacent retail or residential properties. The width of parallel parking stalls can vary from 7 to 10 feet. Streets with higher traffic volumes and higher speeds should have wider parking spaces or a combination of parking space and buffer zone. Narrower parking spaces can be used if a 3 feet buffer zone is painted between the parking stall and a bike or traffic lane. The buffer zone will minimize exposure of doors opening into bicyclists, as well as facilitate faster access into and out of the parking space. Placement of parking stalls near intersections or mid-block crossings is critical so as to not impede sight lines of pedestrians entering crosswalks. Snow plowing could impact the availability of on-street parking intermittently. Requirements for ADA accessible on-street parking numbers and stall design must be adhered to. Information on those requirements can be found in Section 12A-2.

10. **Sidewalks:** Sidewalks are the one element of a complete street that is likely to provide a facility for all ages and abilities. Often sidewalks are the only way for young and older people alike to move throughout the community. Sidewalk connectivity is critical to encourage users. Sidewalks should be provided on both sides of all streets unless specific alternatives exist or safety is of concern. All sidewalks are required to meet ADA guidelines or be a part of a transition plan to be upgraded. Sections 12A-1 and 12A-2 identify the specific ADA requirements for sidewalks.
Sidewalks that are set back from the curb are safer than if the sidewalk is located at the back of curb. Street furniture and landscaping can add character and improve safety for sidewalks that are located at the back of curb. Providing seating areas within the sidewalk area can further enhance the urban environment and encourage pedestrian activity.

11. **Turn Lanes:** Turn lanes located at intersections provide opportunities for vehicles to exit the through lanes and improve capacity of the street. Two Way Left Turn Lanes (TWLTL) provide the opportunity to access midblock driveways without causing backups in the through lanes.

Turn lanes also allow faster speeds in the through lanes so a trade-off with safety exists especially at intersections.

Width of turn lanes should reflect the character of the traffic. Dedicated left and right turn lane widths should match the width of the lanes on the street. Local streets should not provide separate turn lanes. TWLTL should be a minimum of 12 feet wide because of the presence of through traffic on each side.

12. **Medians:** Medians provide for access management, pedestrian refuge, and additional space for landscaping, lighting, and utilities. Use of medians and the functions provided are dependent upon the width of available right-of-way and the other types of facilities that are included. The minimum width for pedestrian refuge is 6 feet. The minimum width of a median for access control and adjacent to left turn lanes is 4 feet. The minimum width for landscaped medians is 10 feet. Greater widths provide more opportunities for more extensive landscaping.

13. **Transit:** Bus service within the state is limited to the larger metropolitan areas. Currently there are a number of fixed route systems in the state. Smaller communities do not have fixed route service due to lack of demand. Children, elderly, and low-income people are the primary users of a fixed route transit system. In addition to system reliability, use of transit systems as a viable commuting option is directly dependent on the frequency of service and the destinations within the fixed route. To have a successful transit system, stops must be within walking or biking distance of residential areas to attract riders and it must have major retail, employment, and civic centers along its route system.

    Transit stops should be located on the far side of intersections to help reduce delays, minimize conflicts between buses and right turning vehicles, and encourage pedestrians to cross behind the bus where they are more visible to traffic. Far side stops also allow buses to take advantage of gaps in vehicular traffic.

    Bus turn out lanes are also best located on the far side of intersections. These turn outs free up the through lanes adjacent to the bus stop. Transit bulb outs are more pedestrian friendly than turnouts because they provide better visibility of the transit riders, as well as potentially providing space for bus shelters without creating congestion along the sidewalk. With buses stopping in the through lane, bulb-outs also provide traffic calming for the curb lane.

14. **Traffic Signals:** Traffic signals are not usually considered an element of complete streets, but they have many components with direct implications for complete streets. The timing, phasing, and coordination of traffic signals impacts all modes. Well-planned signal cycles reduce delay and unnecessary stops at intersections, thus improving traffic flow without street widening. Traffic signal timing can be designed to control vehicle operating speed along the street and to provide differing levels of protection for crossing pedestrians.

    The flashing don’t walk pedestrian phase should be set using a 3.5 feet per second walking speed and the full pedestrian crossing time (walk/flashing don’t walk) set using 3.0 feet per second. Some agencies representing the elderly are indicating that the overall walking speed should be 2.7 feet per second to cover a larger portion of the elderly population. ADA accessible pedestrian signal elements, such as audible signal indications, should be included in all new pedestrian signal installations and any installations being upgraded. See Section 13D-1, F for more information on accessible pedestrian signals.

15. **Summary:** The table below summarizes some of the critical design elements that should be examined if a complete streets project is implemented. Other geometric elements can be found in Table 5C-1.02. Some of the lane width values shown in the table below differ from the

acceptable values from Section 5C-1 because the expectation is that the complete street environment includes the potential for on-street parking and/or bike lanes. Adjustments in the values may be necessary to accommodate large volumes of trucks or buses. Contact the Jurisdictional Engineer if design exceptions are being considered.

Table 5M-1.01: Preferred Design Elements for Complete Streets

Classification	Local				Collector						Arterial					
Posted Speed (mph)	25		30		25		30		35 and Up		25		30		35 and Up	
Land use[1]	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*
Travel lane width (ft)[2]	10[3]	11	10	11	11	11	11	11	11	12	11	11	11	12	12	12
Turn lane width (ft)	--	--	--	--	11	11	11	11	11	12	11	11	11	12	12	12
Two-way left-turn lanes width (ft)	--	--	--	--	12	12	12	12	12	12	12	12	12	12	12	12
Curb Offset (ft)[4]	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	2	2	2	2	2	2
Parallel parking width (no buffer) (ft)[5]	8	8	8	8	8	9	8	9	9	9	10	10	10	10	10	10
Curb radii (ft)[6]	15	15	15	15	15	25	15	25	25	30	15	25	15	25	25	30
Bike lane width (ft)[7]	--	--	--	--	5	5	5	5	5	5	5	5	5	5	5	5

[1] Res. = Residential, C/I = Commercial/Industrial
[2] Minimum sharrow lane width is 13 feet.
[3] For low volume residential streets, two free flowing lanes are not required. They can operate as yield streets if parking is allowed on both sides and vehicles are parked across from each other.
[4] Curb offset, less the width of the curb, may be used in the parallel parking lane width.
[5] For arterial or high speed collectors, the parallel parking stall width may be reduced if a minimum 3 feet wide buffer strip is included.
[6] Curb radii may be adjusted based on design vehicle, presence of bike lanes or parking lanes, and the number of receiving lanes. Encroachment of turning vehicles into opposing lanes is not allowed.
[7] If paving is integral without a longitudinal gutter joint, the curb offset, less the width of the curb, may be used as part of a bike lane.

D. Traffic Calming

Traffic calming is different from but related to complete streets philosophies. Through design measures, traffic calming aims to slow traffic down to a desired speed. By slowing vehicular traffic, biking and pedestrian activities are made safer.

It is absolutely critical that traffic calming measures recognize the need to maintain access for emergency vehicles. Unless the situation is unusual, realizing slower speeds involves a series of traffic calming measures. However, too many measures along a street is likely to divert vehicles to adjacent streets and just move the problem or frustrate drivers to the point of complaining to the level necessary for removal of the traffic calming measures. Because of the anticipation that traffic will be just displaced to adjacent streets, it is very important to study a larger area than a single street when evaluating traffic calming measures.

Many design elements will accomplish traffic calming. These include the following.

- Reduction in lane widths:
  - Short medians
  - Bulb outs
  - Lane striping
- Lateral shifts
  - Chicanes
- Raised/tabled intersections
- Raised/tabled cross walks
- Speed humps or speed cushions
- Traffic circles
- Radar speed signs

Choosing the design elements to use for a particular area will depend on the neighborhood context and the specific concern to be addressed. Prior to evaluating alternative measures, stakeholders must be educated so they can have meaningful involvement. The evaluation needs to involve all stakeholders in the definition of the problem. If possible, all stakeholders, including drivers, pedestrians, bicyclists, and area property owners, would achieve some level of agreement on the traffic calming plan prior to implementation.

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CHAPTER 6
Geotechnical

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General Information

A. Introduction

The performance of pavements depends upon the quality of subgrades and subbases. A stable subgrade and properly draining subbase help produce a long-lasting pavement. A high level of spatial uniformity of a subgrade and subbase in terms of key engineering parameters such as shear strength, stiffness, volumetric stability, and permeability is vital for the effective performance of the pavement system. A number of environmental variables such as temperature and moisture affect these geotechnical characteristics, both in short and long term. The subgrade and subbase work as the foundation for the upper layers of the pavement system and are vital in resisting the detrimental effects of climate, as well as static and dynamic stresses that are generated by traffic. Furthermore, there has been a significant amount of research on stabilization/treatment techniques, including the use of recycled materials, geotextiles, and polymer grids for the design and construction of uniform and stable subgrades and subbases.

However, the interplay of geotechnical parameters and stabilization/treatment techniques is complex. This has resulted in a gap between the state-of-the-art understanding of geotechnical properties of subgrades and subbases based on research findings, and the design and construction practices for these elements. The purpose of this manual is to synthesize findings from previous and current research in Iowa and other states into a practical geotechnical design guide for subgrades and subbases. This design guide will help improve the design, construction, and testing of pavement foundations, which will in turn extend pavement life.

The primary consideration for this chapter is that new and reconstruction projects of pavement require characterization of the foundation soils and a geotechnical design. This chapter presents definitions of the terminology used and summarizes basic soil information needed by designers for different project types for pavement design and construction, including embankment construction, subgrade and subbase design and construction, subsurface drainage, and subgrade stabilization.

B. Definitions

Atterberg Limits:
- **Liquid Limit (LL):** The moisture content at which any increase in the moisture content will cause a plastic soil to behave as a liquid. The limit is defined as the moisture content, in percent, required to close a distance of 0.5 inches along the bottom of a groove after 25 blows in a liquid limit device.
- **Plastic Limit (PL):** The moisture content at which any increase in the moisture content will cause a semi-solid soil to become plastic. The limit is defined as the moisture content at which a thread of soil just crumbles when it is carefully rolled out to a diameter of 1/8 inch.
- **Plasticity Index (PI):** The difference between the liquid limit and the plastic limit. Soils with a high PI tend to be predominantly clay, while those with a lower PI tend to be predominantly silt.

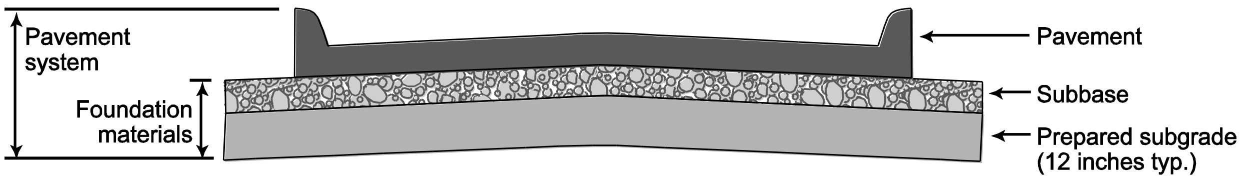
Flexible Pavement: Hot Mix Asphalt (HMA) pavement, also commonly called asphalt pavement.

Pavement System: Consists of the pavement and foundation materials (see Figure 6A-1.01).

Foundation Materials: Material that supports the pavement, which are layers of subbase and subgrade.

Pavement: The pavement structure, the upper surface of a pavement system, or the materials of which the pavement is constructed, including all lanes and the curb and gutter. Consist of flexible or rigid pavements, typically Hot Mix Asphalt (HMA) or PCC, respectively, or a composite of the two.

Figure 6A-1.01: Typical Section



Rigid Pavement: PCC pavement, also commonly called concrete pavement.

Subbase: The layer or layers of specified or selected material of designed thickness, placed on a subgrade to support a pavement. Also called granular subbase.

Subgrade: Consists of the naturally occurring material on which the road is built, or the imported fill material used to create an embankment on which the road pavement is constructed. Subgrades are also considered layers in the pavement design, with their thickness assumed to be infinite and their material characteristics assumed to be unchanged or unmodified. Prepared subgrade is typically the top 12 inches of subgrade.

Basic Soils Information

A. General Information

This section summarizes the basic soil properties and definitions required for designing pavement foundations and embankment construction. Basic soil classification and moisture-density relationships for compacted cohesive and cohesionless soil materials are included. The standard for soil density is determined as follows:

1. **Coarse-grained Soil:** The required minimum relative density and moisture range should be specified if it is a bulking soil.

2. **Fine-grained Soil:** The required minimum dry density should be specified; then the acceptable range of moisture content should be determined through which this density can be achieved.

3. **Inter-grade Soils:** The required minimum dry density or relative density should be specified, depending on the controlling test. Moisture range is determined by the controlling test.

B. Soil Types

1. **Soil:** Soils are sediments or other unconsolidated accumulation of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter. Soil has distinct advantages as a construction material, including its relative availability, low cost, simple construction techniques, and material properties which can be modified by mixing, blending, and compaction. However, there are distinct disadvantages to the use of soil as a construction material, including its non-homogeneity, variation in properties in space and time, changes in stress-strain response with loading, erodability, weathering, and difficulties in transitions between soil and rock.

   Prior to construction, engineers conduct site characterization, laboratory testing, and geotechnical analysis, design and engineering. During construction, engineers ensure that site conditions are as determined in the site characterization, provide quality control and quality assurance testing, and compare actual performance with predicted performance.

   Numerous soil classification systems have been developed, including geological classification based on parent material or transportation mechanism, agricultural classification based on particle size and fertility, and engineering classification based on particle size and engineering behavior. The purpose of engineering soil classification is to group soils with similar properties and to provide a common language by which to express general characteristics of soils.

   Engineering soil classification can be done based on soil particle size and by soil plasticity. Particle size is straightforward. Soil plasticity refers to the manner in which water interacts with the soil particles. Soils are generally classified into four groups using the Unified Soil Classification System, depending on the size of the majority of the soil particles (ASTM D 3282, AASHTO M 145).

a. **Gravel:** Fraction passing the 3 inch sieve and retained on the No. 10 sieve.

b. **Sand:** Fraction passing No. 10 sieve and retained on the No. 200 sieve.

c. **Silt and Clay:** Fraction passing the No. 200 sieve. To further distinguish between silt and clay, hydrometer analysis is required. Manually, clay feels slippery and sticky when moist, while silt feels slippery but not sticky.
   1) **Fat Clays:** Cohesive and compressible clay of high plasticity, containing a high proportion of minerals that make it greasy to the feel. It is difficult to work when damp, but strong when dry.
   2) **Lean Clays:** Clay of low-to-medium plasticity owing to a relatively high content of silt or sand.

2. **Rock:** Rocks are natural solid matter occurring in large masses or fragments.

3. **Iowa Soils:** The three major soils distributed across Iowa are loess, glacial till, and alluvium, which constitute more than 85% of the surface soil.

   a. **Loess:** A fine-grained, unstratified accumulation of clay and silt deposited by wind.

   b. **Glacial Till:** Unstratified soil deposited by a glacier; consists of sand, clay, gravel, and boulders.

   c. **Alluvium:** Clay, silt, or gravel carried by running streams and deposited where streams slow down.

C. Classification

Soils are classified to provide a common language and a general guide to their engineering behavior, using either the Unified Soil Classification System (USCS) (ASTM D 3282) or the AASHTO Classification System (AASHTO M 145). Use of either system depends on the size of the majority of the soil particles to classify the soil.

1. **USCS:** In the USCS (see Table 6A-2.01), each soil can be classified as:
   - Gravel (G)
   - Sand (S)
   - Silt (M)
   - Clay (C)

2. **AASHTO:** In the AASHTO system (see Table 6A-2.02), the soil is classified into seven major groups: A-1 through A-7. To classify the soil, laboratory tests including sieve analysis, hydrometer analysis, and Atterberg limits are required. After performing these tests, the particle size distribution curve (particle size vs. percent passing) is generated, and the following procedure can be used to classify the soil.

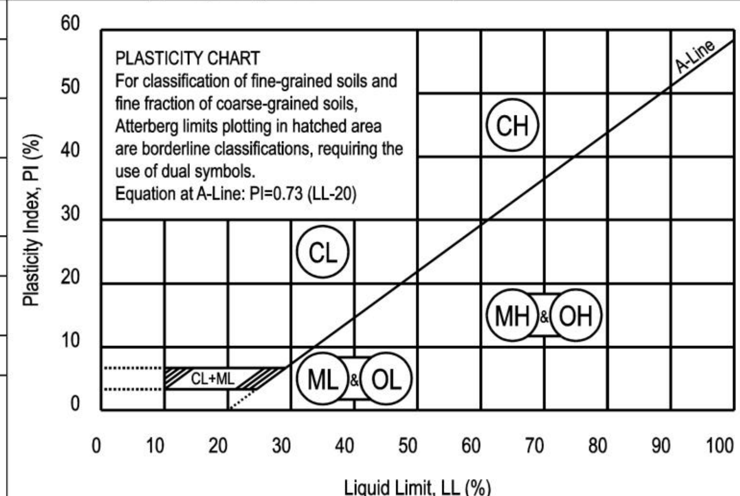
A comparison of the two systems is shown in Table 6A-2.03.

Table 6A-2.01: Unified Soil Classification System Soil Classification Chart

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES
ASTM D 2487 and D 2488
(Unified Soil Classification System)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	FIELD IDENTIFICATION PROCEDURES	CLASSIFICATION CRITERIA		
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve*	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	CLASSIFICATION ON BASIS OF PERCENTAGE OF FINES Less than 5% pass No. 200 sieve=GW, GP, SW, SP. 5% to 12% pass No. 200 sieve=borderline classification, requiring the use of dual symbols. More than 12% pass No. 200 sieve=GM, GC, SM, SC.	$C_u = D_{60}/D_{10}$ *Greater than 4*; $C_z = \frac{(D_{30})^2}{D_{10} x D_{60}}$ *Between 1 and 3*	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	Predominantly one size or a range of sizes with some intermediate sizes missing		Not meeting both criteria for GW	
		GRAVELS WITH FINES	GH	Silty gravels, gravel-sand-clay mixtures	Nonplastic fines or fines with low plasticity (for identification procedures, see ML below)		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures	Plastic fines (for identification procedures, see CL below)		Atterberg limits plot above "A" line and plasticity index greater than 7	
	SANDS More than 50% of coarse fraction passes No. 4 sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands, little or no fines	Wide range in grain size and substantial amounts of all intermediate particle sizes		$C_u = D_{60}/D_{10}$ *Greater than 6*; $C_z = \frac{(D_{30})^2}{D_{10} x D_{60}}$ *Between 1 and 3*	
			SP	Poorly graded sands and gravelly sands, little or no fines	Predominantly one size or a range of sizes with some intermediate sizes missing		Not meeting both criteria for SW	
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures	Nonplastic fines or fines with low plasticity (for identification procedures, see ML below)		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Plastic fines (for identification procedures, see CL below)		Atterberg limits plot above "A" line and plasticity index greater than 7	

MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	Identification Procedure On Fraction Smaller Than No. 40 Sieve Size: Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)
FINE-GRAINED SOILS 50% or more passes No. 200 sieve*	SILTS AND CLAYS Liquid limit 50% or less	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	None to slight	Quick to slow	None
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Medium to high	None to very slow	Medium
		OL	Organic silts and organic silty clays of low plasticity	Slight to medium	Slow	Slight
	SILTS AND CLAYS Liquid limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	Slight to medium	Slow to none	Slight to medium
		CH	Inorganic clays of high plasticity, fat clays	High to very high	None	High
		OH	Organic clays of medium to high plasticity	Medium to high	None to very slow	Slight to medium
Highly Organic Soils		PT	Peat, muck, and other highly organic soils	Readily identified by color, odor, spongy feel, and frequently by fibrous texture		



*Based on the material passing the 3 inch sieve

Table 6A-2.02: AASHTO Soil Classification Chart

General Classification	Granular Materials (35% or Less Passing No. 200)							Silt-Clay Materials (More Than 35% Passing No. 200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, percent passing:											
No. 10	50 max	--	--	--	--	--	--	--	--	--	--
No. 40	30 max	50 max	51 max	--	--	--	--	--	--	--	--
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid limit	--		--	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity limit	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

Source: AASHTO M 145-2

Table 6A-2.03: Comparison of the AASHTO system with the Unified Soil Classification System

Soil groups in AASHTO system	Comparable soil groups in USCS		
	Most probable	***Possible***	***Possible but improbable***
A-1-a	GW, GP	SW, SP	GM, SM
A-1-b	SW, SP, GM, SM	GP	---------------
A-3	SP	---------------	SW, GP
A-2-4	GM, SM	GC, SC	GW, GP, SW, SP
A-2-5	GM, SM	---------------	GW, GP, SW, SP
A-2-6	GC, SM	GM, SM	GW, GP, SW, SP
A-2-7	GM, GC, SM, SC	---------------	GW, GP, SW, SP
A-4	ML, OL	CL, SM, SC	GM, GC
A-5	OH, MH, ML, OL	---------------	SM, GM
A-6	CL	ML, OL, SC	GC, CM, CM
A-7-5	OH, MH	ML, OL, CH	GM, CM, GC, SC
A-7-6	CH, CL	ML, OL, SC	OH, MH, GC, GC, SM

Source: Liu, 1967

D. Moisture-Density Relationships for Soils

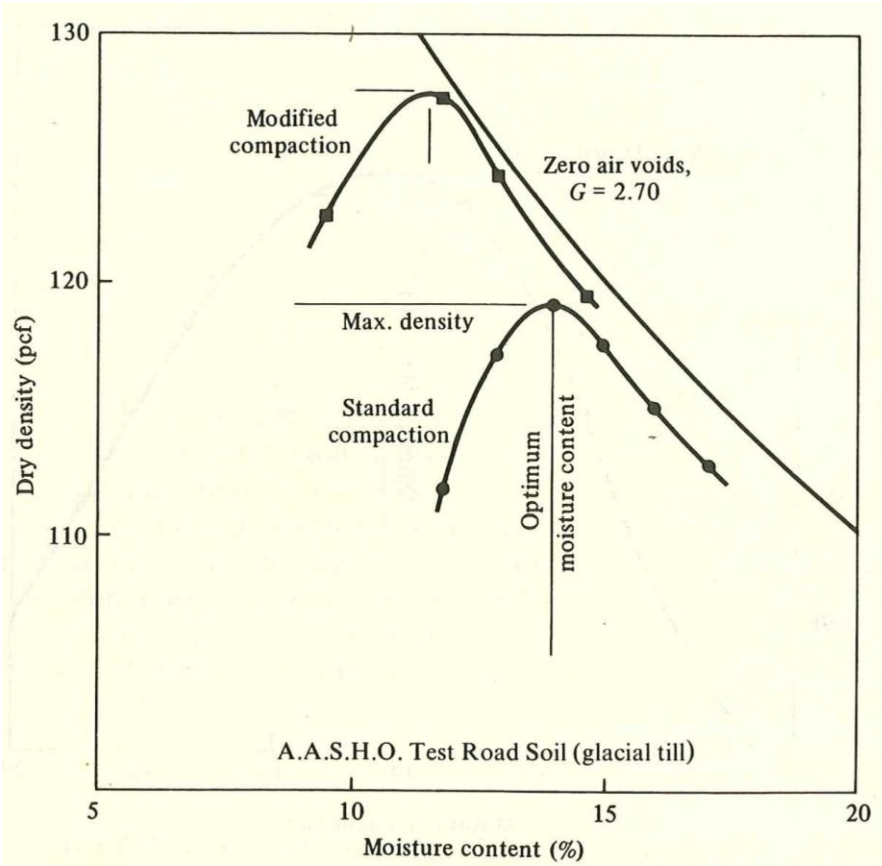
Compaction is the densification of soils by mechanical manipulation. Soil densification entails expelling air out of the soil, which improves the strength characteristics of soils, reduces compressibility, and reduces permeability. Using a given energy, the density of soil varies as a function of moisture content. This relationship is known as the moisture-density curve, or the compaction curve. The energy inputs to the soil have been standardized and are generally defined by Standard Proctor (ASTM D 698 and AASHTO T 99) and Modified Proctor (ASTM D 1557 and AASHTO T 180) tests. These tests are applicable for cohesive soils. For cohesionless soils, the relative density test should be used (ASTM D 4253 and ASTM D 4254). The information below describes the compaction results of both cohesive and cohesionless soils.

1. **Fine-grained (Cohesive) Soils:** The moisture-density relationship for fine-grained (cohesive) soils (silts and clays) is determined using Standard or Modified Proctor tests. Typical results of Standard Proctor tests are shown in Figure 6A-2.02, which represents the relationship between the moisture content and the dry density of the soil. At the peak point of the curve, moisture content is called the optimum moisture content, and the density is called the maximum dry density. If the moisture content exceeds the optimum moisture content, the soil is called wet of optimum. On the other hand, if the soil is drier than optimum, the soil is called dry of optimum.

   The compaction energy used in Modified Proctor is 4.5 times the compaction energy used in Standard Proctor. This increase in compaction energy changes the point-of-optimum moisture content and maximum dry density (see Figure 6A-2.02). In the field, the compaction energy is generally specified as a percentage of the Standard Proctor or Modified Proctor by multiplying the maximum dry density by this specified percent. Figure 6A-2.03 shows Proctor test results with a line corresponding to the specified percentage of the maximum dry density. The area between the curve and the specified percentage line would be the area of acceptable moisture and density.

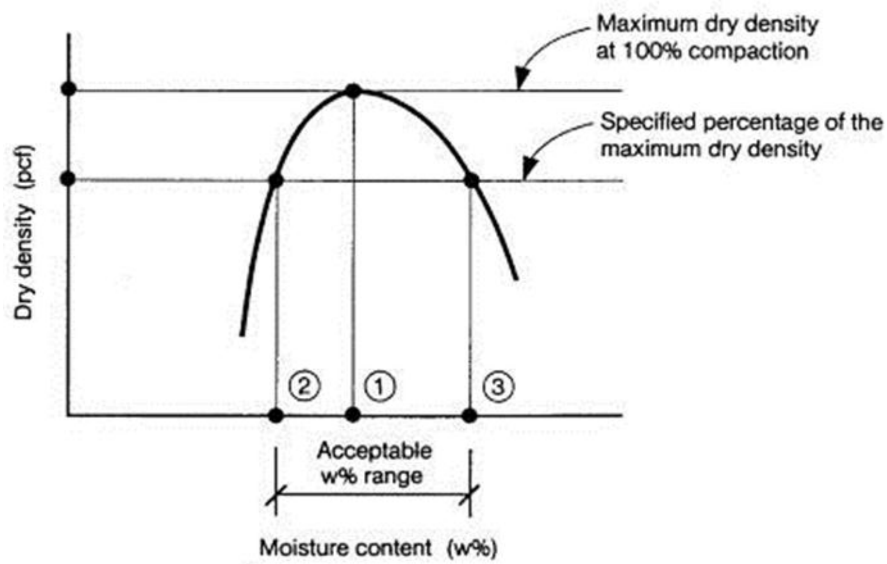
   Soils compacted on the dry side of optimum have higher strength, stability and less compressibility than the same soil compacted on the wet side of optimum. However, soils compacted on the wet side of optimum have less permeability and volume change due to change in moisture content. The question of whether to compact the soil on the dry side of optimum or on the wet side of optimum depends on the purpose of the construction and construction equipment. For example, when constructing an embankment, strength and stability are the main concern (not permeability); therefore, a moisture content on the dry side of optimum should be used. For contractors, compacting the soil on the wet side of optimum is more economical, especially if it is within 2% of the optimum moisture content. However, if the soil is too wet, the specified compaction density will not be reached.

Figure 6A-2.02: An Example of Standard and Modified Proctor Moisture-Density Curves for the Same Soil



Source: Spangler and Handy 1982

Figure 6A-2.03: Example Proctor Test Results with Specified Percentage Compaction Line



Source: Duncan 1992

2. **Coarse-grained (Cohesionless) Soils:** When coarse-grained, cohesionless soils (sands and gravels) are compacted using standard or modified Proctor procedures, the moisture-density curve is not as distinct as that shown for cohesive soils in Figure 6A-2.02. Figure 6A-2.04 shows a typical curve for cohesionless materials, exhibiting what is often referred to as a hump back or camel back shape. It can be seen that the granular material achieves its densest state at 0% moisture, then decreases to a relative low value, and then increases to a relative maximum, before decreasing again with increasing water content. A better way of representing the density of cohesionless soils is through relative density. Tests can be conducted to determine the maximum density of the soil at its densest state and the minimum density at its loosest state (ASTM D 4253 and D 4254). The relative density of a field soil, D_r, can be defined using the density measured in the field, through a ratio to the maximum and the minimum density of the soil, using Equation 6A-2.01.

$$D_r(\%) = \left[\frac{\gamma_{d(field)} - \gamma_{d(\min)}}{\gamma_{d(\max)} - \gamma_{d(\min)}}\right]\left[\frac{\gamma_{d(\max)}}{\gamma_{d(field)}}\right]$$ **Equation 6A-2.01**

where:

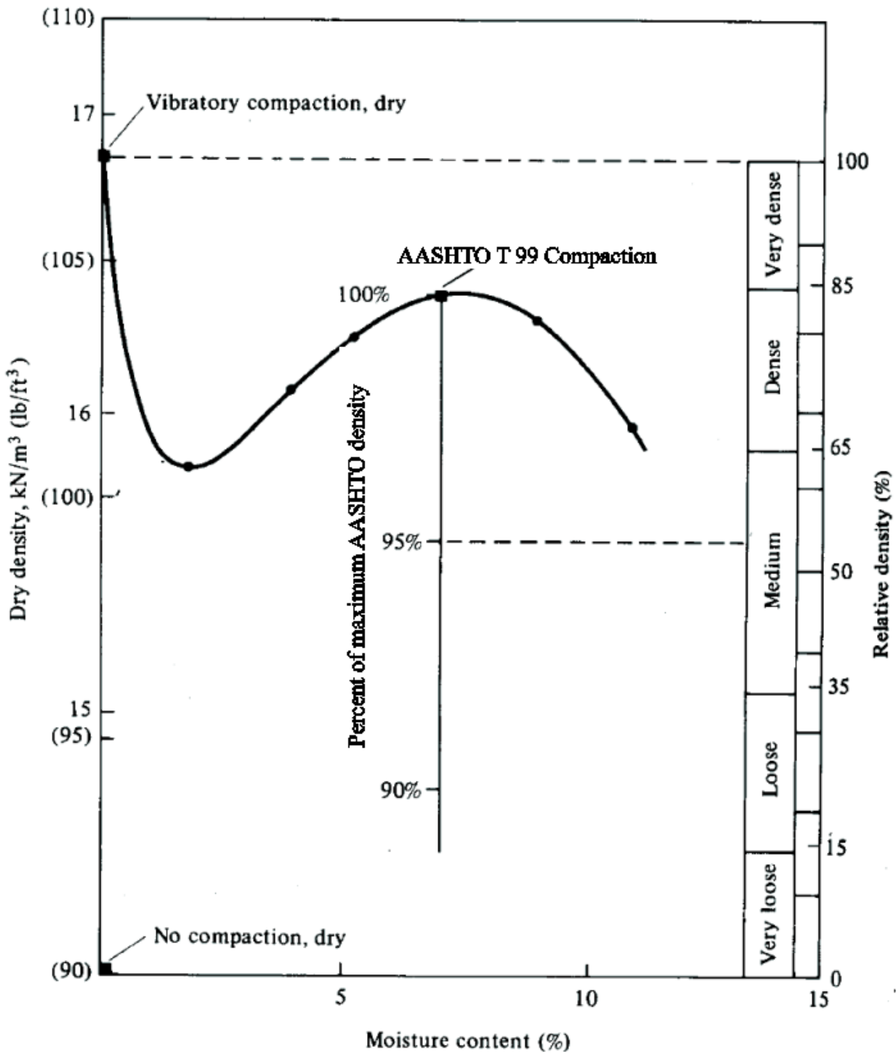
$\gamma_{d(field)}$ = field density

$\gamma_{d(\min)}$ = minimum density

$\gamma_{d(\max)}$ = maximum density

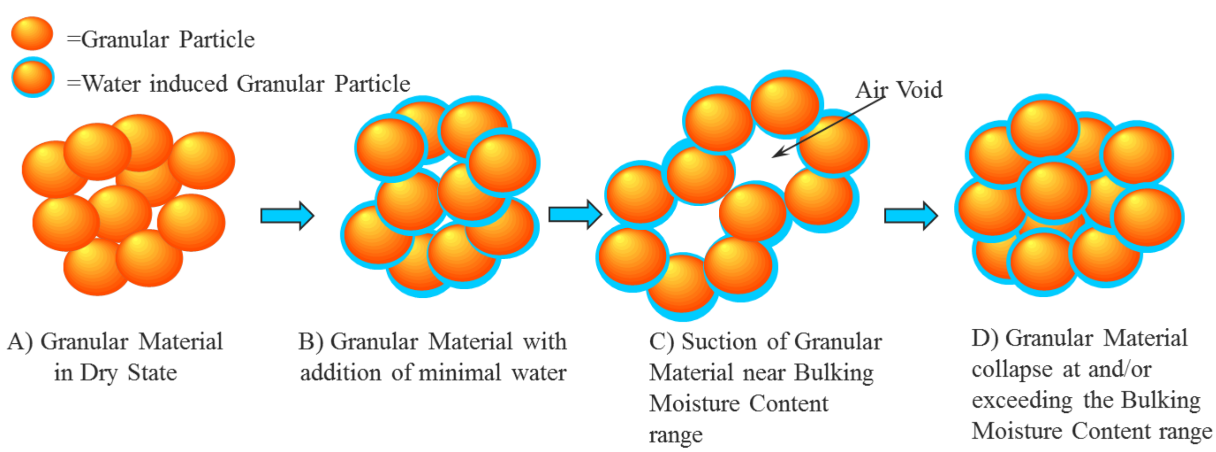
The maximum and minimum density testing is performed on oven-dry cohesionless soil samples. However, soils in the field are rarely this dry, and cohesionless soils are known to experience bulking as a result of capillary tension between soil particles. Bulking is a capillary phenomena occurring in moist sands (typically 3 to 5% moisture) in which capillary menisci between soil particles hold the soil particles together in a honeycomb structure. This structure can prevent adequate compaction of the soil particles and is also susceptible to collapse upon the addition of water (see Figure 6A-2.05). The bulking moisture content should be avoided in the field.

Figure 6A-2.04: Example of Relative Density vs. Standard Proctor Compaction



Source: Spangler and Handy 1982

Figure 6A-2.05: Example Showing the Processes of Collapse due to Bulking Moisture



Source: Schaefer et al. 2005

E. References

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Schaefer, V.R., M.T. Suleiman, D.J. White, and C. Swan. *Utility Cut Repair Techniques - Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Areas*. Iowa: Report No. TR-503, Iowa Department of Transportation. 2005.

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Typical Iowa Soils

A. General Information

There are three major types of soils in Iowa:

1. **Loess:** A fine-grained, unstratified accumulation of clay and silt deposited by wind (37.5%).

2. **Glacial Till:** Unstratified soil deposited by a glacier; consists of clay, silt, sand, gravel, and boulders (28.5%).

3. **Alluvium:** Clay, silt, sand, or gravel carried by running streams and deposited where streams slow down (20.1%).

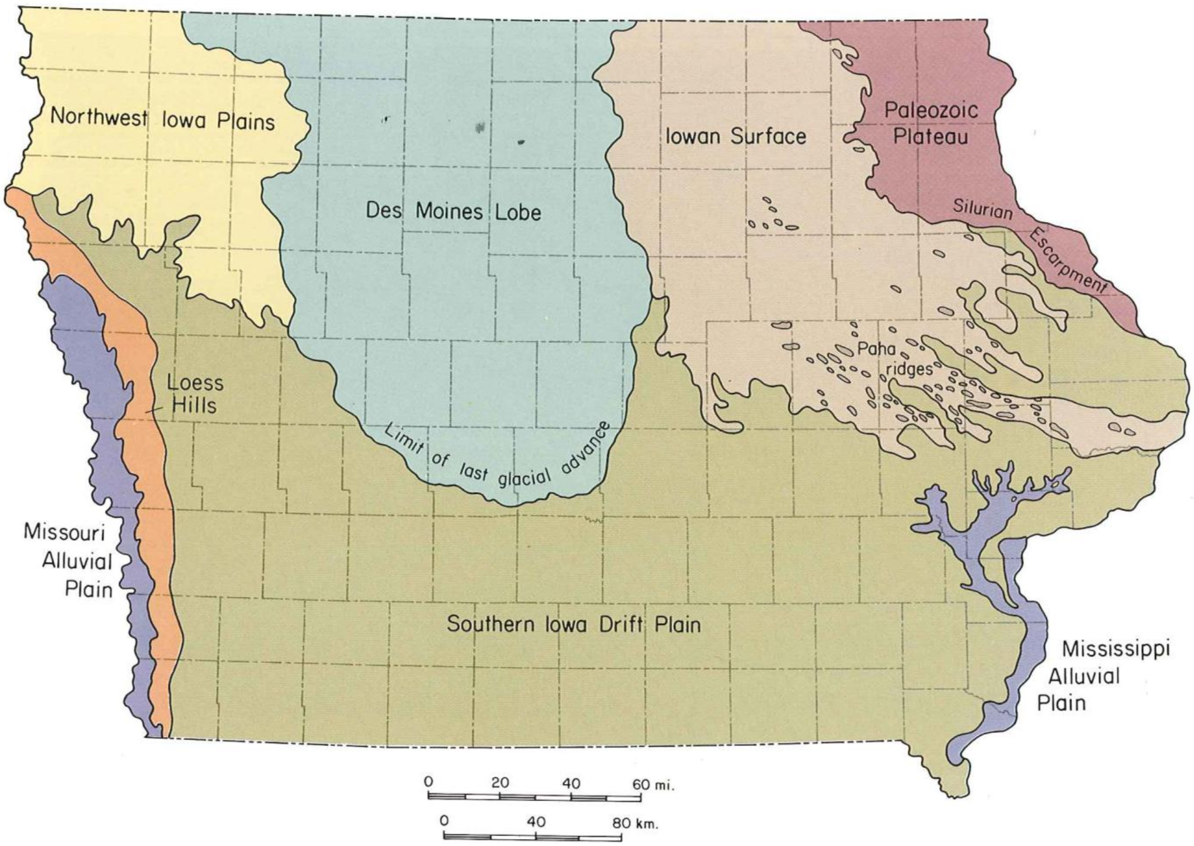
Other types of soils, occurring in smaller amounts in Iowa, are:
- Sand and gravel (4.5%)
- Paleosols (4.0%)
- Bedrock (2.7%)
- Fine sand (1.4%)

B. Iowa Geology

The Iowa landscape consists mainly of seven topographic regions (see Figure 6A-3.01).
- Des Moines Lobe
- Loess Hills
- Southern Iowa Drift Plain
- Iowan Surface
- Northwest Iowa Plains
- Paleozoic Plateau
- Alluvial Plains

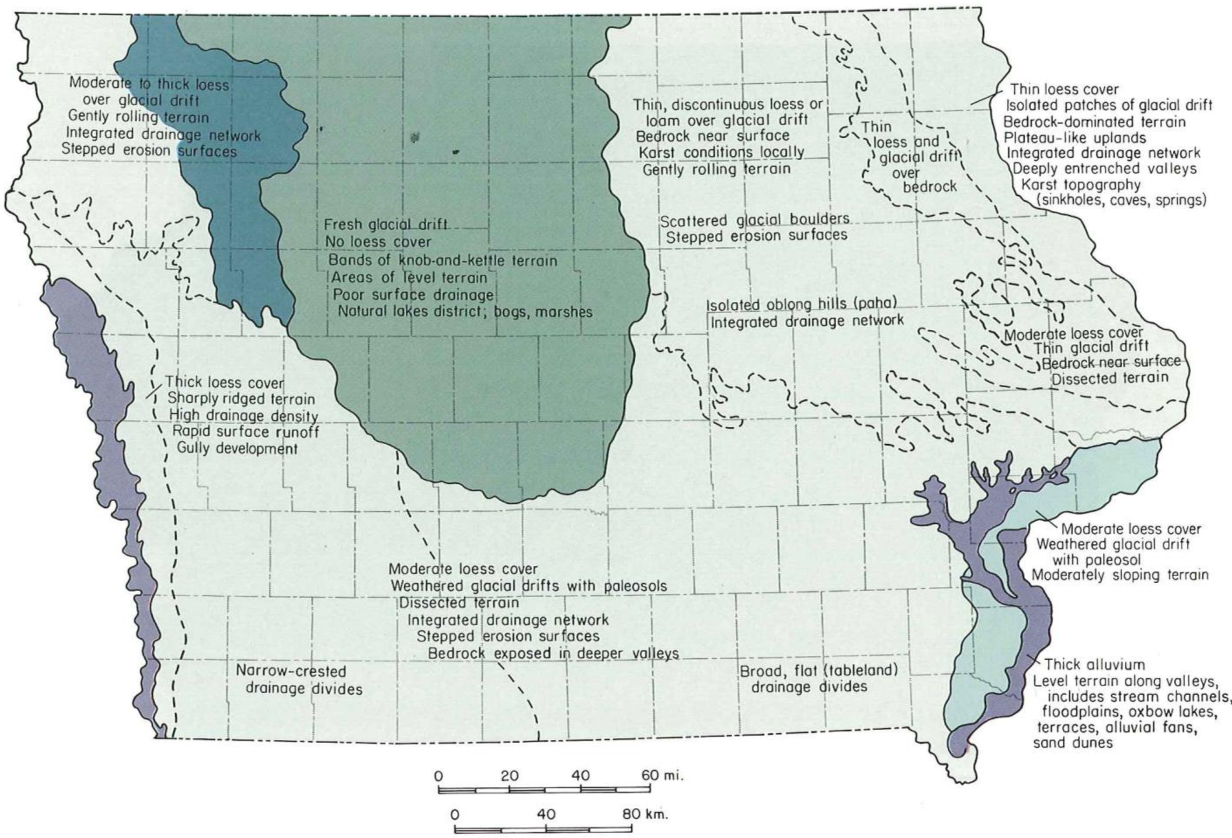
The soils in the Des Moines Lobe, Southern Iowa Drift Plain, Iowan Surface, Northwest Iowa Plains, and Paleozoic Plateau originated from glacial action at different periods in geologic time. The northwestern and southern parts of the state consist of glacial till covered by loess. The engineering properties of glacial till change as the age of glacial action changes. Loess soil engineering properties depend mainly on clay content. Figures 6A-3.01, 6A-3.02, and 6A-3.03 show the landform regions, the landform materials and terrain characteristics, and soil permeability.

Figure 6A-3.01: Landform Regions of Iowa



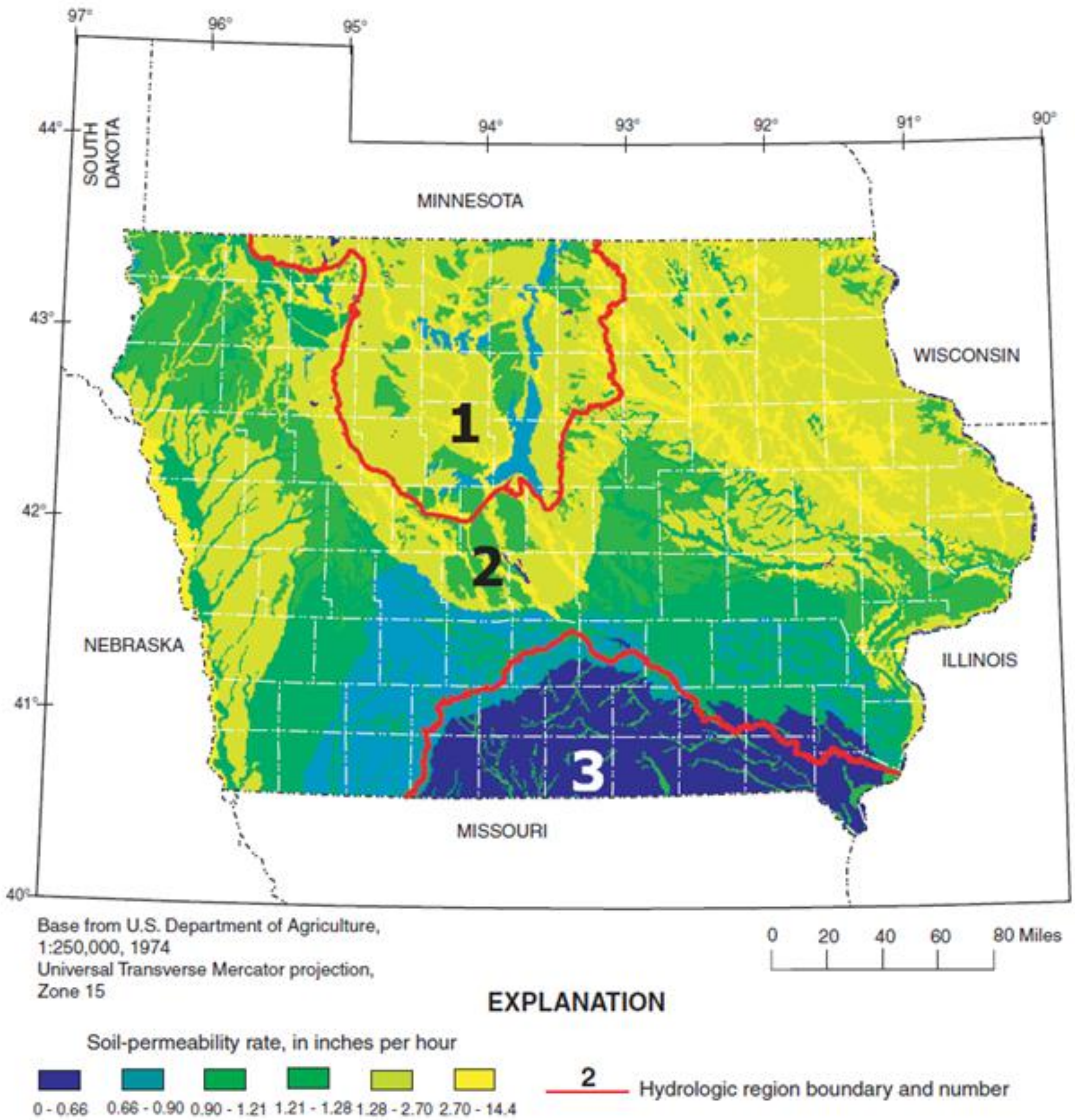
Source: Prior 1991

Figure 6A-3.02: Landform Materials and Terrain Characteristics of Iowa



Source: Prior 1991

Figure 6A-3.03: Soil Permeability Rates and Hydrologic Regions in Iowa



Source: Eash 2001

C. References

Eash, D.A. *Techniques for Estimating Flood-Frequencies Discharges for Streams in Iowa*. Iowa City, Iowa: Iowa Department of Transportation and the Iowa Highway Research Board. 2001.

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Subsurface Exploration Program

A. General Information

A subsurface exploration program is conducted to make designers aware of the site characteristics and properties needed for design and construction. The horizontal and vertical variations in subsurface soil types, moisture contents, densities, and water table depths must be considered during the pavement design process. The purpose of conducting a subsurface exploration is to describe the geometry of the soil, rock, and water beneath the surface; and to determine the relevant engineering characteristics of the earth materials using field tests and/or laboratory tests. More importantly, special subsurface conditions, such as swelling soils and frost-susceptible soils, must be identified and considered in pavement design. The phases of the subsurface exploration program, as well as the in-situ test, are summarized below.

B. Program Phases

The objective of subsurface investigations or field exploration is to obtain sufficient subsurface data to permit selection of the types, locations, and principal dimensions of foundations for all roadways comprising the proposed project. These explorations should identify the site in sufficient detail for the development of feasible and cost-effective pavement designs. Often the site investigation can proceed in phases, including desk study prior to initiating the site investigation. For the desk study, the geotechnical engineer needs to:

1. Review existing subsurface information. Possible sources of information include:

    a. Previous geotechnical reports

    b. Prior construction and records of structural performance problems at the site

    c. U.S. Geological Survey (USGS) maps, reports, publications, and Iowa Geological Survey website

    d. State geological survey maps, reports, and publications

    e. Aerial photographs

    f. State, city, and county road maps

    g. Local university libraries

    h. Public libraries

2. Obtain from the design engineer the geometry and elevation of the proposed facility, load and performance criteria, and the locations and dimensions of the cuts and fills.

3. Visit the site with the project design engineer if possible, with a plan in-hand. Review the following:

    a. General site conditions

    b. Geologic reconnaissance

    c. Geomorphology

    d. Location of underground and aboveground utilities

    e. Type and condition of existing facilities

    f. Access restriction for equipment

    g. Traffic control required during field investigation

    h. Right-of-way constraints

    i. Flood levels

    j. Benchmarks and other reference points

4. Based on the three steps above, plan the subsurface exploration location, frequency and depth. General guidelines are provided below.

C. Site Characterization

1. Frequency and Depth of Borings:

a. Roadways: 200 feet is generally the maximum spacing along the roadway. The location and spacing of borings may need to be changed due to the complexity of the soil/rock conditions.

b. Cuts: At least one boring should be performed for each cut slope. If the length of cuts is more than 200 feet, the spacing between borings should be 200 to 400 feet. At critical locations and high cuts, provide at least three borings in transverse direction to explore the geology conditions for stability analysis. For an active slide, place at least one boring upslope of the sliding area.

c. Embankment: See criteria for cuts.

d. Culverts: At least one boring should be performed at each major culvert. Additional borings may be provided in areas of erratic subsurface conditions.

e. Retaining Walls: At least one boring should be performed at each retaining wall. For retaining walls more than 100 feet in length, the spacing between borings should be no more than 200 feet.

f. Bridge Foundations: For piers or abutments greater than 100 feet wide, at least two borings should be performed. For piers or abutments less than 100 feet wide, at least one boring should be performed. Additional borings may be performed in areas of erratic subsurface conditions.

2. **Depth Requirements for Borings:**

   a. **Roadways:** Minimum depth should be 6 feet below the proposed subgrade.

   b. **Cuts:** Minimum depth should be 16 feet below the anticipated depth of the cut at the ditch line. The depth should be increased where the location is unstable due to soft soils, or if the base of the cut is below groundwater level.

   c. **Embankments:** Minimum depth should be up to twice the height of the embankment unless hard stratum is encountered above the minimum depth. If soft strata are encountered, which may present instability or settlement concerns, the boring depth should extend to hard material.

   d. **Culverts:** See criteria for embankments.

   e. **Retaining Walls:** Depth should be below the final ground line, between 0.75 and 1.5 times the height of the wall. If the strata indicate unstable conditions, the depth should extend to hard stratum.

   f. **Bridge Foundations:**
      1) **Spread Footings:** For isolated footings with a length (L) and width (B):
         a) If $L \leq 2B$, minimum 2B below the foundation level.
         b) If $L \geq 5B$, minimum 4B below the foundation level.
         c) If $2B \leq L \leq 5B$, minimum can determined by interpolation between the depths of 2B and 5B below the foundation level.
      2) **Deep Foundations:**
         a) For piles in soil, use the greater depth of 20 feet or a minimum of two times of the pile group dimension below the anticipated elevation.
         b) For piles on rock, a minimum 10 feet of rock core needs to be obtained at each boring location.
         c) For shaft supported on rock or into the rock, use the greatest depth of 10 feet, three times the isolated shaft diameter, or two times of the maximum of shaft group dimension.

3. **Types of Borings:**

   a. **Solid Stem Continuous Flight Augers:** Solid stem continuous flight auger drilling is generally limited to stiff cohesive soils where the boring walls are stable for the whole depth of boring. This type of drilling is not suitable for investigations requiring soil sampling.

   b. **Hollow Stem Continuous Flight Augers:** Hollow stem augering methods are commonly used in clay soils or in granular soils above the groundwater level, where the boring walls may be unstable. These augering methods allow for sampling undisturbed soil below the bit.

   c. **Rotary Wash Borings:** The rotary wash boring method is generally suitable for use below groundwater level. When boring, the sides of the borehole are supported with either casing or the use of drilling fluid.

   d. **Bucket Auger Borings:** Bucket auger drills are used where it is desirable to remove and/or obtain large volumes of disturbed soil samples. This method is appropriate for most types of soils and for soft to firm bedrock. Drilling below the water table can be conducted where materials are firm and not inclined to large-scale sloughing or water infiltration.

e. **Hand Auger Borings:** Hand augers are often used to obtain shallow subsurface information from the site with difficult access or terrain that a vehicle cannot easily reach.

f. **Exploration Pit Excavation:** Exploration pits and trenches permit detailed examination of the soil and rock conditions at shallow depths at relatively low cost. They can be used where significant variations in soil conditions, large soil, and/or non-soil materials exist (boulders, cobbles, debris, etc.) that cannot be sampled with conventional methods, or for buried features that must be identified.

D. Sampling

1. **Disturbed Sampling:** Disturbed samples are those obtained using equipment that destroys the macrostructure of the soil without altering its mineralogical composition. Specimens from these samples can be used to determine the general lithology of soil deposits, identify soil components and general classification purposes, and determine grain size, Atterberg limits, and compaction characteristics of soils. There are four well-known types of samplers for distributed samples, which are shown in Table 6B-1.01.

Table 6B-1.01: Types of Samplers (Disturbed)

Sampler	Appropriate Soil Types	Method of Penetration	Frequency of Use
Split-barrel (split-spoon)	Sands, silts, clays	Hammer-driven	Very frequent
Modified California	Sands, silts, clays, gravels	Hammer-driven (large split-spoon)	Rare
Continuous auger	Cohesive soils	Drilling with hollow stem augers	Rare
Bulk	Gravels, sands, silts, clays	Hand tools, bucket augering	Rare

2. **Undisturbed Sampling:** Clay and granular samples can be obtained with specialized equipment designed to minimize the disturbance to the in-situ structure and moisture content of the soils. Specimens obtained by undisturbed sampling methods are used to determine the strength, stratification permeability, density, consolidation, dynamic properties, and other engineering characteristics of soils. There are six types of samplers to obtain undisturbed samples, of which the thin-walled Shelby tube is the most common. These samplers are shown in Table 6B-1.02.

Table 6B-1.02: Types of Samplers (Undisturbed)

Sampler	Appropriate Soil Types	Method of Penetration	Frequency of Use
Thin-walled Shelby tube	Clays, silts, fine-grained soils, clayey sands	Mechanically or hydraulically pushed	Frequent
Continuous push	Sands, silts, clays	Hydraulic push with plastic lining	Less frequent
Piston	Silts, clays	Hydraulic push	Less frequent
Pitcher	Stiff to hard clay, silt, sand, partially weathered rock, and frozen or resin-impregnated granular soil	Rotation and hydraulic pressure	Rare
Denison	Stiff to hard clay, silt, sand, and partially weathered rock	Rotation and hydraulic pressure	Rare
Block	Cohesive soils and frozen or resin-impregnated granular soil	Hand tools	Rare

Testing

A. General Information

Several testing methods can be used to measure soil engineering properties. The advantages, disadvantages, and measured soil properties for each test are summarized below.

B. Field Testing

1. **Types of In-situ Equipment:**

   a. **Standard Penetration Test (SPT):** SPT test procedures are detailed in ASTM D 1586 and AASHTO T 206. The SPT consists of advancing a standard sampler into the ground, using a 140 pound weight dropped 30 inches. The sampler is advanced in three 6 inch increments, the first increment to seat the sampler. The SPT blow count is the number of blows required to advance the sampler into the final 12 inches of soil.

   Advantages of the Standard Penetration Test are that both a sample and number are obtained; in addition, the test is simple and rugged, is suitable in many soil types, can perform in weak rocks, and is available throughout the U.S. Disadvantages are that index tests result in a disturbed sample, the number for analysis is crude, the test is not applicable in soft clay and silts, and there is high variability and uncertainty.

   b. **Cone Penetration Test (CPT):** The CPT test is an economical in-situ test, providing continuous profiling of geostratigraphy and soil properties evaluation. The steps can follow ASTM D 3441 (mechanical systems) and ASTM D 5778 (electronic system). The CPT consists of a small-diameter, cone-tipped rod that is advanced into the ground at a set rate. Measurements are made of the resistance to ground penetration at both the tip and along the side. These measurements are used to classify soils, estimate the friction angle of sands, and estimate the shear strength of soft clays.

   Advantages of the Cone Penetration Test include fast and continuous profiling, economical and productive operation, non-operator-dependent results, a strong theoretical basis in interpretation, and particular suitability for soft soils. Disadvantages include a high capital investment, a skilled operator to run the test, unavoidable electronic drift noise and calibration, no collection of soil samples, and unsuitability to test gravel or boulder deposits.

   c. **Borehole Shear Test (BST):** BST is performed according to the instructions published by Handy Geotechnical Instruments, Inc.

   Advantages of the Borehole Shear Test include its direct evaluation of soil cohesion (C), and friction angle (ϕ), at a particular depth, and its yielding of a large amount of soil cohesion and friction angle data in a short time. Disadvantages include difficulty to fix the test rate and the drainage condition of the sample, and no collection of stress-strain data.

d. Flat Plate Dilatometer Test (DMT): DMT is performed according to ASTM D 6635, which provides the overview of this device and its operation sequence.

Advantages of the Flat Plate Dilatometer Test are that it is simple and robust, results are repeatable and operator-independent, and it is quick and economical. Disadvantages are that it is difficult to push in dense and hard materials, it primarily relies on correlative relationships, and that it needs calibration for local geologies.

e. Pressuremeter Test (PMT): There are several types of pressuremeter procedures, such as Pre-bored-Menard (MPM), Self-boring pressuremeter (SBP), Push-in pressuremeter (PIP), and Full-displacement cone pressuremeter (CPM). Procedures and calibrations are given in ASTM D 4719.

Advantages of the Pressuremeter Test are that it is theoretically sound in determination of soil parameters, it tests a larger zone of soil mass than other in-situ tests, and it develops a complete curve. Disadvantages are that the procedures are complicated, it requires a high level of expertise in the field, it is time consuming and expensive (a good day yields 6 to 8 complete tests), and the equipment is delicate and easily damaged.

f. Vane Shear Test (VST): The instructions for the Vane Shear Test are found in ASTM D 2573.

Advantages of the Vane Shear Test are that it provides an assessment of undrained shear strength (S_u), the test and equipment are simple; it can measure in-situ clay sensitivity (S_t), and there is a long history of use in practice. Disadvantages are that application for soft-to-stiff clays is limited, and it is slow and time consuming. In addition, raw, undrained shear strength needs empirical correction and can be affected by sand lenses and seams.

2. Correlations with Soil Properties: Tables 6B-2.01 and 6B-2.02 summarize the measured output values from each in-situ test, the use of the values to evaluate different soil properties, the soil types with which the tests can be used, and correlations used to evaluate soil properties.

Table 6B-2.01: In-situ Methods and General Application

Method	Output	Applicable Soil Properties	Applicable for Soil Properties	Applicable for Soil Types
SPT	N	Soil identification	Medium	Sands
		Establish vertical profile	Medium	
		Relative density (D_r)	Medium	
CPT	Cone resistance (q_c), Sleeve friction (f_s)	Establish vertical profile	Most	Silts, sands, clays, and peat
		Relative density (D_r)	Most	
		Angle of friction (ϕ')	Medium	
		Undrained shear strength (S_u)	Medium	
		Pore pressure (U)	Most	
		Modulus (E)	Medium	
		Compressibility	Medium	
		Consolidation	Most	
		Permeability (k)	Medium	
BST	σ and τ	Angle of friction (ϕ')	Most	Sands, silts and clays
		Cohesion (C')	Most	
DMT	P_0, P_1, P_2, I_D, E_D, K_D	Establish vertical profile	Most	Silts, sands, clays, and peat
		Soil identification	Medium	
		Relative density (D_r)	Medium	
		Undrained shear strength (S_u)	Medium	
PMT (pre-bored)	V_0, V, ΔP, ΔV, E_p	Soil identification	Medium	Clays, silts, and peat; marginal response in some sands and gravels
		Establish vertical profile	Medium	
		Angle of friction (ϕ')	Medium	
		Undrained shear strength (S_u)	Medium	
		Modulus (E & G)	Medium	
		Compressibility	Medium	
VST	T_{max}	Undrained shear strength (S_u)	Most	Clays, some silts, and peat (undrained condition); not for use in granular soils
		Soil identification	Medium	
		Overconsolidation ratio (OCR), K_0	Medium	
		Sensitivity (S_t)	Most	
		Pre-consolidation stress (P_C')	Medium	

Table 6B-2.02: Correlations Between In-situ Tests and Soil Properties

Method	Correlations	Applicable Soil Types
SPT	$\phi=28^{o}+15^{o}D_r$	Granular soils
	$\phi=0.45\,N'_{70}+20$	Granular soils
	$q_u=kN_{70}$	Cohesive soils
CPT	$S_u=\dfrac{q_c-p_0}{N_k}$ ($P_0=\gamma z$, N_k=cone factor, from 5 to 75)	Cohesive soils
	$\phi=29^{o}+\sqrt{q_c}$	Granular soils
BST	$\tau=c+\sigma\tan\phi$	Cohesive soils
DMT	$K_o=\left(\dfrac{K_D}{\beta_D}\right)^{\partial}-C_D$	Granular and cohesive soils
PMT (pre-bored)	$K_o=\dfrac{p_h}{p_0}$	Cohesive soils
VST	$S_u=0.2738\,\dfrac{T}{d^3}$	Cohesive soils

C. Laboratory Testing

1. **Index Testing and Soil Classification:** AASHTO and ASTM standards for frequently used laboratory index testing of soils are summarized in Table 6B-2.03 below.

Table 6B-2.03: Index Testing and Soil Classification

Test	Test Designation		Applicable Soil Properties	Applicable Soil Types	Complexity
	AASHTO	***ASTM***			
Test method for determination of water content	T 265	D 4959	Void ratio (e) and unit weight (γ)	Gravels, sands, Silts, clays, peat	Simple
Test method for specific gravity of soils	T 100	D 854	Specific gravity (G_s)	Sands, silts, Clays, peat	Simple
Method for particle-size analysis of soils	T 88	D 422	Classification	Gravels, sands, Silts	Simple
Test method for amount of material in soils finer than the No. 200 sieve		D 1140	Soil classification	Fine sands, Silts, clays	Simple
Test method for Liquid Limit, Plastic Limit, and Plasticity Index of soils	T 89	D 4318	Soil classification	Clays, silts, peat; silty and clayey sands to determine whether SM or SC	Simple
Unit weight, density		D 1587	Total density (e.g., wet density) (γ_t); Dry density (γ_d)	Undisturbed samples can be taken, i.e., silts, clays, peat	Simple

2. **Shear Strength Testing:** AASHTO and ASTM standards for frequently used laboratory strength properties testing of soils are shown in Table 6B-2.04.

Table 6B-2.04: Shear Strength Tests

Test	Test Designation		Applicable Soil Properties	Applicable Soil Types	Complexity
	AASHTO	***ASTM***			
Unconfined compressive strength of cohesive soil	T 208	D 2166	Undrained shear strength (S_u)	Clays and silts	Simple
Unconsolidated, undrained compressive strength of clay and silt soils in tri-axial compression	T 296	D 2850	Undrained shear strength (S_u)	Clays and silts	Simple
Consolidated, undrained triaxial compression test on cohesive soils	T 297	D 4767	Friction angle (ϕ), Cohesion (C)	Clays and silts	Medium
Direct shear test of soils for consolidated drained conditions	T 236	D 3080	Friction angle (ϕ')	Compacted fill materials; sands, silts, and clays	Simple
Modulus and damping of soils by the resonant-column method (small-strain properties)		D 4015	Shear modulus (G_{max}), Damping (D)	Gravel, sand, silt, and clay	Complicated
Test method for laboratory miniature vane shear test for saturated fine-grained clayey soil		D 4648	Undrained shear strength (S_u) Clay sensitivity (S_t)	Silts and clays	Simple
Test method for CBR (California Bearing Ratio) of laboratory-compacted soils		D 1883	Bearing capacity of a compacted soil	Gravels, sands, silts, and clays	Complicated
Test method for resilient modulus of soils	T 294		Relations between applied stress and deformation of pavement materials	Gravels, sands, silts, and clays	Time consuming
Method for resistance R-value and expansion pressure of compacted soils	T 190	D 2844	Resist lateral deformation resistance	Gravels, sands, silts, and clays	Complicated

3. **Settlement Testing:** AASHTO and ASTM standards for frequently used laboratory compression properties of soils are summarized in Table 6B-2.05.

Table 6B-2.05: Laboratory Test Used to Measure the Compression Properties of Soils

Test	Test Designation		Applicable Soil Types	Complexity
	AASHTO	***ASTM***		
Method for one-dimensional consolidation properties of soils (oedometer test)	T 216	D 2435	Primarily clays and silts	Simple but time consuming
Test methods for one-dimensional swell or settlement potential of cohesive soils	T 256	D 4546	Clays	Medium
Test method for measurement of collapse potential of soils		D 5333	Silts	Medium

Geotechnical Report

A. Geotechnical Report

The results of the explorations and laboratory testing are usually presented in the form of a geology and soils report. This report should contain sufficient descriptions of the field and laboratory investigations performed, the conditions encountered, typical test data, basic assumptions, and the analytical procedures utilized; to allow a detailed review of the conclusions, recommendations, and final pavement design. The amount and type of information to be presented in the design analysis report should be consistent with the scope of the investigation. For pavements, the following items (when applicable) should be included and used as a guide in preparing the design analysis report:

1. A general description of the site, indicating principal topographic features in the vicinity. A plan map should show surface contours, the locations of the proposed structure, and the location of all borings.

2. A description of the general geology of the site, including the results of any previous geologic studies performed.

3. The results of field investigations, including graphic logs of all foundation borings, locations of pertinent data from piezometers (when applicable), depth to bedrock, and a general description of the subsurface materials based on the borings. The boring logs or report should indicate how the borings were made, the type of sampler used, and any penetration test results, or other field measurement data taken on the site.

4. Groundwater conditions, including data on seasonal variations in groundwater level and results of field pumping tests, if performed.

5. Computation of the resilient modulus for the total vertical and horizontal stresses using the constitutive relationship.

6. A generalized soil profile used for design, showing average or representative soil properties and values of design shear strength used for various soil strata. The profile may be described in writing or shown graphically.

7. Recommendations on the type of pavement structure and any special design feature to be used, including removal and replacement of certain soils and stabilization of soils or other foundation improvements, and treatments.

8. Basic assumptions, imposed wheel loads, results of any settlement analyses, and an estimate of the maximum amount of swell to be expected in the subgrade soils. The effects of the computed differential settlement, and also the effects of the swell on the pavement structure, should be discussed.

9. Special precautions and recommendations for construction techniques. Locations at which material for fill and backfill can be obtained should also be discussed as well as the amount of compaction required and procedures planned for meeting these requirements.

In summary, the horizontal and vertical variations in subsurface soil types, moisture contents, densities, and water table depths should be identified for both new and existing pavements. FHWA Report No. FHWA-RD-97-083 (VonQuintus and Killingsworth 1997) provides general guidance and requirements for subsurface investigations for pavement design and evaluations for rehabilitation designs. Each soil stratum encountered should be characterized for its use to support pavement structures and whether the subsurface soils would impose special problems for the construction and long-term performance of pavement structures.

B. References

VonQuintus, H.L. and B.M. Killingsworth. *Design Pamphlet for the Determination of Design Subgrade in Support of the 1993 AASHTO Guide for the Design of Pavement Structures*. McLean, VA: Publication No. FHWA-RD-97-083. 1997.

Additional Resources:

Geotechnical Bulletin. *Plan Subgrades*. Ohio: Ohio Department of Transportation Division of Planning. 2003.

Mayne, P.W., B.R. Christopher, and J. DeJong. *Subsurface Investigation*. Washington, DC: National Highway Institute Federal Highway Administration, Report No. FHWA-NHI-01031, U.S. DOT. 2002.

Skok, E.L., E.N. Johnson, and M. Brown. *Special practices for design and construction of subgrades in poor, wet, and/or saturated soil condition*. Minnesota: Report No. MN/RC-2003-36, Minnesota Department of Transportation. 2003.

Pavement Systems

A. General Information

This section addresses the importance of pavement foundations and the potential for pavement problems due to deficient foundation support.

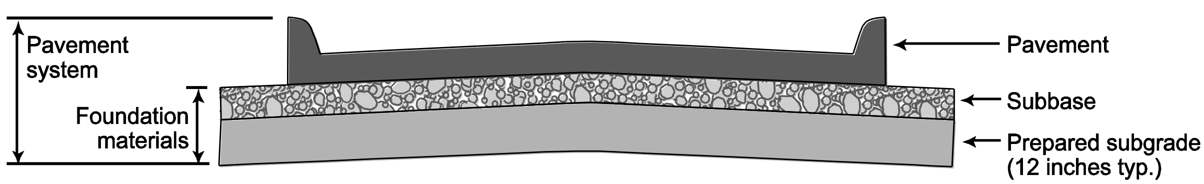
1. **Pavement System:** Consists of the pavement and foundation materials, which are layers of subbase, and subgrade (see Figure 6C-1.01). Failure to properly design or construct any of these components often leads to reduced serviceability or premature failure of the system.

2. **Pavement Materials:** Consist of flexible or rigid pavements, typically HMA or PCC, respectively, or a composite of the two.

3. **Subbase:** Consists of the granular materials underlying the pavement and above the subgrade layer.

4. **Subgrade:** Consists of the naturally occurring material on which the road is built, or the imported fill material used to create an embankment on which the road pavement is constructed. Subgrades are also considered layers in the pavement design, with their thickness assumed to be infinite and their material characteristics assumed to be unchanged or unmodified. Prepared subgrade is typically the top 12 inches of subgrade.

Figure 6C-1.01: Pavement System Cross-section



B. Pavement Support

The prepared subgrade is the upper portion (typically 12 inches) of a roadbed upon which the pavement and subbase are constructed. Pavement performance is expressed in terms of pavement materials and thickness. Although pavements fail from the top, pavement systems generally start to deteriorate from the bottom (subgrade), which often determines the service life of a road. Subgrade performance generally depends on two interrelated characteristics:

1. **Load-bearing Capacity:** The ability to support loads is transmitted from the pavement structure, which is often affected by degree of compaction, moisture content, and soil type.

2. **Volume Changes of the Subgrade:** The volume of the subgrade may change when exposed to excessive moisture or freezing conditions.

In determining the suitability of a subgrade, the following factors should be considered:

- General characteristics of the subgrade soil
- Depth to bedrock
- Depth to water table
- Compaction that can be attained in the subgrade
- CBR values of uncompacted and compacted subgrades
- Presence of weak or soft layers or organics in the subsoil
- Susceptibility to detrimental frost action or excessive swell

C. Pavement Problems

There are a number of ways that a pavement section can fail as well as many mechanisms, which lead to distress and failure.

1. Pavement Failures:

a. Structural Failure: Occurs when a collapse of the entire structure or a breakdown of one or more of the pavement components renders the pavement incapable of sustaining the loads imposed on its surface.

b. Functional Failure: Occurs when the pavement, due to its roughness, is unable to carry out its intended function without causing discomfort to drivers or passengers or imposing high stresses on vehicles.

2. Foundation Failures: The cause of these failure conditions may be due to inadequate maintenance, excessive loads, climatic and environmental conditions, poor drainage leading to poor subgrade conditions, non-uniform support of the surface layer, poor subgrade soil, and disintegration of the component materials. Utility cuts through existing pavements also result in premature pavement failure if not properly restored. Excessive loads, excessive repetition of loads, and high tire pressures can also cause either structural or functional failures.

Pavement failures may occur due to the intrusion of subgrade soils into the granular subbase, which results in inadequate drainage and reduced stability. Distress may also occur due to excessive loads that cause a shear failure in the subgrade, subbase, or surface layer. Other causes of failures are surface fatigue and excessive settlement, especially differential settlement of the subgrade. Volume change of subgrade soils due to wetting and drying, freezing and thawing, or improper drainage may also cause pavement distress. Inadequate drainage of water from the subbase and subgrade is a major cause of pavement problems. If the subgrade is saturated, excess pore pressures will develop under traffic loads, resulting in subsequent softening of the subgrade. Under traffic (dynamic) loading, fines can be pumped up into the subbase layers.

Improper construction practices may also cause pavement distress. Wetting of the subgrade during construction may permit water accumulation and subsequent softening of the subgrade in the rutted areas after construction is completed. Use of dirty aggregates or contamination of the subbase aggregates during construction may produce inadequate drainage, instability, and frost susceptibility. Reduction in design thickness during construction due to insufficient subgrade preparation may result in undulating subgrade surfaces, failure to place proper layer thicknesses, and unanticipated loss of subbase materials due to subgrade intrusion. A major cause of pavement deterioration is inadequate Quality Control/Quality Assurance (QA/QC) of pavement materials and pavement surface during construction. The following are the some of the significant causes leading to pavement distress and failure:

a. **Poor Soils:** Poor soils can seriously impede construction of adequate subgrades, as well as affect the long-term performance of a pavement during its service life. In use as subgrades, these soils often lack the strength and stability necessary to support trucks hauling construction materials, which forces project delays and adds costs. Special problem soil conditions include frost heave-susceptible soils, swelling or expansive soils, and collapsible soils.

   Highly compressible (very weak) soils are susceptible to large settlements and deformations with time that can have a detrimental effect on pavement performance. Highly compressible soils are very low in density, saturated, and are usually silts, clays, peat, organic alluvium, or loess. If these compressible soils are not treated properly, large surface depressions with random cracking can develop. The surface depressions can allow water to pond on the pavement's surface and more readily infiltrate the pavement structure, compounding a severe problem. More importantly, the ponding of water will create a safety hazard to the traveling public during wet weather. The selection of a particular treatment technique for poor soils is discussed in Section 6H-1 - Foundation Improvement and Stabilization.

   As with highly compressible soils, collapsible soils can lead to significant localized settlement of the pavement. Collapsible soils are very low-density silt-type soils, usually alluvium or wind-blown (loess) deposits, and are susceptible to sudden decreases in volume when wetted. Often, their unstable structure has been cemented by clay binders or other deposits, which will dissolve upon saturation, allowing a dramatic decrease in volume. Native subgrades of collapsible soils need to be soaked with water prior to construction and rolled with heavy compaction equipment. In some cases, residual soils may also be collapsible due to leaching of colloidal and soluble materials. If pavement systems are to be constructed over collapsible soils, special remedial measures may be required to prevent large-scale cracking and differential settlement.

   Swelling or expansive soils are susceptible to volume change (shrink and swell) with seasonal fluctuations in moisture content. The magnitude of this volume change is dependent on the type of soil (shrink-swell potential) and its change in moisture content. A loss of moisture will cause the soil to shrink, while an increase in moisture will cause it to expand or swell. This volume change of clay-type soils can result in longitudinal cracks near the pavement's edge and significant surface roughness (varying swells and depressions) along the pavement's length. Expansive soils are a significant problem in many parts of the United States and are responsible for premature maintenance and rehabilitation. Expansive soils are especially a problem when deep cuts are made in a dense (over-consolidated) clay soil.

b. **Utility Cuts:** The impact of utility cuts on pavement performance has been a concern of public agencies for many years. In large cities, thousands of utility cuts are made every year. These cuts are made to install, inspect, or repair buried facilities (See Chapter 9 - Utilities).

   The results of studies conducted by public agencies show that the presence of utility cuts lower measured pavement condition scores (indexes) compared to pavements of the same age with no utility cuts. The link between the presence of utility cuts and accelerated pavement deterioration is understood by most agencies.

   The resulting reduction in pavement life, despite high-quality workmanship repairing the cut can be explained by the trenching operation. The process of opening the trench causes sagging or slumping of the trench sides as the lateral support of the soil is removed. This zone of weakened pavement adjacent to the utility cut (known as the zone of influence) can fail more rapidly than other parts of the pavement. This can be observed in the field by the presence of fatigue (alligator) cracking occurring around the edges of the cut or spalling around the cut edges.

c. Transition Between Cuts and Fills: The alignment for many roadway projects does not always follow the site topography, and consequently a variety of cuts and fills will be required. The geotechnical design of the pavement will involve additional special considerations in cut-and-fill areas. Attention must also be given to transition zones (e.g., between a cut and an at-grade section) because of the potential for non-uniform pavement support and subsurface water flow.

The main additional concern for cut sections is drainage, as the surrounding site will be sloping toward the pavement structure; and the groundwater table will generally be closer to the bottom of the pavement section in cuts. Stabilization of moisture-sensitive natural foundation soils may also be required. Stability of the cut slopes adjacent to the pavement will also be an important design issue, but one that is treated separately from the pavement design itself.

The embankments for fill sections are constructed from compacted material, and in many cases, this construction results in a higher-quality subgrade than the natural foundation soil. In general, drainage and groundwater issues will be less critical for pavements on embankments, although erosion of side slopes from pavement runoff may be a problem, along with long-term infiltration of water. The primary additional concern for pavements in fill sections will be the stability of the embankment slopes and settlements, either due to compression of the embankment itself or to consolidation of soft foundation soils beneath the embankment. This is usually evaluated by the geotechnical unit as part of the roadway embankment design (see Section 6D-1 - Embankment Construction).

d. Foundation Non-uniformity: Non-uniform subgrade/subbase support increases localized deflections and causes stress concentrations in the pavement, which can lead to premature failures, fatigue cracking, faulting, pumping, rutting, and other types of pavement distresses for rigid and flexible pavement systems. Some recognized direct causes of subgrade/subbase non-uniformity include the following.

- Expansive soils
- Differential frost heave and subgrade softening
- Non-uniform strength and stiffness, due to variable soil type, moisture content, and density
- Pumping and rutting
- Cut/fill transitions
- Poor grading

Some techniques to overcome these subgrade deficiencies are:

- Moisture-density control during construction
- Proper soil identification and placement
- Over-excavation and replacement with select materials
- Mechanical and chemical soil stabilization
- Onsite soil mixing to produce well-graded composite materials
- Good grading techniques (e.g., uniform compaction energy/lift thickness)
- Waterproofing of the subgrade and control of moisture fluctuations

Although emphasis is placed on subgrade stiffness (i.e., modulus of subgrade reaction, k) for designing PCC thickness, performance monitoring suggests that uniformity of stiffness is the key for ensuring long-term performance. Because of the relatively high flexural stiffness of PCC pavements, the subgrade does not necessarily require high strength, but the subgrade/subbase should be uniform with no abrupt changes in degree of support. The uniformity has a significant influence on the stress intensity and deflection of the pavement layer, and the magnitude of stresses in the upper pavement layer depends on a combination of traffic loads and uniformity of subgrade support. Non-uniform stiffness and the resulting stress intensity contribute to fatigue cracking and differential settlement (deflection) in the pavement layer, and eventually to an uneven pavement surface. This uneven surface causes a rough ride for traffic and contributes to early pavement deterioration and high maintenance costs.

e. Poor Moisture Control: Pavements are strongly influenced by moisture and other environmental factors. Water migrates into the pavement structure through a combination of surface infiltration (e.g., through cracks in the surface layer), edge inflows, and from the underlying groundwater table (e.g., via capillary potential in fine-grained foundation soils). In cold environments, the moisture may undergo seasonal freeze/thaw cycles. Moisture within the pavement system nearly always has detrimental effects on pavement performance. It reduces the strength and stiffness of the pavement foundation materials, promotes contamination of coarse granular material due to fines migration, and can cause swelling (e.g., frost heave and/or soil expansion) and subsequent consolidation. Moisture can also introduce substantial spatial variability in the pavement properties and performance, which can be manifested either as local distresses like potholes, or more globally as excessive roughness. The design of the geotechnical aspects of pavements must consequently focus on the selection of moisture-insensitive, free-draining subbase materials, stabilization of moisture-sensitive subgrade soils, and adequate drainage of any water that does infiltrate into the pavement system.

To avoid moisture-related problems, a major objective in pavement design should seek to prevent the subbase, subgrade, and other susceptible paving materials from becoming saturated, or even exposed to constantly high-moisture levels. The three common approaches for controlling or reducing the problems caused by moisture include:

- Preventing moisture from entering the pavement system.
- Using materials and design features that are insensitive to the effects of moisture.
- Quickly removing the moisture that enters the pavement system.

No single approach can completely negate the effects of moisture on the pavement system under heavy traffic loading over many years. For example, it is practically impossible to completely seal the pavement, especially from moisture that may enter from the sides or beneath the pavement section. While materials can be incorporated into the design which are insensitive to moisture, this approach is often costly and in many cases not feasible (e.g., may require replacing the subgrade). Drainage systems also add costs to the road, as maintenance is required to maintain drainage systems as well as to seal systems for effective performance over the life of the system. Thus, it is often necessary to employ all approaches in combination for critical design situations.

Embankment Construction

A. General Information

Quality embankment construction is required to maintain smooth-riding pavements and to provide slope stability. Proper selection of soil, adequate moisture control, and uniform compaction are required for a quality embankment. Problems resulting from poor embankment construction have occasionally resulted in slope stability problems that encroach on private property and damage drainage structures. Also, pavement roughness can result from non-uniform support. The costs for remediation of such failures are high.

Soils available for embankment construction in Iowa generally range from A-4 soils (ML, OL), which are very fine sands and silts that are subject to frost heave, to A-6 and A-7 soils (CL, OH, MH, CG), which predominate across the state. The A-6 and A-7 groups include shrink/swell clayey soils. In general, these soils rate from poor to fair in suitability as subgrade soils. Because of their abundance, economics dictate that these soils must be used on the projects even through they exhibit shrink/swell properties. Because these are marginal soils, it is critical that the embankments be placed with proper compaction and moisture content, and in some cases, stabilization (see Section 6H-1 - Foundation Improvement and Stabilization).

Soils for embankment projects are identified during the exploration phase of the construction process. Borings are taken periodically along the proposed route and at potential borrow pits. The soils are tested to determine their engineering properties. Atterberg limits are determined and in-situ moisture and density are compared to standard Proctor values. However, it is impossible to completely and accurately characterize soil profiles because of the variability between boring locations. It is necessary for field staff and contractors to be able to recognize that soil changes have occurred and make the proper field adjustments.

Depending on roller configuration, soil moisture content, and soil type, soils may be under- or over-compacted. If soil lifts are too thick, the “Oreo cookie effect” may result, where only the upper part of the lift is being compacted. If the soils are too wet, over-compaction from hauling equipment can occur with resultant shearing of the soil and building in shear planes within the embankment, which can lead to slope failure.

Construction with soil is one of the most complicated procedures in engineering. In no other field of engineering are there so many variables as to the material used for construction. It is also widely recognized that certain soils are much more suitable for some construction activities than others.

A general understanding of soil and its different properties is essential for building a quality embankment. The engineering properties of a soil can vary greatly from gravel to clays. In order to build a quality embankment, the specific properties of the soil being used must be understood in order to make proper field judgments.

Ongoing debate exists among practitioners in geotechnical engineering about whether to compact soil wet-of-optimum-moisture content or dry-of-optimum moisture content. There is no decisive answer to this question. The only correct answer is that the ideal moisture content depends on material type and the desired characteristics (which often are competing) of the embankment. Strength, stability, density, low permeability, low shrink/swell behavior, and low collapsibility are all desired outcomes of a quality embankment.

Strength is obviously a desirable characteristic and is a function of many factors but can be directly related to moisture content. The U.S. Army Corps of Engineers (USACE) used the California Bearing Ratio (CBR) as an efficient measurement of strength in cohesive soils. The USACE reports, “the unsoaked CBR values are high on the dry side of optimum, but there is a dramatic loss in strength as molding moisture content is increased” (Ariema and Butler 1990; Atkins 1997). Hilf (1956) produced the same results from tests using penetration resistance as a measure of strength. When a soil is in a dry state, it exhibits high strength due to an appreciable inter-particle, attractive force created by high curvature of the menisci between soil particles. However, further wetting greatly reduces this friction strength by lubrication of the soil particles. Alternatively, in cohesionless soils, the strength is not as significantly affected by an increase in moisture, due to its high hydraulic conductivity.

Stability is a second desirable characteristic. However, stability cannot be defined as one characteristic. There is stability related to strength, which reacts to moisture contents described above; and there is also volumetric stability. When dealing with highly plastic clays, this is an extremely important factor since these clays exhibit shrink/swell behavior with a change in moisture content. Swelling of clays causes more damage in the United States than do the combined effects of all other natural disasters. It is general practice when dealing with fat clays to place the fill wet of optimum. This basically forces the clay to swell before compacting it in the embankment. Moisture content becomes important in cohesionless materials with respect to volumetric stability when the bulking phenomenon is considered. At the bulking moisture content a cohesionless soil will undergo volumetric expansion, or “bulk” (see Section 6A-2 - Basic Soils Information). Additionally, the material will exhibit apparent cohesion, and compaction cannot be achieved. Therefore, in terms of volumetric stability, truly cohesionless materials should be compacted when dry or saturated.

Density is perhaps the characteristic most widely associated with embankment construction. The Proctor test is the most widely used laboratory test to determine maximum dry density and optimum moisture content of cohesive soils as a function of compaction energy. However, the standard Proctor test is not a valid test for all cohesionless soils. Cohesionless soils require the relative density test to determine a maximum and minimum dry density.

Once the desirable material properties have been identified, the next process in building a quality embankment is the correct placement of the soil. The importance of soil preparation before rolling is not adequately appreciated. Blending of the soil to achieve a homogeneous composition and moisture content is essential for quality embankment construction. Proper roller identification and use are also essential. Not all rollers are adequate for all soil types. Sheepsfoot rollers are ideal for cohesive soils, while vibratory rollers must be used on cohesionless materials. Inter-grade soils require inter-grade rollers, such as a vibratory sheepsfoot (Chatwin et al. 1994).

B. Site Preparation

1. **Clearing and Grubbing:** The site should be prepared by first clearing the area of vegetation, fencing rubbish, and other objectionable materials.

2. **Stripping, Salvaging, and Spreading Topsoil:** The site should be mowed and any sod shredded by shallow plowing or blading and thorough disking so the soil can be easily placed in a thin layer over areas to be covered.

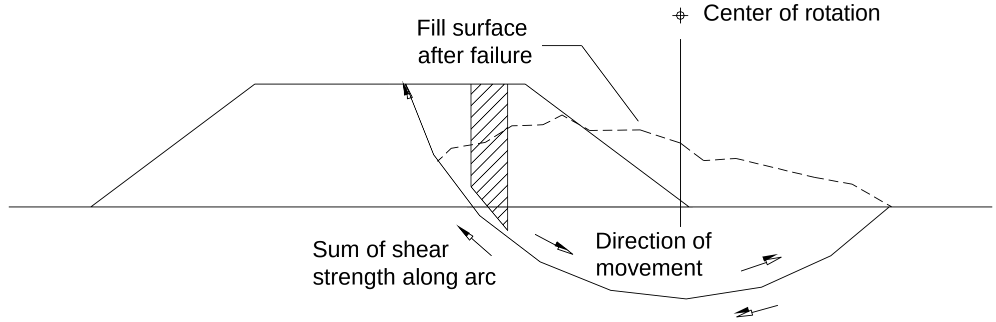
   An adequate amount of topsoil should be removed from the upper 12 inches of existing onsite topsoil to allow a finished grade of 8 inches of salvaged or amended topsoil. The topsoil may be moved directly to an area where it is to be used or may be stockpiled for future use. If existing topsoil lacks adequate organic content, off-site soil may be required, or existing topsoil may be blended with compost (see SUDAS Specifications Section 2010, 2.01 for proper blending ratios).

C. Design Considerations

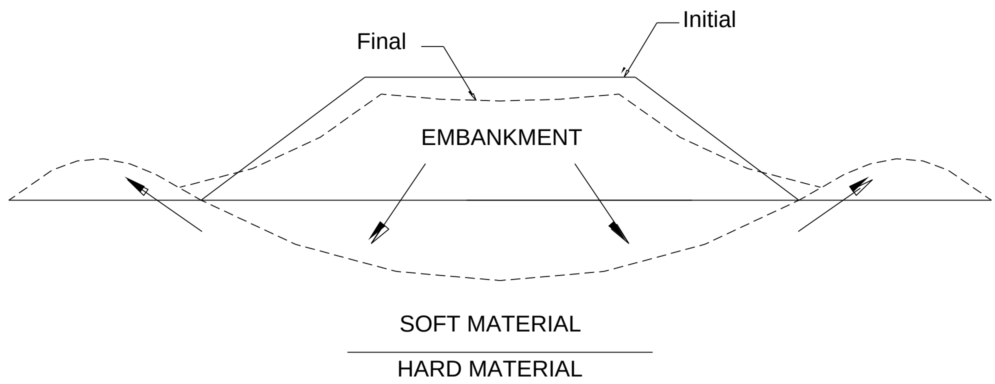
1. **Slope Stability Evaluation:** Foundation soils and embankments provide adequate support for roadways and other transportation infrastructure if the additional stress from traffic loads and geo-structures does not exceed the shear strength of the embankment soils or underlying strata (Ariema and Butler 1990). Overstressing the embankment or foundation soil may result in rotational, displacement, or translatory failure, as illustrated in Figure 6D-1.01.

   Factors of safety are used to indicate the adequacy of slope stability and play a vital role in the rational design of engineered slopes (e.g. embankments, cut slopes, landfills). Factors of safety used in design account for uncertainty and thus guard against ignorance about the reliability of the items that enter into the analysis, such as soil strength parameter values, pore water pressure distributions, and soil stratigraphy (Abramson et al. 2002). As with the design of other geostructures, higher factors of safety are used when limited site investigation generates uncertainty regarding the analysis input parameters. Investment in more thorough site investigation and construction monitoring, however, may be rewarded by acceptable reduction in the desired factor of safety. Typically minimum factors of safety for new embankment slope design range from 1.3 to 1.5. Factors of safety against slope instability are defined considering the likely slope failure mode and the strength of slope soils.

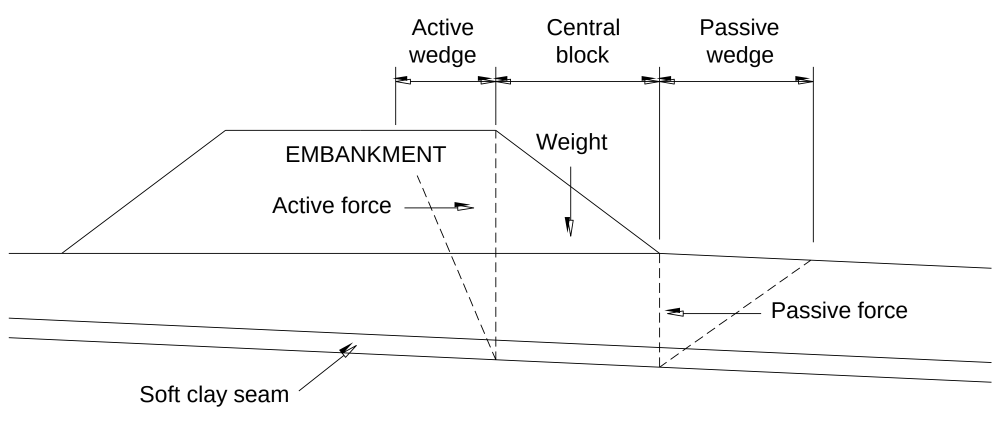
Figure 6D-1.01: Typical Embankment Failures



ROTATIONAL FAILURE



DISPLACEMENT FAILURE



TRANSLATORY FAILURE

Source: Ariema and Butler 1990

2. **Causes of Slope Instability:** Stable slopes are characterized by a balance between the gravitational forces tending to pull soils downslope and the resisting forces comprised of soil shear strength. The state of temporary equilibrium may be compromised when the slope is subject to de-stabilizing forces. The factors affecting slope stability may include those that increase the gravitational force (e.g. slope geometry, undercutting, surcharging) or those that reduce soil shear strength (e.g. weathering, pore water pressure, vegetation removal) (Chatwin et al. 1994).

3. **Slope Stability Problems in Iowa:** Slope instability poses problems for roadway systems in Iowa. Failures occur on both new embankments and cut slopes. The failures occur because identifying factors that affect stability at a particular location, such as soil shear strength parameter values, ground water surface elevations, and negative influences from construction activities, are often difficult to discern and measure. Hazard identification is a cornerstone of landslide hazard mitigation (Spiker and Gori 2003). Once a failure occurs or a potential failure is identified (i.e. low factor of safety), roadway agencies need information and knowledge of which methods of remediation will be most effective to stabilize the slope. Ideally, these stability problems can be discovered and addressed before a slope failure occurs.

   Approximately 50% of slope remediation projects involve changes in slope geometry (in effect, creating a stability berm). The design and construction of stability berms have historically been a simple and effective option of departments of transportation for preserving transportation infrastructure.

4. **Stabilization Methods:** A number of methods are available to stabilize slopes, including re-grading to flatten the slope; construction of stability berms; the use of lightweight fill, geofoam or shredded tires to reduce the load; and structural reinforcing methods such as geosynthetic reinforcements, stone columns, rammed aggregate piers, soil nailing, and piles. Additional information on such methods to address slope instability can be found in Section 6H-1 - Foundation Improvement and Stabilization.

D. Equipment

Table 6D-1.01 provides suggested compaction equipment and compacted lift thicknesses for coarse- and fine-grained soils, according to the USCS and AASHTO soil classification systems.

Table 6D-1.01: Recommended Field Compaction Equipment

Soil	First Choice	Second Choice	Comment
Rock fill	Vibratory	Pneumatic	--
Plastic soils, CH, MH	Sheepsfoot or pad foot	Pneumatic	Thin lifts usually needed
Low-plasticity soils, CL, ML	Sheepsfoot or pad foot	Pneumatic, vibratory	Moisture control often critical for silty soils
Plastic sands and gravels, GC, SC	Vibratory, pneumatic	Pad foot	--
Silty sands and gravels, SM, GM	Vibratory	Pneumatic, pad foot	Moisture control often critical
Clean sands, SW, SP	Vibratory	Impact, pneumatic	--
Clean gravels, GW, GP	Vibratory	Pneumatic, impact, grid	Grid useful for over-size particles

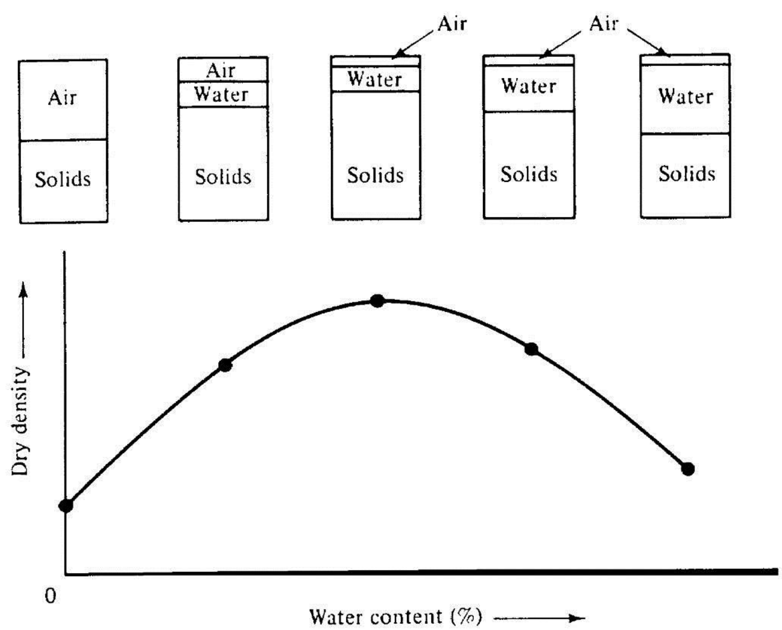
Source: Rollings and Rollings 1996

E. Density

Maximum Dry Density: Compaction requirements are measured in terms of the dry density of the soil. The expected value for dry density varies with the type of soil being compacted. For example, a clay soil may be rolled many times and not reach 125 pcf, whereas a granular soil may have a dry density above this value without any compactive effort. Therefore, a value for the maximum possible dry density must be established for each soil (Atkins 1997).

For any compactive effort, the dry density of a soil will vary with its water content. A soil compacted dry will reach a certain dry density. If compacted again with the same compactive effort, but this time with water in the soil, the dry density will be higher, since the water lubricates the grains and allows them to slide into a denser structure. Air is forced out of the soil, leaving more space for the soil solids, as well as the added water. With even higher water content, a still greater dry density may be reached since more air is expelled. However, when most of the air in the mixture has been removed, adding more water to the mixture before compaction results in a lower dry density, as the extra water merely takes the place of some of the soil solids. This principle is illustrated in Figure 6D-1.02.

Figure 6D-1.02: Variation of Dry Density with Water Content



The first step in compaction control is to determine the maximum dry density that can be expected for a soil under a certain compactive effort, and the water content at which this density is reached. These are obtained from a compaction curve, as discussed in Section 6A-2 - Basic Soils Information. The compaction curve is also called a moisture-density curve or a Proctor curve (named after the originator of the test). The curve is plotted from the results of the compaction test. Dry density is plotted against water content, and a curve is drawn through the test points. The top of the curve represents the maximum dry density for the soil with the test compactive effort and the corresponding water content, which is called the optimum water content (W_O).

F. Compaction

In-situ soils used as subgrades for the construction of roadway pavements or other structures and transported soils used in embankments or as leveling material for various types of construction projects are usually compacted to improve their density and other properties. Increasing the soil's density improves its strength, lowers its permeability, and reduces future settlement.

The evaluation of the density reached as a result of compactive efforts with rollers and other types of compaction equipment is the most common quality-control measurement made on soils at construction sites. The density of the soil as compacted is measured and compared to a density goal for that soil, as previously determined in laboratory tests. The moisture-density relationships for fine-grained (cohesive) soils and coarse-grained (cohesionless) soils are discussed in Section 6A-2 - Basic Soils Information.

1. **Compaction of Fine-grained Soils:** The compaction method for a fine-grained soil is entirely different than that for a coarse-grained soil. The reason is that fine-grained soils possess cohesion. It should be remembered that the finer fraction of the fine-grained soils exists in a colloidal state, and all colloids possess cohesion. The mineral grains of a cohesive soil are not in physical contact, as they are in a coarse-grained soil. Every grain is surrounded by a blanket of water, whose molecules are electrically bonded to the grains. This blanket of water isolates the grains and prevents them from being in physical contact with adjacent grains (Duncan 1992).

   The degree to which a fine-grained soil can be compacted is almost wholly dependent on the in-situ moisture content of the soil. The moisture content that corresponds to the maximum degree of compaction (under a given compaction energy) is called the optimum moisture content. The approximate optimum moisture content of several soil groups is given in Table 6D-1.02.

Table 6D-1.02: Maximum Dry Density and Optimum Moisture Content
(Typical for Standard Compaction Energy)

AASHTO Classification	Maximum Dry Density (pcf)	Moisture Content (%)
A-1	115-135	7-15
A-2	110-135	9-18
A-3	110-115	10-18
A-4	95-130	10-20
A-5	85-100	15-30
A-6	95-120	10-25
A-7	85-115	15-30

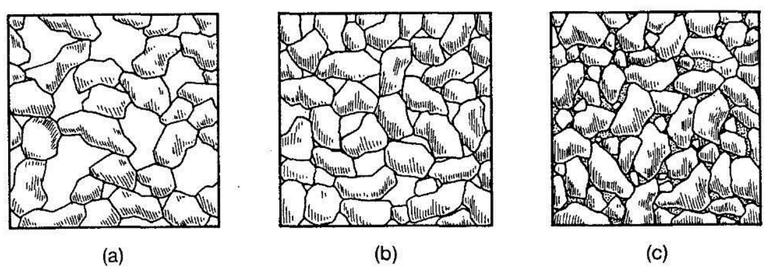
2. **Compaction of Coarse-grained Soils:** The method behind why compaction works for a coarse-grained soil is entirely different than that for a fine-grained soil. Coarse-grained soils exist by their very nature in inter-granular contact, much like a bucket of marbles. The way these grains are arranged within the mass and the distribution of particle size throughout the mass, will ultimately determine the density, stability, and load-bearing capacity of that particular soil (Duncan 1992).

   The honeycombed structure shown in Figure 6D-1.03a is representative of very poor inter-granular seating. Such a structure is inherently unstable and can collapse suddenly when subjected to shock or vibration. The stability and load-bearing capacity of this type of soil will be improved by compaction because of the resulting rearrangement in inter-granular seating. With sufficient compaction, this structure will take on the characteristics of the arrangement shown in Figure 6D-1.03c.

The arrangement of particles shown in Figure 6D-1.03b provides maximum inter-granular contact, but there are insufficient fines to lock the larger particles in place. Compaction of this type of arrangement is ineffective, since neither additional particle contact nor additional stability can be achieved. This soil is inherently stable, however, when it is laterally restrained, and demonstrates good load-bearing characteristics. When insufficiently restrained, however, this soil will be free to move laterally, in which case there is a pronounced loss in stability and load-bearing characteristics.

The arrangement of particles shown in Figure 6D-1.03c not only provides maximum inter-granular contact, but also inherent stability. This very important property of stability is due to the inclusion of fines in the spaces between the larger particles. One cautionary note must be made concerning fines: too many fines are detrimental to the mix because they may separate the larger grains, thereby destroying the inter-granular contact between them. In this instance, the larger grains are more or less floating in a sea of fines.

Figure 6D-1.03: Inter-granular Seating and Gradation of Coarse-grained Particles



(a) Poorly graded, poorly seated particles
(b) Poorly graded, but well-seated
(c) Well-graded and well-seated particles

The inter-granular seating of a coarse-grained soil can be improved by the process of compaction. Particle distribution can be improved by the physical addition and mixing of fines into the soil. Both of these separate actions increase the density of the soil. Density is a function of the amount of voids contained within a given volume of soil. The potential for a soil to be further densified depends upon how much of a reduction can be made in the void ratio. This reduction is not without limit. Every mixture of granular material inherently has a minimum void ratio (maximum density), and for a given mixture, this ratio cannot be changed. Once a soil has been compacted to its maximum density, continued efforts at compaction will only result in the crushing of the individual grains as described in Section 6A-2 - Basic Soils Information.

Compaction of coarse-grained soils is usually considered to be adequate when the relative density of the soil in place is no less than some specified percentage of its maximum possible density. Relative density is a term used to numerically compare the density of an in-place natural or compacted soil, with the densities represented by the same soil in the extreme states of looseness and denseness, as described in Section 6A-2 - Basic Soils Information.

3. **Compaction of Mixed-grained Soils:** Natural deposits of soil frequently contain gravel, sand, silt, and clay in various proportions. Such soils are referred to as mixed-grained. Soils that are mixed-grained will, in all likelihood, exhibit some of the characteristics of both coarse-grained and fine-grained soils. The deciding factor as to whether a particular soil should be compacted according to coarse-grained or fine-grained requirements is that of cohesion (true or apparent) (Duncan 1992).

a. **Soils that do not Exhibit any Measurable Cohesion:** Treat as coarse grained soil; base compaction on the relative density.

b. **Soils that do Exhibit Measurable Cohesion:** Treat as fine-grained soil; base compaction on the Proctor Density Test.

c. **Inter-grade Soils:** Conduct both Relative Density and Proctor Density Tests; base compaction on the test method yielding the highest maximum density.

G. Embankment Soils

SUDAS classifies Iowa cohesive soils into select subgrade materials, suitable soils, or unsuitable soils, depending on soil index properties and Proctor test results. See Section 6E-1 - Subgrade Design and Construction for more information.

1. **Select Subgrade Soils:** Select materials (see Section 6E-1 - Subgrade Design and Construction) or subgrade treatments (see Section 6H-1 - Foundation Improvement and Stabilization) may be used in the prepared subgrade (the top 12 inches immediately below the pavement or subbase, if present) to provide adequate volumetric stability, low frost potential, and good bearing capacity as it relates to the California Bearing Ratio (CBR $\geq$ 10).

2. **Suitable Soils:** Suitable soils are used throughout the fill and under the prepared subgrade. Suitable soils may be used in the prepared subgrade if they meet the requirements of select subgrade soils or are stabilized to meet those requirements (i.e., CBR $\geq$ 10). Suitable soils must meet all of the following conditions:

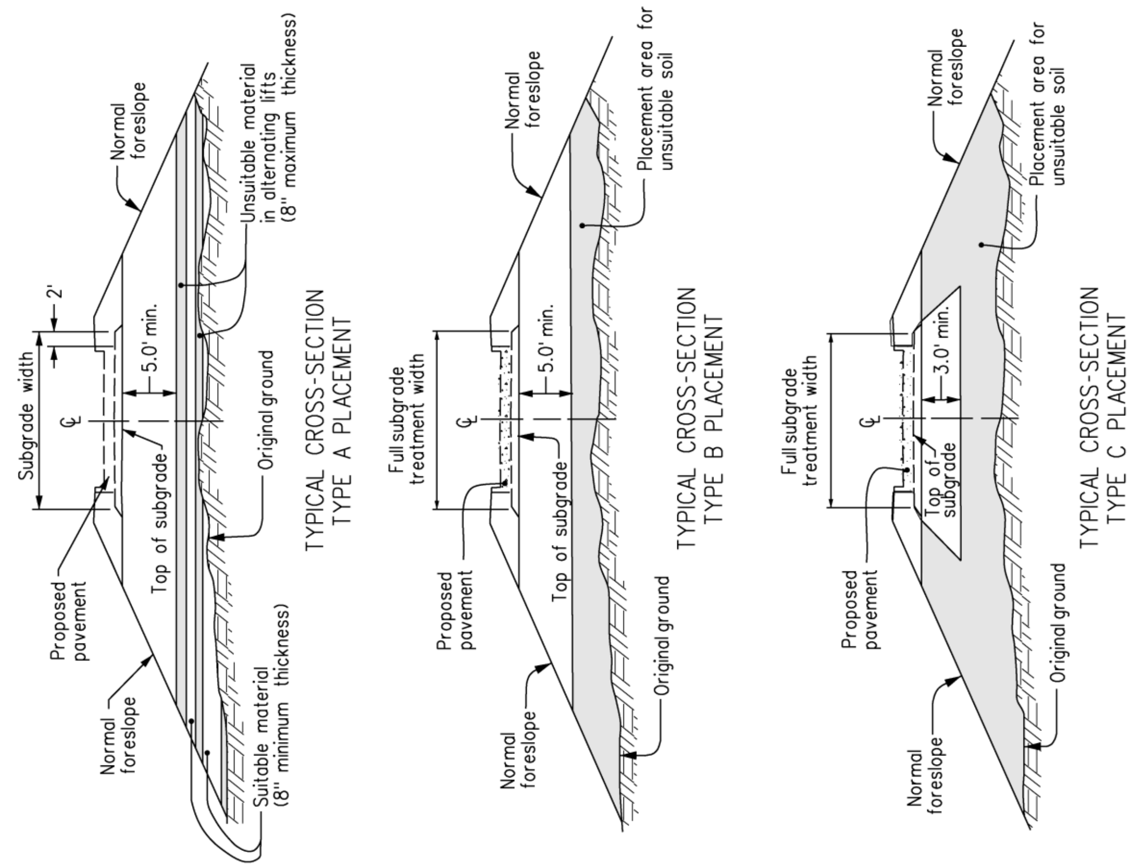
   a. Standard Proctor Density $\geq$ 95 pcf

   b. Group index < 30 (AASHTO M 145)

3. **Unsuitable Soils:** The SUDAS Specifications do not allow use of unsuitable soils in the right-of-way. However, there may be situations where the Engineer might consider the placement of unsuitable soils in the right-of-way. The Iowa DOT allows this placement. Figure 6D-1.04, modified from Iowa DOT Standard Road Plan RL-1B, illustrates Iowa DOT's guidance for the use of unsuitable soils in an urban embankment section.

Figure 6D-1.04: Placement of Unsuitable Soils

Placed 4 feet below subgrade in fills outside curbline	1. Broken PCC in 6 inch sizes or smaller (pulverized HMA may be used as subgrade replacement)
Type A Placement Place in layers (8 inch max. thickness) 5 feet below subgrade and 2 feet outside curbline in fills. Provide alternate layers of suitable soils or soils other than A-7 or A-5 containing 3% or more carbon)	1. Shale 2. A-7-5 or A-5 soils having a density greater than 86 pcf but less than 95 pcf (ASTM D 698 Standard Proctor Density).
Type B Placement Placed 5 feet below subgrade and outside curbline in fills	1. A-7-6 (Plasticity index of 30 or greater) 2. Residual clays (overlying bedrock) regardless of classification.
Type C Placement Placed 3 feet below subgrade in fills (may be placed 2 feet outside of curbline).	1. All soils other than A-7-5 or A-5 having a density of 95 pcf or less (ASTM D 698 Standard Proctor Density). 2. All soils other than A-7 or A-5 containing 3% or more carbon.
Slope dressing only	1. Peat or muck 2. Soils with a plasticity index of 35 or greater 3. A-7 or A-5 (AASHTO) having a density less than 85 pcf (ASTM D 698 Standard Proctor Density)



Source: Modified version of Iowa DOT's Standard Road Plan RL-1B.

H. Testing

Inherent to the quality construction of roadway embankments is the ability to measure soil properties to enforce quality control measures. In the past, density and moisture content have been the most widely measured soil parameters in conjunction with acceptance criteria.

1. **In-place Soil Density Requirements:** The Engineer must first establish the standard to which the field work must conform. This standard differs depending upon whether the soil is classified as coarse-grained, fine-grained, or inter-grade (Duncan 1992).

   a. **In-place Soil Density:** The SUDAS Specifications require 95% Standard Proctor Density for cohesive soils and 70% Relative Density for cohesionless soils. If different density requirements are warranted for a project, the Engineer must specify those modifications. As the default, SUDAS Specifications require moisture and density control for embankment construction. In lieu of moisture and density control, the Engineer may specify Type A compaction, which is roller walkout and does not require moisture and density testing.

   b. **Tests to Verify In-place Soil Density:** For these classifications of soil, the dry density of the in-place, compacted soil must be determined. There are three procedures whereby the wet density of the in-place soil can be readily determined in the field. Once the in-place wet density and the moisture content are known, the dry density can be easily computed. These procedures are described in the following ASTM Standards:
      1) **Density of Soil in Place by the Sand-cone Method (ASTM D 1556):** This method is generally limited to soil in an unsaturated condition. It is not recommended for soil that is soft or easily crumbled or for deposits where water will seep into the test hole.
      2) **Density and Unit Weight of Soil in Place by the Rubber Balloon Method (ASTM D 2167):** This method is not suitable for use with organic, saturated, or highly plastic soils. The use of this method will require special care with unbonded granular soils, soils containing appreciable amounts of coarse aggregate larger than 1½ inches, granular soils having a high void ratio, and fill materials having particles with sharp edges.
      3) **Density of Soil and Soil Aggregate in Place by Nuclear Methods (ASTM D 2922):** This method provides a rapid, non-destructive technique for the determination of in-place wet soil density. Test results may be affected by chemical composition, heterogeneity, and surface texture of the material being tested. The techniques also exhibit a spatial bias in that the apparatus is more sensitive to certain regions of the material being tested. Nuclear methods, of course, pose special hazards and require special care. The work must be done in strict conformance with all safety requirements and must be performed only by trained personnel.

2. **Field Control of Moisture Content:** SUDAS Specifications Section 2010 requires a moisture content of optimum moisture to 4% over optimum moisture. As discussed earlier, the moisture content may need to be modified, depending on the material type and desired characteristics. There are four general procedures whereby moisture content can be determined:

   a. Accurate results can be achieved by the laboratory analysis of samples using a drying oven according to AASHTO T 265. This method, however, may be too time consuming.

   b. Fast results can be obtained in the field with a portable moisture tester. This particular tester, which conforms to AASHTO T 217, provides for almost continuous monitoring of the moisture content because the test can usually be performed in three minutes or less.

   c. A microwave may be used for fine-grained soils, according to ASTM D 600.

d. A nuclear density unit may be used to provide an estimate of the moisture content, according to AASHTO T 239.

It is important that the moisture content of the soil be maintained as close to the target moisture content as can reasonably be expected during all stages of the compaction process. When the soil is too dry, the moisture content can be increased by sprinkling water over the surface, after which it must be thoroughly mixed into the soil to produce uniform moisture content throughout the mass. When the soil is too wet, the moisture content can be reduced by spreading the soil out, disking it, and letting it dry in the sun.

3. Strength and Stability of Compacted Soil: Two methods are used to determine the strength and stability of compacted soil.

a. California Bearing Ratio (CBR): This method is probably the most widely used. A subgrade generally requiring a CBR of 10 or greater is considered good and can support heavy loading without excessive deformation (see Section 6E-1 - Subgrade Design and Construction, for additional information). For reference, some typical values of CBR soils are shown in Table 6D-1.03.

b. Dynamic Cone Penetrometer (DCP) Index: This index, expressed in millimeters per blow, has been correlated to CBR for use in pavement design and evaluation, and is presented in ASTM Section B, Test Method No. 8. The correlation is advantageous because most flexible pavement design procedures are based on CBR. Several other DCP versus CBR relationships have been developed as well.

Table 6D-1.03: Typical CBR Values for Various Soils

Material Description	CBR
SC: clayey sand	10-20
CL: lean clays, sandy clays, gravelly clays	5-15
ML: silts, sandy silts	5-15
OL: organic silts, lean organic clays	4-8
CH: fat clays	3-5
MH: plastic silts	4-8
OH: fat organic clays	3-5

Source: Rollings and Rollings, 1996

Table 6D-1.04: Simple CBR Indicators of Wet Clay Soil

Material Description	CBR
Thumb penetration into the wet clay soil	
Easy	< 1
Possible	1
Difficult	2
Impossible	3+
A trace of a footprint left by a walking man	1

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Subgrade Design and Construction

A. General Information

The subgrade is that portion of the pavement system that is the layer of natural soil upon which the pavement or subbase is built. Subgrade soil provides support to the remainder of the pavement system. The quality of the subgrade will greatly influence the pavement design and the actual useful life of the pavement that is constructed. The importance of a good quality subgrade to the long term life of the pavement cannot be understated. As the pavement reaches design life, the subgrade will not have to be reconstructed in order to support the rehabilitated subgrade or the reconstructed pavement. In urban areas, subgrade basic engineering properties are required for design. This section summarizes the design and construction elements for subgrades.

B. Site Preparation

Site preparation is the first major activity in constructing pavements. This activity includes removing or stripping off the upper soil layer(s) from the natural ground. All organic materials, topsoil, and stones greater than 3 inches in size should be removed. Removal of surface soils containing organic matter is important not only for settlement, but also because these soils are often moisture-sensitive, they lose significant strength when wet and are easily disturbed under construction activities. Most construction projects will also require excavation or removal of in-situ soil to reach a design elevation or grade line.

C. Design Considerations

Subgrade soil is part of the pavement support system. Subgrade performance generally depends on three basic characteristics:

1. **Strength:** The subgrade must be able to support loads transmitted from the pavement structure. This load-bearing capacity is often affected by degree of compaction, moisture content, and soil type. A subgrade having a California Bearing Ratio (CBR) of 10 or greater is considered essential and can support heavy loads and repetitious loading without excessive deformation.

2. **Moisture Content:** Moisture tends to affect a number of subgrade properties, including load-bearing capacity, shrinkage, and swelling. Moisture content can be influenced by a number of factors, such as drainage, groundwater table elevation, infiltration, or pavement porosity (which can be affected by cracks in the pavement). Generally, excessively wet subgrades will deform under load.

3. **Shrinkage and/or Swelling:** Some soils shrink or swell, depending upon their moisture content. Additionally, soils with excessive fines content may be susceptible to frost heave in northern climates. Shrinkage, swelling, and frost heave will tend to deform and crack any pavement type constructed over them.

Pavement performance also depends on subgrade uniformity. However, a perfect subgrade is difficult to achieve due to the inherent variability of the soil and influence of water, temperature, and construction activities. Emphasis should be placed on developing a subgrade CBR of at least 10. Research has shown that with a subgrade strength of less than a CBR of 10, the subbase material will deflect under traffic loadings in the same manner as the subgrade. That deflection then impacts the pavement, initially for flexible pavements, but ultimately rigid pavements as well.

To achieve high-quality subgrade, proper understanding of soil properties, proper grading practices, and quality control testing are required. However, pavement design requirements and the level of engineering effort should be consistent with relative importance, size, and cost of design projects. Therefore, knowledge of subgrade soil basic engineering properties is required for design. These include soil classification, soil unit weight, coefficient of lateral earth pressure, and estimated CBR or resilient modulus. Table 6E-1.01 summarizes the suitability of different soils for subgrade applications, and Table 6E-1.02 gives typical CBR values of different soils depending on soil classification.

Table 6E-1.01: Suitability of Soils for Subgrade Applications

Subgrade Soils for Design	Unified Soil Classifications	Load Support and Drainage Characteristics	Modulus of Subgrade Reaction (k), psi/inch	Resilient Modulus (M_R), psi	CBR Range
Crushed Stone	GW, GP, and GU	Excellent support and drainage characteristics with no frost potential	220 to 250	Greater than 5,700	30 to 80
Gravel	GW, GP, and GU	Excellent support and drainage characteristics with very slight frost potential	200 to 220	4,500 to 5,700	30 to 80
Silty gravel	GW-GM, GP-GM, and GM	Good support and fair drainage, characteristics with moderate frost potential	150 to 200	4,000 to 5,700	20 to 60
Sand	SW, SP, GP-GM, and GM	Good support and excellent drainage characteristics with very slight frost potential	150 to 200	4,000 to 5,700	10 to 40
Silty sand	SM, non-plastic (NP), and >35% silt (minus #200)	Poor support and poor drainage with very high frost potential	100 to 150	2,700 to 4,000	5 to 30
Silty sand	SM, Plasticity Index (PI) <10, and <35 % silt	Poor support and fair to poor drainage with moderate to high frost potential	100 to 150	2,700 to 4,000	5 to 20
Silt	ML, >50% silt, liquid limit <40, and PI <10	Poor support and impervious drainage with very high frost value	50 to 100	1,000 to 2,700	1 to 15
Clay	CL, liquid limit >40 and PI >10	Very poor support and impervious drainage with high frost potential	50 to 100	1,000 to 2,700	1 to 15

Source: American Concrete Pavement Association; Asphalt Paving Association; State of Ohio; State of Iowa; Rollings and Rollings 1996.

D. Strength and Stiffness

Subgrade materials are typically characterized by their strength and stiffness. Three basic subgrade stiffness/strength characterizations are commonly used in the United States: California Bearing Ratio (CBR), modulus of subgrade reaction (k), and elastic (resilient) modulus. Although there are other factors involved when evaluating subgrade materials (such as swell in the case of certain clays), stiffness is the most common characterization and thus CBR, k-value, and resilient modulus are discussed here.

1. **California Bearing Ratio (CBR):** The CBR test is a simple strength test that compares the bearing capacity of a material with that of a well-graded crushed stone (thus, a high-quality crushed stone material should have a CBR of 100%). It is primarily intended for, but not limited to, evaluating the strength of cohesive materials having maximum particle sizes less than 0.75 inches. Figure 6E-1.01 is an image of a typical CBR sample.

Figure 6E-1.01: In-situ CBR

Source: ELE International

The CBR method is probably the most widely used method for designing pavement structures. This method was developed by the California Division of Highways around 1930 and has since been adopted and modified by numerous states, the U.S. Army Corps of Engineers (USACE), and many countries around the world. Their test procedure was most generally used until 1961, when the American Society for Testing and Materials (ASTM) adopted the method as ASTM D 1883, CBR of Laboratory-Compacted Soils. The ASTM procedure differs in some respects from the USACE procedure and from AASHTO T 193. The ASTM procedure is the easiest to use and is the version described in this section.

The CBR is a comparative measure of the shearing resistance of soil. The test consists of measuring the load required to cause a piston of standard size to penetrate a soil specimen at a specified rate. This load is divided by the load required to force the piston to the same depth in a standard sample of crushed stone. The result, multiplied by 100, is the value of the CBR. Usually, depths of 0.1 to 0.2 inches are used, but depths of 0.3, 0.4, and 0.5 inches may be used if desired. Penetration loads for the crushed stone have been standardized. This test method is intended to provide the relative bearing value, or CBR, of subbase and subgrade materials. Procedures are given for laboratory-compacted swelling, non-swelling, and granular materials. These tests are usually performed to obtain information that will be used for design purposes. The CBR value for a soil will depend upon its density, molding moisture content, and moisture content after soaking. Since the product of laboratory compaction should closely represent the results of field compaction, the first two of these variables must be carefully controlled during the preparation of laboratory samples for testing. Unless it can be ascertained that the soil being tested will not accumulate moisture and be affected by it in the field after construction, the CBR tests should be performed on soaked samples.

Relative ratings of supporting strengths as a function of CBR values are given in Table 6E-1.02.

Table 6E-1.02: Relative CBR Values for Subbase and Subgrade Soils

CBR (%)	Material	Rating
> 80	Subbase	Excellent
50 to 80	Subbase	Very Good
30 to 50	Subbase	Good
20 to 30	Subgrade	Very good
10 to 20	Subgrade	Fair-good
5 to 10	Subgrade	Poor-fair
< 5	Subgrade	Very poor

The higher the CBR value of a particular soil, the more strength it has to support the pavement. This means that a thinner pavement structure could be used on a soil with a higher CBR value than on a soil with a low CBR value. Generally, clays have a CBR value of 6 or less. Silty and sandy soils are next, with CBR values of 6 to 8. The best soils for road-building purposes are the sands and gravels whose CBR values normally exceed 10. Most Iowa soils rate fair-to-poor as subgrade materials.

The change in pavement thickness needed to carry a given traffic load is not directly proportional to the change in CBR value of the subgrade soil. For example, a one-unit change in CBR from 5 to 4 requires a greater increase in pavement thickness than does a one-unit change in CBR from 10 to 9.

2. **Resilient Modulus (M_R):** M_R is a subgrade material stiffness test. A material's M_R is actually an estimate of its modulus of elasticity (E). While the modulus of elasticity is stress divided by strain for a slowly applied load, M_R is stress-divided by strain for rapidly applied loads like those experienced by pavements. Flexible pavement thickness design is normally based on M_R. See Table 6E-1.01 for typical M_R values.

   The resilient modulus test applies a repeated axial cyclic stress of fixed magnitude, load duration, and cycle duration to a cylindrical test specimen. While the specimen is subjected to this dynamic cyclic stress, it is also subjected to a static confining stress provided by a triaxial pressure chamber. It is essentially a cyclic version of a triaxial compression test; the cyclic load application is thought to more accurately simulate actual traffic loading.

   The M_R is a slightly different measurement of somewhat similar properties of the soil or subbase. It measures the amount of recoverable deformation at any stress level for a dynamically loaded test specimen. Both measurements are indications of the stiffness of the layer immediately under the pavement.

   The environment can affect pavement performance in several ways. Temperature and moisture changes can have an effect on the strength, durability, and load-carrying of the pavement and roadbed materials. Another major environmental impact is the direct effect roadbed swelling, pavement blowups, frost heave, disintegration, etc. can have on loss of riding quality and serviceability. If any of these environmental effects have a significant loss in serviceability or ride quality during the analysis period, the roadbed soil M_R takes the environmental effects into account if seasonal conditions are considered.

The purpose of using seasonal modulus is to qualify the relative damage a pavement is subject to during each season of the year and treat it as part of the overall design. An effective road bed soil modulus is then established for the entire year which is equivalent to the combined effects of all monthly seasonal modulus values. AASHTO provides different methodology to obtain the effective M_R for flexible pavement only. The method that was selected for use in this manual was based on the determination of M_R values for six different climatic regions in the United States that considered the quality of subgrade soils.

Figure 6E-1.03: Resilient Modulus

Source: Federal Highway Administration

3. **Modulus of Subgrade Reaction (k, k_c):** This is a bearing test that rates the support provided by the subgrade or combination of subgrade and subbase. The k-value is defined as the reaction of the subgrade per unit of area of deformation and is typically given in psi/inch. Concrete pavement thickness design is normally based on the k-value. See Table 6E-1.01 for typical k-values.

   Modulus of subgrade reaction is determined with a plate bearing test. Details for plate bearing tests are found in AASHTO T 221 and AASHTO T 222 or ASTM D 1195 and ASTM D 1196.

   Several variables are important in describing the foundation upon which the pavement rests:

   a. **Modulus of Subgrade Reaction (k):** For concrete pavements, the primary requirement of the subgrade is that it be uniform. This is the fundamental reason for specifications on subgrade compaction. The k-value is used for thickness design of concrete pavements being placed on prepared subgrade.

   b. **Composite Modulus of Subgrade Reaction (k_c):** In many highway applications the pavement is not placed directly on the subgrade. Instead, some type of subbase material is used. When this is done, the k value actually used for design is a "composite k" (k_c), which represents the strength of the subgrade corrected for the additional support provided by the subbase.

4. Correlation of Strength and Stiffness Values:

a. Relationship of CBR and Dynamic Cone Penetrometer (DCP) Index: The dual mass Dynamic cone Penetrometer (DCP) is a method for estimating in-place stability from CBR correlations. As shown in Figure 6E-1.05, the dual mass DCP consists of an upper and lower 5/8 inch diameter steel shaft with a steel cone attached to one end. The cone at the end of the rod has a base diameter of 0.79 plus 0.01 inches. As an option, a disposable cone attachment can be used for testing of soils where the standard cone is difficult to remove from the soil. According to Webster et al. (1992), the disposable cone allows the operator to perform twice the number of tests per day than with the standard cone. At the midpoint of the upper and lower rods, an anvil is located for use with the dual mass sliding hammers. By dropping either a 10.1 or a 17.6 pound hammer 22.6 inches and impacting the anvil, the DCP is driven into the ground. For comparison, the penetration depth caused by one blow of the 17.6 pound sliding hammer would be approximately equivalent to two blows from the 10.1 pound hammer. The 10.1 pound hammer is more suitable for sensitive clayey soils with CBR values ranging from 1 to approximately 10; however, it is capable of estimating CBR values up to 80. In general, the 17.6 pound hammer is rated at accurately measuring CBR values from 1 to 100. At its full capacity, the DCP is designed to penetrate soils up to 39 inches. In highly plastic clay soils, the accuracy of the DCP index decreases with depth due to soil sticking to the lower rod. If necessary, hand-augering a 2 inch diameter hole can be used to open the test hole in 12 inch increments, preventing side friction interference.

CBR and DCP index (PI):

1) For all soils except CL below CBR of 10, and CH soils:

$$CBR = \left(\frac{292}{PI}\right)^{1.12}$$

2) For soils with CBR less than 10:

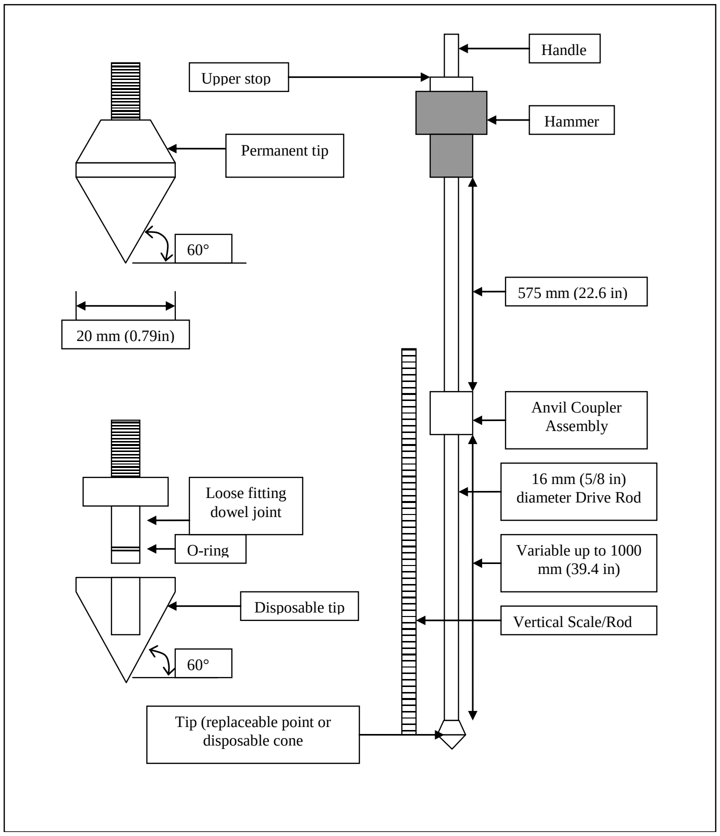
$$CBR = \left(\frac{1}{0.0170019xPI}\right)^{2}$$

3) For CH soils:

$$CBR = \left(\frac{1}{0.002871xPI}\right)$$

Where PI = Penetration index from DCP, (mm/blow)

Figure 6E-1.05: DCP Design and Cone Tip Details



b. Relationship of M_R and k-value: An approximate relationship between k and M_R published by AASHTO is fairly straightforward.

$k = M_R/19.4$

where

k = modulus of subgrade reaction (psi/inch)
M_R = roadbed soil resilient modulus of the soil as determined by AASHTO T 274.

c. Relationship of CBR, MR, and k-value: See approximate relationships in Table 6E-1.01.

E. Subgrade Construction

1. General: The most critical element for subgrade construction is to develop a CBR of at least 10 in the prepared subgrade using on-site, borrow, or modified soil (see Section 6H-1 - Foundation Improvement and Stabilization). Uniformity is important, especially for rigid pavements, but the high level of subgrade support will allow the pavement to reach the design life.

In most instances, once heavy earthwork and fine grading are completed, the uppermost zone of subgrade soil (roadbed) is improved. The typical improvement technique is achieved by means of mechanical stabilization (i.e., compaction). Perhaps the most common problem arising from deficient construction is related to mechanical stabilization. Without proper quality control and quality assurance (QC/QA) measures, some deficient work may go unnoticed. This is most common in utility trenches and bridge abutments, where it is difficult to compact because of vertical constraints. This type of problem can be avoided, or at least minimized, with a thorough plan and execution of the plan as it relates to QC/QA during construction. This plan should pay particular attention to proper moisture content, proper lift thickness for compaction, and sufficient configuration of the compaction equipment utilized (weight and width are the most critical). Failure to adequately construct and backfill trench lines will most likely result in localized settlement and cracking at the pavement surface.

2. **Compaction:** Compaction of subgrade soils is a basic subgrade detail and is one of the most fundamental geotechnical operations for any pavement project. The purpose of compaction is generally to enhance the strength or load-carrying capacity of the soil, while minimizing long-term settlement potential. Compaction also increases stiffness and strength, and reduces swelling potential for expansive soils.

   a. **Density/Moisture:** The most common measure of compaction is density. Soil density and optimum moisture content should be determined according to ASTM D 698 (Standard Proctor Density) or ASTM D 4253 and D 4254 (Maximum and Minimum Index Density for Cohesionless Soils). At least one analysis for each material type to be used as backfill should be conducted unless the analysis is provided by the Engineer.

      Field density is correlated to moisture-density relationships measured in the lab. Moisture-density relationships for various soils are discussed in Part 6A - General Information. Optimal engineering properties for a given soil type occur near its compaction optimum moisture content, as determined by the laboratory tests. At this state, a soils-void ratio and potential to shrink (if dried) or swell (if inundated with water) is minimized.

      For pavement construction, cohesive subgrade soil density should satisfy 95% of Standard Proctor tests, with the moisture content not less than optimum and not greater than 4% above optimum. For cohesionless soils (sands and gravel), a minimum relative density of 65% should be achieved with the moisture content greater than the bulking moisture content.

   b. **Strength/Stiffness:** Inherent to the construction of roadway embankments is the ability to measure soil properties to enforce quality control measures. In the past, density and moisture content have been the most widely measured soil parameters in conjunction with acceptance criteria. However, it has been shown recently that density and moisture content may not be an adequate analysis. Therefore, alternate methods of in-situ testing have been reviewed. The dual mass Dynamic Cone Penetrometer (DCP) is a method for estimating in-place stability from CBR correlations.

   c. **Equipment:** Several compaction devices are available in modern earthwork, and selection of the proper equipment is dependent on the material intended to be densified. Generally, compaction can be accomplished using pressure, vibration, and/or kneading action. Different types of field compaction equipment are appropriate for different types of soils. Steel-wheel rollers, the earliest type of compaction equipment, are suitable for cohesionless soils. Vibratory steel rollers have largely replaced static steel-wheel rollers because of their higher efficiency. Sheepsfoot rollers, which impart more of a kneading compaction effort than smooth steel wheels, are most appropriate for plastic cohesive soils. Vibratory versions of sheepsfoot rollers are also available. Pneumatic rubber-tired rollers work well for both cohesionless and cohesive soils. A variety of small equipment for hand compaction in confined areas is also available. Table 3 summarizes recommended field compaction equipment for various soil types.

Table 6E-1.03: Recommended Field Compaction Equipment

Soil	First Choice	Second Choice	Comment
Rock fill	Vibratory	Pneumatic	--
Plastic soils, CH, MH	Sheepsfoot or pad foot	Pneumatic	Thin lifts usually needed
Low-plasticity soils, CL, ML	Sheepsfoot or pad foot	Pneumatic, vibratory	Moisture control often critical for silty soils
Plastic sands and gravels, GC, SC	Vibratory, pneumatic	Pad foot	--
Silty sands and gravels, SM, GM	Vibratory	Pneumatic, pad foot	Moisture control often critical
Clean sands, SW, SP	Vibratory	Impact, pneumatic	--
Clean gravels, GW, GP	Vibratory	Pneumatic, impact, grid	Grid useful for over-size particles

Source: Rollings and Rollings 1996

The effective depth of compaction of all field equipment is usually limited, so compaction of thick layers must be done in a series of lifts, with each lift thickness typically in the range of 6 to 8 inches.

The soil type, degree of compaction required, field compaction energy (type and size of compaction equipment and number of passes), and the contractor’s skill in handling the material are key factors determining the maximum lift thickness that can be compacted effectively. Control of water content in each lift, either through drying or addition of water plus mixing, may be required to achieve specified compacted densities and/or to meet specifications for compaction water content.

Proof-rolling with heavy rubber-tired rollers is used to identify any remaining soft areas. The proof-roller must be sized to avoid causing bearing-capacity failures in the materials that are being proof-rolled. Proof-rolling is not a replacement for good compaction procedures and inspection. An inspector needs to be present onsite to watch the deflections under the roller in order to identify soft areas. Construction equipment such as loaded scrapers and material delivery trucks can also be used to help detect soft spots along the roadway alignment. It is very difficult to achieve satisfactory compaction if the lift is not on a firm foundation.

3. **Overexcavation/Fill:** The installation of structural features (e.g., sewer, water, and other utilities) adjacent to or beneath pavements can lead to problems during or following construction. Proper installation of such utilities and close inspection during construction are critical.

   A key element in the installation of these systems is proper compaction around and above the pipe. Granular fill should always be used to form a haunch below the pipe for support. Some agencies are using flowable fill or controlled low strength material (CLSM) as an alternative to compacted granular fill. Without this support feature, the weight above the pipe may cause it to deform, creating settlement above the pipe, and often pipe collapse. Even if a sinkhole does not appear, leaks of any water-bearing utility will inundate the adjacent pavement layers, reducing their support capacity.

   Pavement problems also occur when improper fill is used in the embankment beneath the pavement system. Placement of tree trunks, large branches, and wood pieces in embankment fill must not be allowed. Over time, these organic materials decay, causing localized settlement, and they eventually form voids in the soil. Again, water entering these voids can lead to collapse and substantial subsidence of the pavement section. Likewise, placement of large stones and boulders in fills create voids in the mass, either unfilled due to bridging of soil over the large particles or filled with finer material that cannot be compacted with conventional equipment. Soil above these materials can migrate into the void space, creating substantial subsidence in the pavement section. These issues can be mitigated with well-crafted specifications that will prohibit the use of these types of materials.

Transitions between cut zones and fill zones can also create problems, particularly related to insufficient removal of weak organic material (clearing and grubbing), as well as neglect of subsurface water movements. A specific transition also occurs at bridge approaches. These problems are typically related to inadequate compaction, usually a result of improper compaction equipment mobilized to the site or lack of supervision and care (e.g., lift placement greater than compaction equipment can properly densify).

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Pavement Subbase Design and Construction

A. General Information

Pavement systems generally consist of three layers: prepared subgrade, subbase, and pavement. This section will deal with the proper design and construction of subbases. The subbase is the layer of aggregate material that lies immediately below the pavement and usually consists of crushed aggregate or gravel or recycled materials (see Section 6C-1 - Pavement Systems for more information). Although the terms "base" and "subbase" are sometimes used interchangeably to refer to the subsurface layers of a pavement, base course is typically used in asphalt pavements, primarily as a structural load-distributing layer, whereas the subbase layer used in concrete pavements primarily serves as a drainage layer. Aggregate subbase is typically composed of crushed rock, comprised of material capable of passing through a 1 1/2 inch screen, with component particles varying in size from 1 1/2 inch down to dust. The material can be made of virgin (newly mined) rock or of recycled asphalt and concrete.

The function of the pavement subbase is to provide drainage and stability to achieve longer service life of the pavement. Most pavement structures now incorporate subsurface layers, part of whose function is to drain away excess water that can be deleterious to the life of the pavement (see Section 6G-1 - Subsurface Drainage Systems). However, aggregate materials for permeable bases must be carefully selected and properly constructed to provide not only permeability, but uniform stability as well. Proper construction and QC/QA testing operations can help to ensure good performance of the subbase layer. Excessive compaction can alter the gradation and create additional fines that may result in lower permeabilities than determined in laboratory tests and used in the pavement system design. However, the optimization of structural contributions from high stability, versus the need to provide adequate drainage for pavement materials is still a point of debate. The focus of this section is to provide guidance on selection of proper subbase materials, best construction practices, and suitable QC/QA testing methods.

B. Granular Subbases

1. **Purpose:** Subbases serve a variety of purposes, including reducing the stress applied to the subgrade and providing drainage for the pavement structure. The granular subbase acts as a load-bearing layer, and strengthens the pavement structure directly below the pavement surface, providing drainage for the pavement structure on the lowest layer of the pavement system. However, it is critical to note that the subbase layer will not compensate for a weak subgrade. Subgrades with a CBR of at least 10 should provide adequate support for the subbase.

2. **Materials:** As the granular subbase provides both bearing strength and drainage for the pavement structure, proper size, grading, shape, and durability are important attributes to the overall performance of the pavement structure. Granular subbase aggregates consist of durable particles of crushed stone or gravel capable of withstanding the effects of handling, spreading, and compacting without generation of deleterious fines.

3. **Gradation:** Aggregates used as subbase tend to be dense-graded with a nominal maximum size, commonly up to 1 1/2 inches. The percentage of fines (passing No. 200 sieve) in the subbase is limited to 10% for drainage and frost-susceptibility purposes. The Engineer may authorize a change in the gradation at the time of construction based on materials available.

    a. **Particle Shape:** Equi-dimensional aggregate with rough surface texture is preferred.

    b. **Permeability:** The fines content is usually limited to a maximum of 10% for normal pavement construction and 6% where free-draining subbase is required.

    c. **Plasticity:** Plastic fines can significantly reduce the load carrying capacity of subbase; plasticity index (PI) of the fines of 6 or less is required.

4. **Construction:** Granular subbases are typically constructed by spreading the materials in thin layers compacting each layer by rolling over it with heavy compaction equipment to achieve a density greater or equal to 70% relative density.

5. **Thickness Requirement:** Typically, the thickness of the subbase is 6 inches with a minimum of 4 inches. Additional thickness beyond 6 inches could allow consolidation of the subbase over time as traffic loads accumulate. Pavement problems may result from this consolidation.

C. Recycled Materials

Recycled materials with the required particle distribution, high stiffness, low susceptibility to frost action, high permeability, and high resistance to permanent deformation can be successful subbases. Recycled aggregate can solve disposal problems, conserve energy, and lower the cost of road construction.

1. **Recycled Concrete Aggregate:** To reduce the use of natural aggregate and help preserve the environment, recycled concrete aggregate can be used. Consider the following precautions:
    - The breakage of particles results in faces, which can react with water and produce high pH. This may result in poor freeze-thaw performance.
    - The breakage of particles due to compaction and traffic loading will increase the fines percentage. This increasing fine percentage will reduce freeze-thaw resistance and permeability of bases.
    - Increased pH due to cement hydration can cause corrosion of aluminum and steel pipes.

2. **Recycled Asphalt Pavement:** Consider the following precautions.
    - 20% to 50% RAP is typically used. High percentages of RAP are not used in normal construction.
    - The stiffness increases with higher percentage of RAP, while there must be limits on percentage of RAP to incorporate into virgin material.

D. Effects of Stability and Permeability on Pavement Foundation

The subbase is the layer of aggregate material that lies immediately below the pavement and usually consists of crushed aggregate or recycled materials.

1. **The Main Roles of the Subbase Layer in Pavements:** Include provision of the following (Dawson 1995).
   - Protection for the subgrade from significant deformation due to traffic loading
   - Adequate support for the surface layer
   - Stable construction platform during pavement surfacing
   - Adequate drainage for the infiltration of rain water through cracks and joints, particularly in PCC pavements (see Section 6G-1 - Subsurface Drainage Systems)
   - Subgrade protection against frost and environmental damage

2. **Effect of Undrained Water on Pavement Foundation:** Undrained water in the pavement supporting layers is a major contributor to distress and premature failure in pavements. Some of the detrimental effects of water, when entrapped in the pavements structure are that (Yang 2004):
   - Water reduces the strength of unbounded granular materials and subgrade soils.
   - Water causes pumping of concrete pavements with subsequent faulting, cracking, and general shoulder deterioration.
   - With the high hydrodynamic pressure generated by moving traffic, pumping of fines in the base course of flexible pavements may also occur with resulting loss of support.
   - In northern climates with a depth of frost penetration greater than the pavement thickness, high water table causes frost heave and the reduction of load-carrying capacity during the frost melting period.
   - Water causes differential heaving over swelling soils.
   - Continuous contact with water causes stripping of asphalt mixture and durability or “D” cracking of concrete.

   Accumulated water in the subbase is a key contributing factor to subbase instability and pavement distress. Thus it is important to understand how water becomes trapped in the subbase layer. A number of other factors also affect the engineering behavior of aggregates, including fines content; aggregate type, grading, size, and shape; density; stress history; and mean stress level. Table 6F-1.01 summarizes the relative effects of these factors. From this table, it can be seen that:
   - Aggregate stiffness is increased by an increase in most of the controlling factors, with the exception of fines content and moisture content, which decrease the stiffness.
   - An increase in susceptibility to permanent deformation can be caused by increasing fines content and moisture content, while most other factors decrease the susceptibility.
   - Strength is generally increased with an increase in density; good grading; and aggregate angularity, size, and stress level.
   - Fines content has a major effect on permeability, with increased fines leading to a decrease in permeability. A well-graded aggregate is also much less permeable than a uniform gradation.
   - Increased fines content decreases durability, while the changes caused by most of the other factors are minor in comparison.

Table 6F-1.01: Effects of Intrinsic and Manufactured Properties of Aggregates as Controlling Factors on Engineering Properties of Granular Material in Pavement Layers

Controlling Factor	Property				
	Stiffness	***Susceptibility to Permanent Deformation***	***Strength***	***Permeability***	***Durability***
Fines content	↓?	↑	varies	major↓	↓
Type: gravel instead of crushed rock	↑	↑	↑	none	usually↑
Grading: well graded instead of single-sized	minor↑	↓	↑	major↓	↓
Maximum size: large instead of small	↑	↓?	minor↑	↑	↓?
Shape: angular/rough instead of rounded/smooth	↑	↓	↑	minor	minor
Density	↑	↓	↑	↓	minor
Moisture content	major↓	major↑	major↓	major↑	varies
Stress history	↑?	major↓	minor↓	none	?
Mean stress level	↑	↓	↑	minor↓	↓

Notes:

↑ = Value of property increases with increase (or indicated change) in controlling factor

↓ = Value of property decreases with increase (or indicated change) in controlling factor

? = Effect of property variation not well established

Source: Dawson et al. 2000

E. Effect of Compaction

According to Merriam-Webster's Collegiate Dictionary Eleventh Edition (2003), compaction is defined as "the act or process of compacting; the state of being compacted; to closely unite or pack, to concentrate in a limited area or small space." It is thus a process of particles being forced together to contact one another at as many points as physically possible with the material. Density is defined as "the quality or state of being dense; the quantity per unit volume," as the weight of solids per cubic foot of material. Thus, density is simply a measure of the number of solids in a unit volume of material; density and degree of compaction differ. Two aggregate bases may have the same density but different degrees of compaction due to differences in gradation.

Also, the maximum achievable density, when calculated based on standard lab procedures at a certain level of degree of compaction, is true only when material tested in the laboratory is identical to the field material in all respects of engineering parameters, or the same compactive effort is used to achieve compaction. Therefore, differences in materials and compactive effort can significantly change the density, thereby rendering the calculated percent compaction meaningless. Laboratory compaction testing performed on subbase layers according to AASHTO T 99; Standard Proctor density shows a significant change in density and optimum water content with change in gradation in similar aggregate types. Therefore, it is recommended to use relative density values correlated to gradation for compaction control of aggregate materials in the field to avoid inadequate compaction. A relative density of at least 70% is recommended.

F. Influence of Aggregate Properties on Permeability of Pavement Bases

The drainability of a pavement subbase is measured using the coefficient of permeability, denoted as k, which defines the quantity of water that flows through a material for a given set of conditions. The quantity of flow through a given medium increases as the coefficient of permeability increases.

The coefficient of permeability is defined as "the rate of discharge of water at 20^{o} C under conditions of laminar flow through a unit cross-sectional area of a soil medium under a unit hydraulic gradient" (Thornton and Leong 1995). Coefficient of permeability measured in pavement subbases is denoted as hydraulic conductivity, which has the same units as velocity, and is expressed in units of length per time (cm/sec or feet per day). (Note: 1 cm/s = 2835 feet per day). Various properties that influence hydraulic conductivity of a pavement subbase include: gradation and shape of aggregate, hydraulic gradient, viscosity of the permeant, porosity and void ratio of the mix, and degree of saturation (Das 1990).

1. **Effect of Gradation and Shape of Aggregate:** According to Cedergren (1974), the life of a poorly drained pavement is reduced to one-third or even less of the life of a well drained pavement.

   Miyagawa (1991) conducted both laboratory and in-situ hydraulic conductivity tests on a wide range of pavement subbases in Iowa. Laboratory test results indicate that crushed limestone has higher hydraulic conductivity with a range of 7,000 to 36,900 feet per day, compared to crushed concrete with a range of about 340 to 12,780 feet per day. A procedure was developed to obtain a relative idea of in-situ hydraulic conductivity tests. This consisted of coring out an approximately 4 inch diameter hole to a depth of 4 to 5 inches, filling the hole with 1 liter of water, and measuring the time taken to drain the water from the hole. Compared to laboratory test results, in-situ tests produce on the order of 20 to 1,000 feet per day. This reduction is believed to be a result of changes in gradation during compaction of the subbase material.

2. **Thickness Design for Achieving Desired Drainability:** The major sources of water in pavement systems are surface infiltration, ground water seepage, and melting of ice lenses. A complete pavement drainage system is typically composed of an aggregate subbase, subdrains, and connections to storm sewage systems (see 6G-1 - Subsurface Drainage Systems). A positive drainage system should transport water from the point of infiltration to the final exit (transverse drains) through material having high hydraulic conductivity and should eliminate any conditions that would restrict the flow (Moulton 1980).

G. Construction Methods

Benefits of using open-graded permeable subbase layers are widely accepted throughout the world. But working with open-graded material in the field and obtaining a workable platform for the overlying surface is not yet well defined. According to White et al. (2004), significant segregation of fines is observed on subbase projects in Iowa, thus contributing to the high variation (coefficient of variation = 100%) in the measured in-place permeability. To reduce segregation, the following construction operations were recommended:

- A motor grader with a sharp angle (i.e., 45 degrees), should be used to push the aggregate transversely from a center windrow/pile, instead of spreading the aggregate material longitudinally along the pavement section (Pavement Technology Workshop 2000).
- When recycled PCC is used for granular subbases, construction traffic on the subbase should be minimized.
- A motor grader with GPS-assisted grading (i.e., stakeless grading control should be used to prepare the final surface for paving), rather than trimming equipment.

If trimming equipment must be used, the aggregate should be delivered to the site with sufficient water content (7% to 10%) to bind the fines during trimming to prevent segregation.

The key to a properly constructed subbase is keeping the material uniformly moist and homogeneously blended. The amended subbase material may be placed and trimmed with an auto-trimmer or dumped from trucks and spread with a motorgrader. The placement and compaction should be completed to minimize segregation and with a minimal increase in fines.

H. Quality Control/Quality Assurance Testing

1. **In-situ Measurement of Stability of Aggregate Subbase:**

    a. **Dynamic Cone Penetrometer (DCP) Test:** DCP is an instrument designed for rapid in-situ measurement of the structural properties of existing pavements with unbound granular materials (Ese et al.1994). The cone penetration is inversely related to the strength of the material. DCP test is conducted according to ASTM D 6951 (Standard Test Method for Use of Dynamic Cone Penetrometer in Shallow Pavement Applications), which was first released in 2003. This test involves measurement of penetration rate per each blow of a standard 17.6-pound hammer, through undisturbed and/or compacted materials. Primary advantages of this test are its availability at lower costs and ease to collect and analyze the data rapidly (See Section 6E-1 - Subgrade Design and Construction, for more information).

    b. **Clegg Impact Hammer Test:** This test was standardized in 1995 as ASTM D 5874, (Standard Test Method for Determination of the Impact Value IV of a Soil). This is a simple and rapid in-situ test that can be performed on subbase and subgrade materials. This test method is suitable to evaluate the strength characteristics of soils and soil aggregates having maximum particle size less than 1.5 inches (ASTM D 5874).

    c. **GeoGauge Vibration Stiffness Test:** The GeoGauge is a 22 pound electro-mechanical instrument, which provides a direct measure of in-situ stiffness (MN/m) and modulus (MPa). The test is a simple non-nuclear test on soils and granular materials that can be performed without penetrating into the ground.

    d. **Portable Falling Weight Deflectometer (PFWD) Test:** The PFWD test is a simple and rapid non-destructive test that does not entail removal of pavement materials, and hence is often preferred over other destructive methods. In addition, the testing apparatus is easily transported. Layer moduli can be back-calculated from the observed dynamic response of the subbase surface to an impulse load.

    e. **Falling Weight Deflectometer (FWD) Test:** The FWD is a trailer-mounted system that is similar to the PFWD but generally imparts a higher load pulse to simulate vehicle wheel loads. FWD tests are normally performed on the pavement surface, but, with special testing criteria, they can be performed directly on granular base layers and can be used to back-calculate layer moduli up to about 6 feet deep. FWD results are often dependent on factors such as the particular model of the test device, the specific testing procedure, and the method of back-calculation (FAA 2004).

2. **In-situ Hydraulic Conductivity Testing:** Construction operations might significantly alter the material properties from what are tested in the laboratory. Hence, in-situ hydraulic conductivity testing provides better insights to evaluate the performance of pavement subbases. Although a variety of approaches to determine the field permeability have been documented (Moulton and Seals 1979), virtually no in-situ testing is being conducted as part of the construction practice to verify the hydraulic conductivity of granular subbase layers; yet the impact of drainage on design calculations and long-term performance is well documented. This lack of field permeability measurement provides little confidence that assumed design values are representative of the actual field conditions and does not address the fact that permeability is one of the most highly variable parameters in geotechnical engineering practice. Some of the factors that contribute to the high level of variability include inherent variations in the material gradation and morphology; segregation caused from construction activities to deposit and spread the aggregate; and particle breakdown from compaction and construction traffic (White et al. 2004).

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Subsurface Drainage Systems

A. General Information

Subsurface drainage is a key element in the design of pavement systems. Indiscriminate exclusion of this element will assuredly lead to the premature failure of pavement systems, thereby resulting in high life-cycle costs. Faulting and associated pumping in rigid pavements systems, extensive cracking from loss of subgrade support in flexible pavements, and distress from frost heave are clear signs of inadequate drainage. The two basic design strategies promoted are to (1) prevent water from entering in the first place and (2) quickly remove any water that does infiltrate. After years of unsuccessful sealing attempts, the profession has learned that we cannot prevent water from entering a pavement and that removal of water is essential for the pavement elements to perform as desired (Christopher and McGuffey 1997).

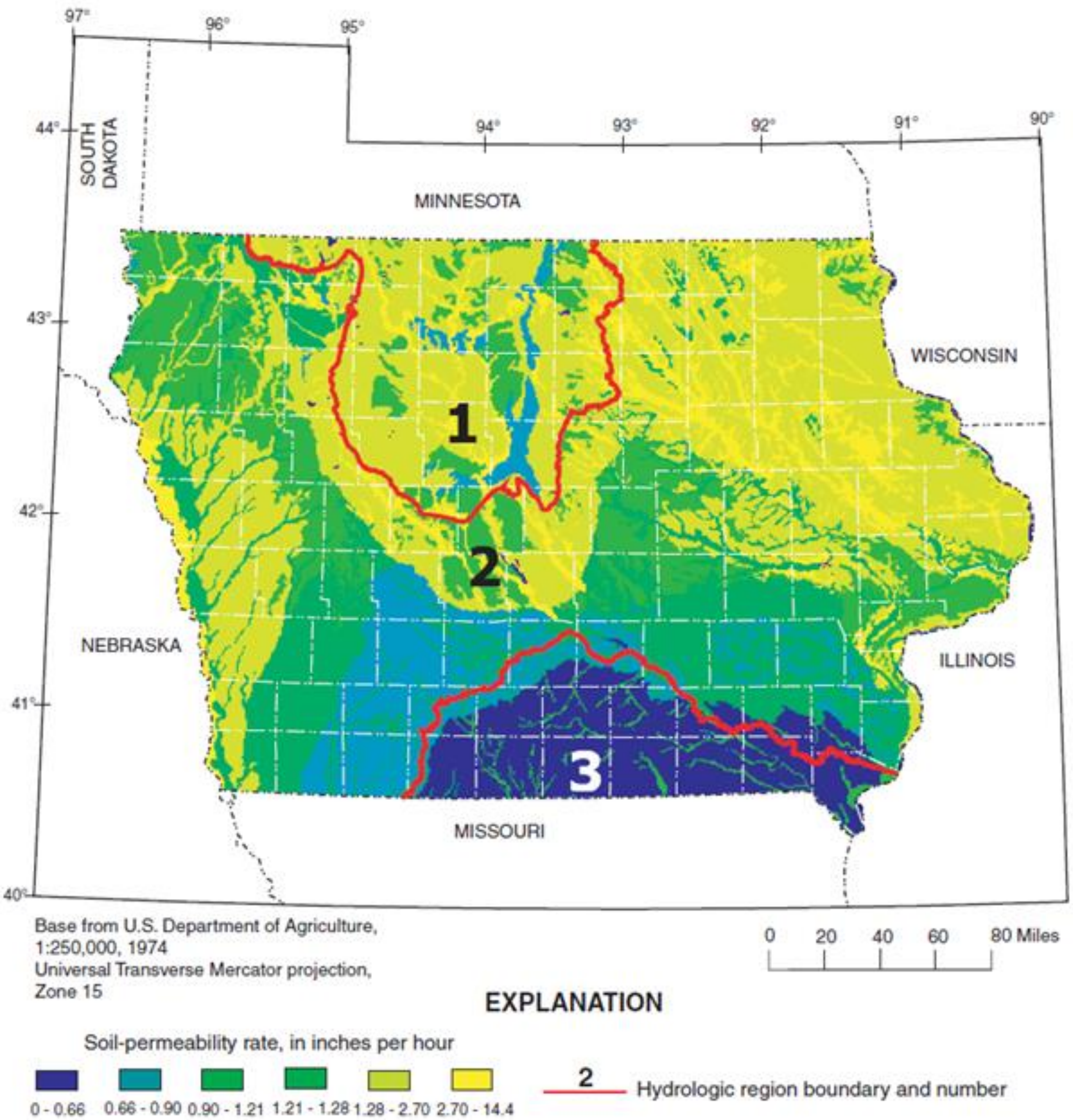
Proper drainage cannot be overstressed in road construction. Water affects the entire serviceability of a road. In general, Iowa soils are fine-grained with low permeability. Coupled with a wet climate, if there is no subsurface drainage in pavement construction, the subgrade and subbase can be saturated for long periods. Starting from the bottom up, subsurface drainage may be the most important factor contributing to the longevity of a pavement section. Water in the subgrade and subbase weakens the support provided to the pavement. Maintaining the integrity of the subgrade and subbase can be accomplished through subsurface drainage and separation of the subbase from the subgrade using geotextiles.

Urban pavements with curbs are generally designed to direct surface stormwater within the right-of-way and adjacent property toward the pavement, where it is intercepted and transported by a system of stormwater intakes and pipes. This encourages the introduction of additional subsurface and surface water to the pavement system. Footing drains for adjacent structures may drain to this storm sewer system, a specially-constructed footing drain collector, or a combination subdrain/footing drain collector.

Proper surface drainage can reduce the amount of water infiltrating through the pavement and is a strategy that goes hand in hand with proper subsurface drainage. Most free water will enter the pavement through joints, cracks, and pores in the surface of the pavement. Water also will enter from backup in ditches and groundwater sources. Drainage prevents the buildup of free water in the pavement section, thereby reducing the damaging effects of load and environment. Based on documented case histories, studies have shown that pavement life can be extended up to three times if adequate subsurface drainage systems are installed and maintained (Cedergren 1989).

The importance and design of subgrade and subbase drainage is discussed in Section 6E-1 - Subgrade Design and Construction, and Section 6F-1 - Pavement Subbase Design and Construction. Generally, Iowa's soils are fine-grained and will have low permeability as indicated in the state permeability map shown in Figure 6G-1.01. Most subgrade soils in Iowa have poor drainage quality by AASHTO standards, less than 10 feet per day (< 5 inches/hour). Coupled with the fact that Iowa receives over 20 inches of precipitation a year and is considered a wet climate, subgrades and subbases can be saturated for long periods if subsurface drainage is not accommodated in pavement system construction. Subdrain systems, specifically designed to drain subsurface water, are a solution to remove water from permeable subbases and drainable subgrades. The advantage of a functional subsurface pavement drainage system will vary based on climate, subgrade soils, and the design of overall pavement system.

Figure 6G-1.01: Permeability of Iowa Soils



Source: Eash 2001

Unless a subsurface exploration determines subsurface drainage systems are not necessary, they should be installed for most paving projects in Iowa. A successful drainage design process must adequately and consistently address the following:

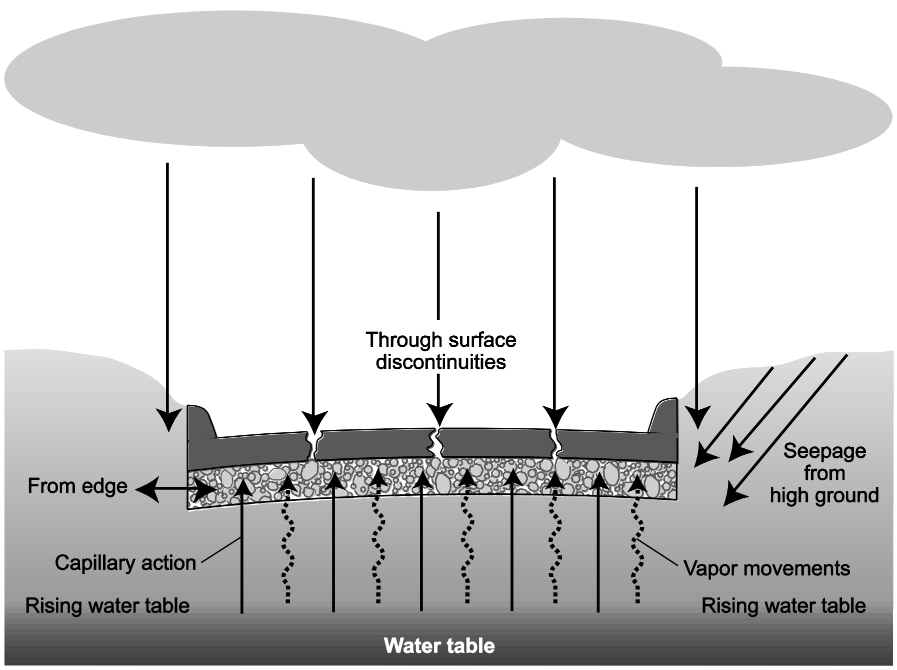
- Evaluation of the need for subdrainage.
- Determination of the necessary subdrainage components for the given situation.
- The hydraulic and structural design of subsurface drainage systems and their integration into the overall pavement design process.
- Property specifications of drainage materials for achieving long-term performance.
- Documentation of special construction and maintenance considerations.

B. Need for Subsurface Drainage

The damaging effects of excess moisture on pavement have long been recognized. Moisture from a variety of sources can enter a pavement structure. Figure 6G-1.02 shows that moisture in the subgrade and pavement structure can come from many different sources. Water may seep upward from a high ground water table, or it may flow laterally from the pavement edges. Knowledge of ground water and its movement are critical to the performance of pavement as well as the stability of adjacent sideslopes. Ground water can be particularly troublesome for pavements in low-lying areas. When pavements are constructed below the permanent or a seasonally high water table, drainage systems must perform or rapid pavement failure will occur. This moisture, when combined with traffics loads, voids in pavement sections, and freezing temperatures, can have a negative effect on both material properties and overall performance of a pavement system.

The most significant source of excess water in pavements is typically infiltration through the surface through joints, cracks, and other defects in the surface that provide an easy path for water. The problem only worsens with time. As pavements age and deteriorate, cracks become wider and more abundant and joints and edges deteriorate into channels through which water is free to flow. The result is more water being allowed into the pavement structure with increasing age, which leads to accelerated development of moisture-related distresses and pavement deterioration. Excess moisture in a pavement structure can adversely affect pavement performance. While a pavement structure can be stable at given moisture contents, the pavement structure may become unstable if the materials become saturated. High water pressures can develop under traffic loads. Water in the pavement structure can freeze and expand, developing high internal pressures on the pavement structure. Flowing water can carry soil particles and lead to clogging of drains and, in combination with traffic, lead to pumping of fines from the subbase or the subgrade.

Figure 6G-1.02: Sources of Moisture in Pavement Systems



Source: Based on FHWA-NHI 2004

C. Types of Drainage Systems

To avoid moisture-related problems, a major objective in pavement design should be to keep the subgrade, subbase, and pavement structure from becoming saturated or exposed to high moisture levels. Three approaches exist for controlling or reducing the problems caused by moisture:

1. Prevent moisture from entering the pavement system

2. Use materials and design features that are insensitive to the effects of moisture

3. Quickly remove moisture that enters the pavement system.

No single approach can completely negate the effects of moisture on the pavement system over a period of many years. It is practically impossible to effectively seal the pavement from water intrusion. While materials that resist moisture can be incorporated, this is often not cost effective and in many cases such materials are simply not available locally. Indeed, subgrades that are susceptible to moisture deterioration cannot easily or cost effectively be replaced. Thus, the need for drainage systems that can quickly and effectively remove water from the pavement system is necessary.

Positive drainage can be affected with three elements:

1. Subbase to provide rapid drainage of free water that may enter the pavement structure.

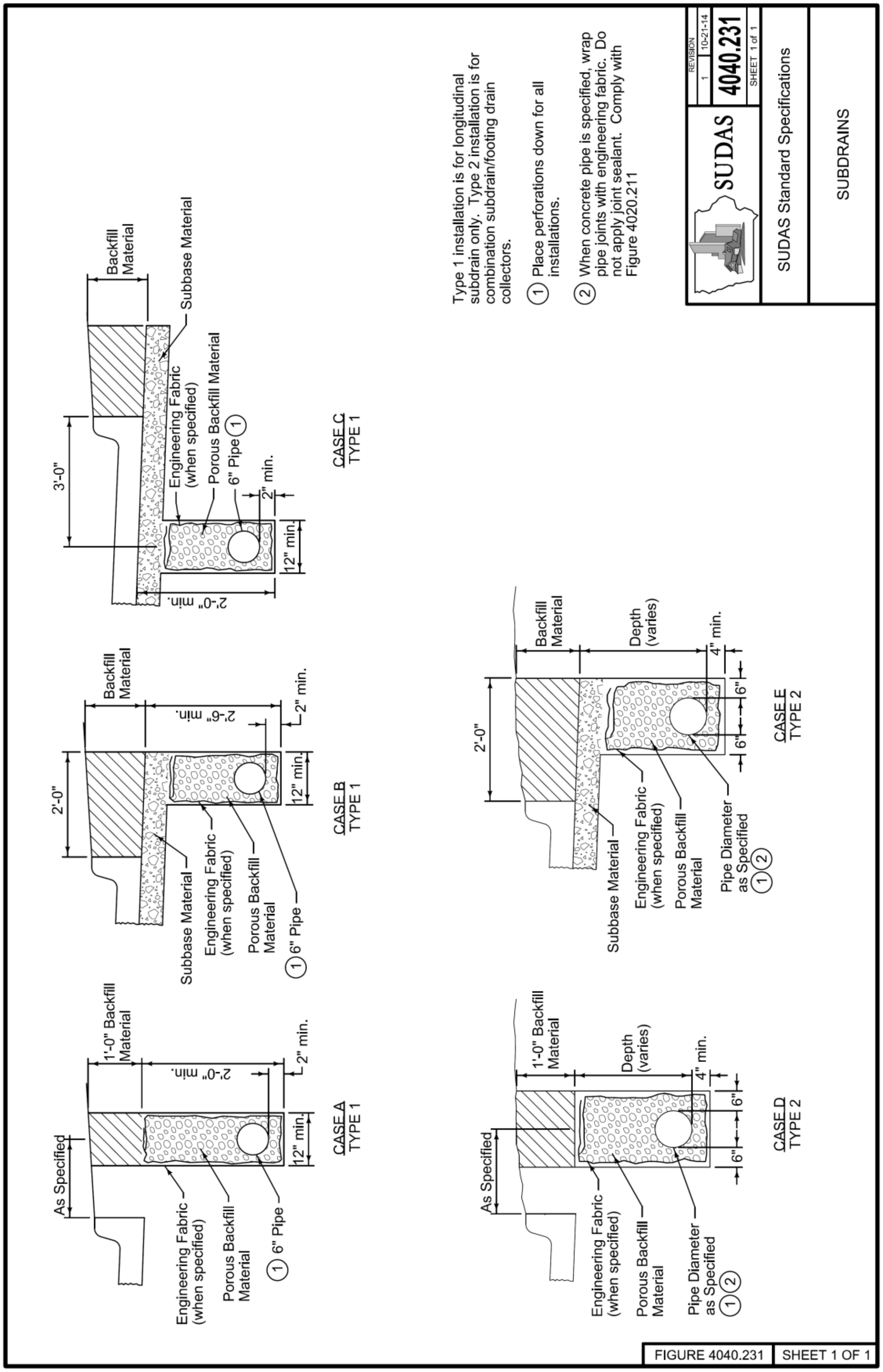
2. Longitudinal subdrain collector system to convey accumulated water from the subbase.

3. Filter-separator layer to prevent the migration of fines (minus 200 sieve material) into the subbase from the subgrade (see Figure 6G-1.03, Cases A and C).

Unrestricted flow to the subbase must be ensured. The filter-separator layer, whether aggregate or geotextile, must be properly designed to prevent migration of fines and possible base contamination. Since many existing pavements have been designed and constructed with impermeable subgrades, rapid lateral drainage from the base of these rehabilitated pavement sections is not feasible. Here, retrofit with longitudinal subdrains can affect drainage of water that has infiltrated the pavement structure and migrated to the slab/subgrade interface. Subdrains placed adjacent to the pavement can intercept this water and shorten the time it is present at the interface, thereby minimizing the potential degradation effects (see Figure 6G-1.03, Case B).

Generally, footing drains for adjacent structures may drain to a storm sewer system or a combination subdrain/footing drain collector. However, a combination subdrain/footing drain collector, as shown in Figure 6G-1.03, Cases D and E, may be installed to serve both purposes. See Chapter 2 - Stormwater, for guidance on sizing of footing drain collectors; normally pipe sizes range from 8 to 12 inches in diameter.

Figure 6G-1.03: Subdrains
(SUDAS Specifications Figure 4040.231)



D. Design

Design of subsurface pavement drainage systems consists of balancing permeability and stability and removing collected water rapidly. Important components consist of subbase material, a separating layer to prevent infiltration of subgrade materials into the subbase, and a collection and removal system. Design approaches for each of the components are summarized below.

1. **Subbase:** For the design of subbases, see Section 6F-1 - Pavement Subbase Design and Construction. One of the purposes of the subbase is to remove infiltration water. The subbase should consist of durable, crushed, angular aggregate with the best porosity so that it will release the maximum amount of water. However, the structural requirements for the overall pavement section must be met using appropriate pavement design practices. The subbase can be stabilized or unstabilized. Effective subbase design must address structural, hydraulic, material durability and quality, constructability, and maintenance requirements.

   Hydraulic requirements must be addressed for specific project conditions; however, the time period that free water is present within the pavement structure should be minimized, preferably less than 2 hours following end of precipitation. To maintain positive flow through the base, the road section should be sloped as much as possible, with a minimum cross slope of 2%. The highest permeability materials are unstable under construction traffic; therefore, it is desirable to use a more stable material with a lower permeability, such as 150 to 350 feet per day (75 to 175 inches per hour).

   FHWA (1992) guidelines indicate that the quality of crushed aggregates is the single most important factor for the stability of a subbase. Breakdown of the aggregate could cause both loss of support and a decrease in permeability. Los Angeles Abrasion Wear should not exceed 50%, and aggregate soundness loss should not exceed the requirements for a Class B aggregate as specified by AASHTO M 283 (i.e., 12% for sodium sulfate test or 18% for magnesium sulfate test).

   To enable proper construction of subbases, several construction guidelines have been proposed (Christopher and McGuffey 1997). Unstabilized materials generally are used in thicknesses of 4 inches or more. Asphalt and cement stabilized materials can be built as thin as 2 inches, however, 4 inches is recommended as a minimum. Material gradations vary widely; see White et al. (2004) for a review.

   Of the subbase materials included in SUDAS Specifications Section 2010, only granular subbase and modified subbase will provide adequate permeability. Granular subbase provides the highest permeability, however it is generally unstable under construction traffic. Modified subbase provides both stability and good permeability.

2. **Separator/Filter Layers:** There is usually a need for a separator/filter layer between the subbase and the subgrade. Filtration compatibility of the subbase must be evaluated with respect to both the subgrade and the subbase to prevent migration of the subgrade into the subbase.

   Geotextiles are commonly used as separators/filters. The FHWA geosynthetics manual (Holtz et al. 1995) provides guidelines on design procedures. Care must be exercised in the amount of cover material over geotextiles as there is potential for damage from equipment. Normally, 6 inches is considered the minimum thickness when earthmoving equipment is used for placement.

   Dense-graded (low permeability) subbase can be placed below the permeable subbase and provide adequate separation. Filter criteria need to be checked for impermeable subbase materials that will be adjacent to the permeable subbase.

3. **Subdrains:**

a. **New Construction:** Subdrains for new construction generally consist of pipe in a trench lined with non-woven geotextile (engineering fabric) and filled with aggregate. Typical installation sections are shown in Figure 6G-1.03, Cases B, C, and E. Design of subdrains for new construction and major reconstruction projects consists of ensuring that the trench backfill and subdrain pipe have the capacity to handle the design flow from the subbase.

The size of pipe is often based on maintenance requirements for cleaning capabilities and reasonable distance between outlets. Although FHWA recommends a minimum pipe diameter of four inches, the SUDAS Specifications require a minimum of 6 inch diameter pipe for Type 1 subdrain installations and a minimum of eight inch diameter pipe for Type 2 combination subdrain/footing drain collectors. The larger diameter subdrain pipe allows for additional capacity, easier cleaning, and inspection. Cleanouts are required for all Type 2 subdrains, at the end of line or at 300 feet spacings. For exceptionally long Type 1 installations, greater than 300 feet from an outlet, consideration should be given to providing cleanouts as required for Type 2 subdrains.

Trench backfill aggregate could be the same as the subbase or a material with greater permeability. AASHTO No. 57 stone, Iowa DOT Gradation No. 3 has been used for trench backfill. The SUDAS Specifications Section 3010 requires porous backfill to comply with Iowa DOT Gradation No. 29 or the use of commercially available pea gravel. The non-woven geotextile used to line the subdrain trench must be designed as a filter, considering both the subbase and subgrade soils. The geotextile should not be extended between the interface of the subbase and the trench backfill aggregate because it may form a barrier. Also, geotextile should not be wrapped around the perforated drainage pipe.

One of the most critical items for subdrains is the grade of the invert. Construction control of very flat grades usually is not possible, leaving ponding areas that result in subgrade weakening and premature failures. It may be necessary to raise the pavement grade to develop adequate drain slopes for the subsurface drainage facilities. To achieve a desirable drainage capacity, a minimum slope that is greater than the slope of the road may be required for the subdrain, although this is often not practical and the pipe will mostly be sloped the same as the roadway. When adequate slopes cannot be achieved, rigorous maintenance should be anticipated.

The outlet for the subdrain must be low and large enough so that flow from the subdrain does not back up. FHWA recommends that the outlet pipe be at least 6 inches above the 10-year storm flow line of the ditch or hydraulic structure into which the outlet is flowing.

The designed drain trench and backfill must be constructible with normal construction equipment. Construction of subdrains is time-consuming. Care must be taken so that the trench backfill does not become contaminated with adjacent soil that might clog the drainage capacity.

b. **Retrofit Subdrains:** A majority of pavement distress problems are related to excess moisture in the pavement structure. Retrofit subdrains can be used in rehabilitation projects to remove water. The design of retrofit subdrains is substantially different than new construction. Subdrains should be just one of the methods to consider to correct water problems. The principles for the design of retrofit subdrains apply to both HMA and PCC pavements. For the design of retrofit subdrains, the designer is referred to the Concrete Pavement Preservation Guide, 2nd Edition (National Concrete Pavement Technology Center, September 2014) and the Material Subsurface Pavement Drainage Manual (Idaho Transportation Department, 2007).

c. **Geocomposite Subdrains:** Prefabricated, geocomposite subdrains (PGEDs) have recently been in high use and have been found to be very effective in removing water, with drainage rates equal to or better than pipe drains. Although many states have found PGEDs to be cost effective for retrofit applications, problems of clogging and intrusion of fines and buckling during construction have somewhat limited their use. Design considerations for PGEDs are detailed in NCHRP Report 367 (Koerner et al. 1994).

E. Construction Issues

Construction decisions and actions can have a significant impact on the performance of the pavement section. The design and construction groups must consider (1) each phase of construction, including subgrade preparation, placement of separation/filtration layers, construction of drains, placement of subbase, and construction of the pavement section; and (2) how the decisions of one group will affect the actions and decisions of the other group.

In the design phase, the designer must be concerned with how construction details, sequencing of work, site accessibility, and protection of drainage components will integrate with both the methods and equipment that can be used for pavement and drainage facility construction. Design decisions such as location of collector pipes and outlets, temporary and permanent surface drainage, and aesthetic treatments will influence how construction can be conducted. Such decisions will affect the right-of-way required for construction of the drainage systems.

Sequencing is best left to the contractor unless there is a significant impact on the performance of the drainage system. An important construction related design consideration is pipe access at the upstream end of a segment so that inspection and maintenance flushing activities can take place.

One of the primary reasons for bringing construction personnel in at the design phase is to acquaint them with the impact of construction on design. Care exercised during construction of the designed section without compromising the effectiveness of the design is essential to the pavement’s long-term performance. Key performance elements for construction personnel include the following (Christopher and McGuffey 1997).

- Good pavement starts with a good foundation. A stable platform is required for construction of the subbase.
- Quality of aggregate and its ability to meet gradation requirements is essential for meeting expected design performance levels.
- Awareness is needed concerning the fact that the introduction of fines into the subbase during construction could result in premature failure of the pavement.
- Unstabilized base tends to displace under traffic loadings.
- Too much compaction or fine grading can significantly reduce the expected permeability of the subbase.

1. **Subgrade Preparation:** The foundation/subgrade surfaces are required to be level, somewhat smooth, and constructed to required grades. On drainable pavement sections, constructing and maintaining required subsurface grades is essential to maintain positive drainage until the pavement is constructed. Local depressions resulting from soft areas or depressions from equipment trafficking can lead to ponding of water below the pavement structure and subsequent loss of foundation support.

2. **Separator/Filter Layers:** For granular subbase separator/filter layers, the gradation of materials needs to be checked carefully against the design specifications. Materials that are more openly-graded than specified requirements may allow migration of fines through or from the subbase, which can contaminate the permeable layer. Good compaction of the separator/filter layer is

essential for placement of the subbase. The subbase should be observed for rutting during compaction and subsequent trafficking; surface rutting may be an indication of subgrade rutting, which requires immediate attention. Increasingly, geotextile separation/filter layers are being used. For these, material and certification should be checked against the design requirements to ensure that the proper materials have been received and are being use. In constructing geotextile separation or filter layer, a smooth subgrade surface is essential. Therefore, sharp rock protrusion and loose rocks should be removed to avoid damage to the geotextile.

3. **Subdrains:** Proper grade control is required for subdrains to be effective. Undulating lines are not acceptable because water will accumulate in depressed portions of the pipe. Good practice dictates that subdrains be properly connected to the subbase and the outlets. For maintenance purposes, outlet spacing is limited to 300 feet. Subdrains need to be properly connected to the permeable subbase and outlets. Outlets are required to be set at the proper grades, and ditch lines are graded according to drainage requirements. Subdrain lines should be carefully marked to avoid damage due to construction equipment. Therefore, subdrains can sometimes be constructed after pavement construction. In this case, temporary subdrains are required for the permeable subbase.

4. **Permeable Subbase Materials:** Unstabilized subbase material requires close control of material gradation and activities that might produce segregation of the material during placement.

   Subbase materials are very susceptible to segregation during placement. Special care is needed to prevent fines from migrating into the material and clogging the system. The addition of 2% to 3% water by weight reduces the potential for segregation during hauling and placement.

   Excessive compaction with heavy vibratory compactors is not recommended on subbases because of the potential for damage and reduced permeability. Adequate compaction may be achieved with lightweight vibratory compactors or smooth drum rollers because of the relatively narrow gradation range of subbase.

   Care is required to protect the subbase from contamination from dirty equipment, adjacent backfilling operations, or erosion sediment. The subbase should not be allowed to be used as a haul road. Good practice dictates that traffic be minimized and restricted to low speeds with minimal turning. No equipment should be allowed on the permeable materials until the complete drainage of the base and subbase has been confirmed.

F. Maintenance

Maintenance of pavement subsurface drainage systems has been identified as essential to the long-term success of drainage systems and, subsequently, pavements. The most effective maintenance programs use a five-phase approach:

- Routine inspection and monitoring
- Routine preventive maintenance
- Spot detection of problems (occurrences)
- Repair
- Continued monitoring and feedback

Budget constraints have resulted in usually only two phases being conducted: spot detection and repair. Studies show that inspection in conjunction with preventative maintenance can be very cost effective with $3 to $4 return in benefits for every $1 invested (Christopher and McGuffey 1997).

1. **Inspection and Monitoring:** The inspection phase of maintenance provides important data on the effectiveness of drainage elements and the need for further maintenance. Inspection practices include visual inspection and effectiveness testing. Visual inspection consists of inventorying outflow during storm events and assessing outlet condition. Outflow inventories are generally qualitative (e.g., high, moderate, low, or no flow). Visual inspection can be enhanced through the use of video cameras. Effectiveness testing can provide a more quantitative assessment of performance through the use of post-storm event monitoring with bucket sampling or direct upstream inflow coupled with downstream outflow measurements.

2. **Preventative Maintenance:** Preventative maintenance actions that promote good subsurface drainage system performance include: clean and seal joints and cracks, clean and verify the grade of outlet ditches, clean catch basins and other discharge points, and clean outlet screens and area around headwalls. Based on the results of the outlet inspection program, a routine outlet cleaning program should be implemented.

3. **Repair:** It is generally accepted that once pavement damage from blocked subsurface drainage is visible, the damage is irreversible, and that pavement life has been shortened. For this reason, any problems observed, no matter how minor in appearance, should be addressed immediately to confine the problems to a localized area.

4. **Continuous Monitoring and Feedback:** Monitoring is a continuous improvement process and improvements are achieved only through providing feedback to the design and construction groups. Thus maintenance should provide inspection results long with performance indicators to design and construction groups for review. Pavement management methodologies and maintenance strategies are reviewed in NCHRP Syntheses 222 and 223 (Zimmerman and ERES Consultants 1995 and Geoffroy 1996).

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Foundation Improvement and Stabilization

A. General Information

Soft subgrade and moisture-sensitive soils such as expansive soils, frost-prone soils, and collapsing soils present a construction challenge as well as a pavement performance challenge. Proper treatment of problem soils and the preparation of the foundation are important to ensure a long-lasting pavement structure that does not require excessive maintenance. Such soils can be stabilized to form a construction pad or a long-term subsurface layer capable of carrying pavement applied loads. In all cases, the provision for a uniform soil relative to textural classification, moisture, and density in the upper portion of the subgrade cannot be over emphasized. This uniformity can be achieved through soil sub-cutting or other techniques. Five techniques can be used to improve the strength and reduce the climatic variation of the foundation on pavement performance: stabilization of weak or moisture-sensitive soils, thick granular layers, subsurface drainage systems, geosynthetics, and soil encapsulation. Thick granular layers are generally greater than 18 inches in thickness and require readily accessible, good quality aggregates. Therefore, thick granular layers are seldom used in Iowa and will not be discussed further in this section.

B. Stabilization

Soil that is highly susceptible to volume and strength changes can cause severe roughness and accelerate the deterioration of the pavement structure in the form of increased cracking and decreased ride quality when combined with truck traffic. Generally, the strength and stiffness of some soils are highly dependent on moisture and stress state. In some cases, the subgrade soil can be treated with various materials to improve the strength and stiffness characteristics of the soil. Stabilization of soils is usually performed for two reasons:

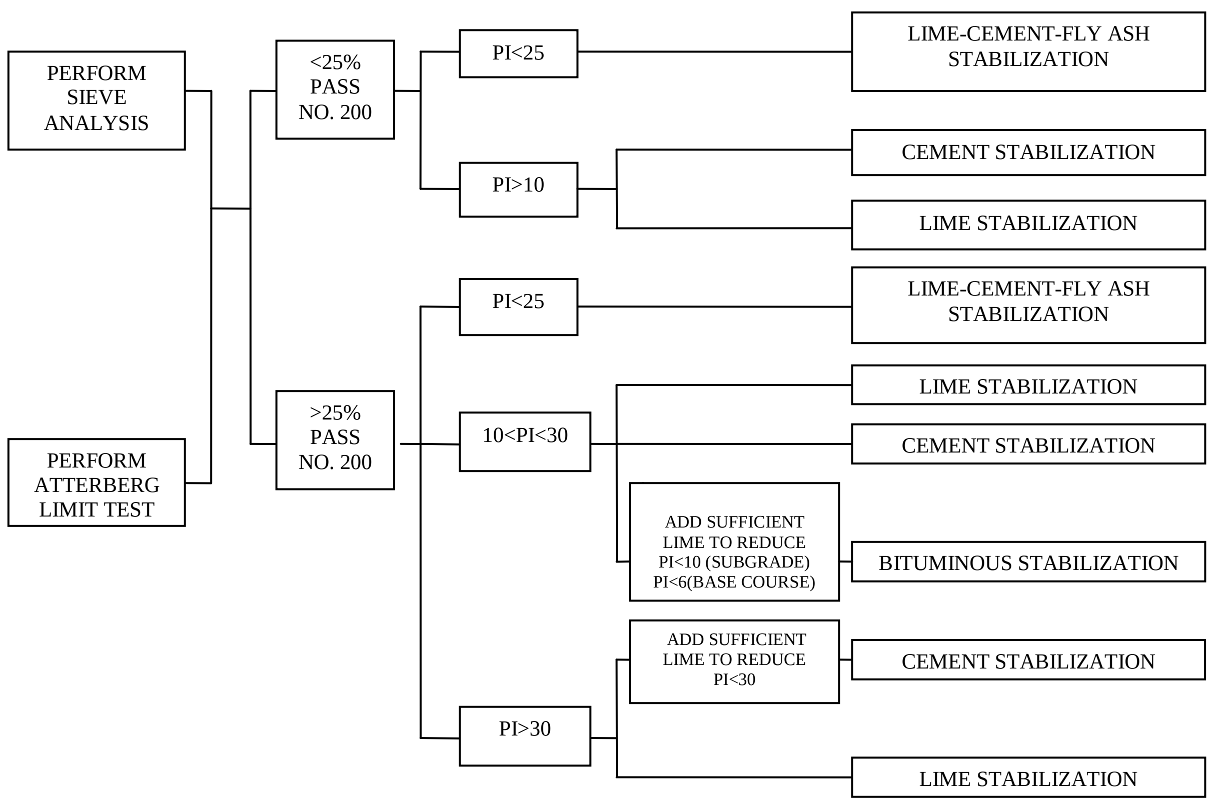
1. As a construction foundation to dry very wet soils and facilitate compaction of the upper layers. In this case, the stabilized soil is usually not considered as a structural layer in the pavement design process. This process is also sometimes referred to as soil modification.

2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil. In this case, the stabilized soil is usually given some structural value in the pavement design process.

   Lime, fly ash, cement, and asphalt stabilization have been used for controlling the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. For stabilization or modification of cohesive soils, hydrated lime is most widely used. Lime modification is used in many areas of the U.S. to obtain a good construction foundation in wet weather above highly plastic clays and other fine-grained soils. Lime is applicable in clayey soils (i.e., CH and CL type soils) and in granular soils containing clay binder (i.e., GC and SC), while Portland cement is more commonly used in non-plastic soils. Lime reduces the Plasticity Index (PI) and renders a clay soil less sensitive to moisture changes. The use of lime should be considered whenever the PI of the soil is greater than 10. It is important to note that changing the physical properties of a soil through chemical stabilization can produce a soil that is susceptible to frost heave.

Some basic definitions of soil modification and stabilization using lime, cement, and asphalt are provided below. Additional guidance on how stabilization is achieved using lime, cement, and asphalt can be found in TRB 1987; PCA 1995; and AI MS19, respectively. A flow chart for the determination of chemical treatment options for soil stabilization based on the percent passing the No. 200 sieve and the plasticity index of the soil is shown in Figure 6H-1.01.

Figure 6H-1.01: Selection of Stabilizer



Source: U.S. Department of Transportation 1976

a. Lime Treatment: Lime treatment or modification consists of the application of 1 to 3% hydrated lime to aid drying of the soil and permit compaction. As such, it is useful in the construction of a working foundation to expedite construction. Lime modification may also be considered to condition a soil for follow-up stabilization with cement or asphalt. Lime treatment of subgrade soils is intended to expedite construction, and no reduction in the required pavement thickness should be made.

b. Lime Stabilization: Lime stabilization of soils improves the strength characteristics and changes the chemical composition of some soils. The strength of fine-grained soils can be improved significantly with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective with highly plastic clay soils containing montmorillonite (expansive clay mineral).

Lime stabilization has been found to be an effective method to reduce the volume change potential of many soils. However, lime treatment of soils can convert soil that shows negligible-to-moderate frost heave potential into a soil that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. It has been reported that this adverse effect has been caused by an insufficient curing period accompanied by an inadequate compaction effort. Adequate curing is also important if the strength characteristics of the soil are to be improved.

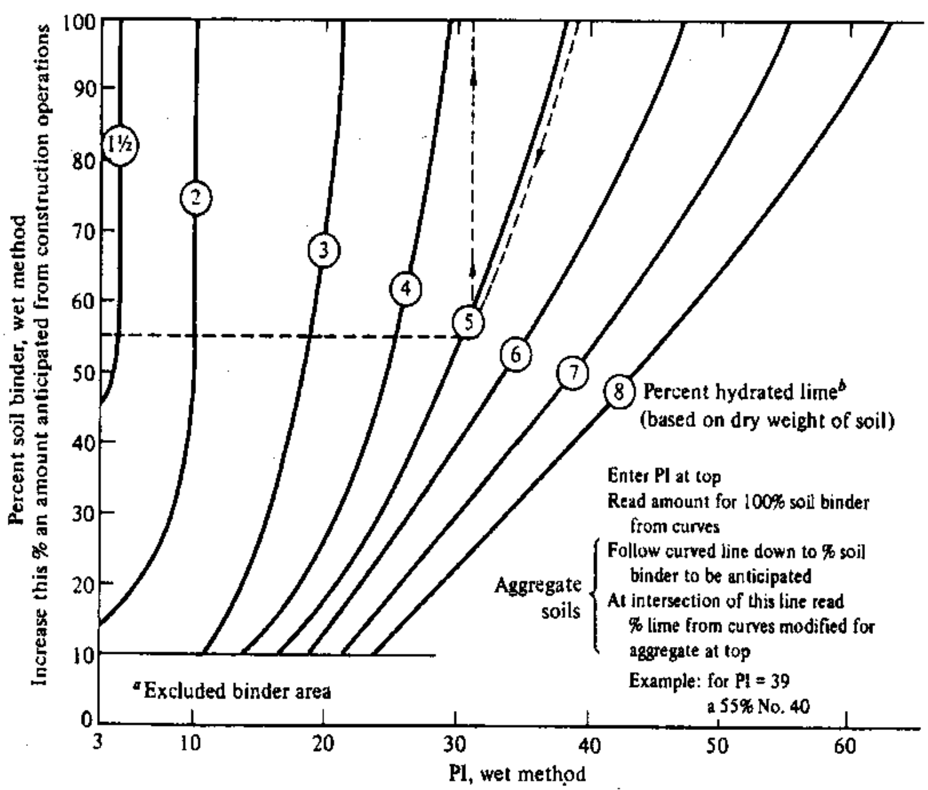
For successful lime stabilization of clay (or other highly plastic) soils, the lime content should be from 3 to 8% of the dry weight of the soil, and the cured mass should have an unconfined compressive strength increase of at least 50 psi after a 28 day curing period over the uncured material. The optimum lime content should be determined with the use of unconfined compressive strength and the Atterberg limits tests on laboratory lime-soil mixtures molded at varying percentages of lime. The lime-stabilized subgrade layer should be compacted to a minimum density of 95%, as defined by Standard Proctor density. The minimum strength requirement for this material is a function of pavement type and the importance of the layer within the pavement structure.

When soils are treated properly with lime, it has been observed that the lime-soil mixture may be subject to durability problems caused by the cyclic freezing and thawing of the soil.

Lime-fly ash stabilization is applicable to a broader range of soils because the cementing action of the material is less dependent on the fines contained within the soil. However, long-term durability studies of pavements with lime-fly ash stabilization are rather limited.

Soils classified as CH, CL, MH, ML, SM, SC, and GC with a plasticity index greater than 10 and with at least 25% passing the No. 200 sieve potentially are suitable for stabilization with lime. Hydrated lime, in powder form or mixed with water as slurry, is used most often for stabilization. Figure 2 can be used to estimate the design lime content for a subgrade. The quantities found from this chart should be used as a guideline, and laboratory testing mix design studies should be conducted for specific applications. Additional information can be obtained in the National Lime Association's Lime Stabilization Construction Manual (1972).

Figure 6H-1.02: Recommended Amounts of Lime for Stabilization of Subgrade and Bases



Source: National Lime Association 1972

a. **Cement Stabilization:** Portland cement is used widely for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties of strength and stiffness. Increasing the cement content increases the quality of the mixture. At low cement contents, the product is generally termed cement-modified soil. A cement-modified soil has improved properties of reduced plasticity or expansive characteristics and reduced frost susceptibility. At higher cement contents, the end product is termed soil-cement. Higher cement contents will unavoidably induce higher incidences of shrinkage cracking caused by moisture/temperature changes.

   For soils to be stabilized with cement, proper mixing requires that the soil have a PI of less than 20 and a minimum of 45% passing the No. 40 sieve. However, highly plastic clays that have been pre-treated with lime or fly ash are sometimes suitable for subsequent treatment. For cement stabilization of granular and/or non-plastic soils, the cement content should be 3 to 10% of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 150 psi within seven days. The Portland cement should meet the minimum requirements of AASHTO M 85. The cement-stabilized subgrade should be compacted to a minimum density of 95%, as defined by AASHTO T 134. Only fine-grained soils can be treated effectively with lime for marginal strength improvement.

b. **Asphalt Stabilization:** Generally, asphalt-stabilized soils are used for subbase construction. Use of asphalt as a stabilizing agent produces different effects, depending on the soil, and may be divided into three major groups:
   1) Sand-asphalt, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent
   2) Soil-asphalt, which stabilizes the moisture content of cohesive fine-grained soils
   3) Sand-gravel asphalt, which provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength. The durability of asphalt-stabilized mixtures generally can be assessed by measurement of their water absorption characteristics.

c. **Fly Ash Stabilization:** Fly ash and similar materials can be used in the stabilization of clay soils either in place of lime or cement or in combination with lime and cement. Generally, the use of fly ash and similar materials reduces the shrink-swell properties of the soils. Additionally, the act of drying the soil facilitates soil compaction. These materials are used with clay-type soils that are above the optimum water content.

3. **Characteristics of Stabilized Soils:** The improvement of subgrade or unbound aggregate by application of a stabilizing agent is intended to cause the improvements outlined above. These improvements arise from several important mechanisms that must be considered and understood by the pavement designer. Subgrade stabilizing agents may fill or partially fill the voids between the soil particles. This reduces the permeability of the soil by increasing the tortuosity of the pathways for water to migrate through the soil. Reduction of permeability may be relied upon to create a waterproof surface to protect underlying, water-sensitive soils from the intrusion of surface water. This mechanism must be accompanied by other aspects of the geometric design into a comprehensive system. The reduction of void spaces may also tend to change the volume change under shear from a contractive to a dilative condition. The stabilizing agent also acts by binding the particles of soil together, adding cohesive shear strength, and increasing the difficulty with which particles can move into a denser packing under load. Particle binding serves to reduce swelling by resisting the tendency of particles to move apart. The particles may be bound together by the action of the stabilizing agent itself (as in the case of asphalt cement), or may be cemented by byproducts of chemical reactions between the soil and stabilizing agent (as in the case of lime or portland cement).

The zone that may be selected for improvement depends upon a number of factors. Among these are the depth of soft soil, anticipated traffic loads, the importance of the transportation network, and the drainage characteristics of the geometric design and the underlying soil. When only a thin zone is subject to improvement, removal and replacement will usually be the preferred alternative by most agencies, unless a suitable replacement soil is not economically available. The zone can be described as thick or thin, based primarily on the economics of the earthwork requirements and the depth of influence for the vehicle loads.

4. **Pavement Design Considerations for Stabilized Subgrades:** The application of the stabilizing agent will usually increase the strength properties of the soil. This increase will generally appear in the pavement design process as an increase in the modulus of the improved soil, reducing the pavement structural layer thicknesses. The cost of the stabilization process, therefore, can be offset by savings in the pavement structural layers. However, it is important that the actual increase used in the design process be matched in the constructed product, making construction quality control and quality assurance programs very important. When pavement design is performed using only a single parameter to describe the subgrade condition, the thickness of the stabilized zone is a critical component in determining the increased modulus to use in design.

   The thickness of the improved subgrade zone is both a design and a construction consideration. From the design standpoint, it would obviously be advantageous to stabilize and improve the properties of a zone as thick as may be reasonably stabilized. From a constructability perspective, there are practical and economic implications related to the thickness of the stabilized zone. Stabilization requires that the agent be thoroughly distributed into the soil matrix, and that the soil matrix must be well pulverized to prevent unimproved clumps from remaining isolated within the mass. The construction equipment used to mix must be capable of achieving high levels of uniformity throughout the depth of desired improvement. If the zone to be improved is very thick, it may be necessary to process the stabilized soil in multiple lifts, which will usually require the stripping and stockpiling of upper lifts within the subgrade. Stabilization therefore rarely exceeds a few inches in depth in transportation applications, except for deep mixing applications that might be used in the vicinity of bridge foundations or abutments to provide improved foundation support.

C. Subsurface Drainage

Subsurface drainage systems are used for three basic reasons:

- To lower the groundwater level
- To intercept the lateral flow of subsurface water beneath the pavement structure
- To remove the water that infiltrates the pavement's surface

Deep subdrains (below frost line) are usually installed to handle groundwater problems. The design and placement of these subdrains should be handled as part of the geotechnical investigation of the site. Edgedrains placed in trenches under the shoulders at shallower depths are used to handle water infiltrating the pavement from above. The design and placement of these drainage systems is discussed in Section 6G-1 - Subsurface Drainage Systems.

D. Geosynthetics

Geosynthetics are a class of geomaterials that are used to improve soil conditions for a number of applications. They consist of manufactured polymeric materials used in contact with soil materials or pavements as an integral part of a man-made system (ASTM D 4439). The most common applications in general use are in pavement systems for both paved and unpaved roadways, for reinforcing embankments and foundation soils, for creating barriers to water flow in liners and cutoffs, and for improving drainage. The generic term "geosynthetic" is often used to cover a wide range of different materials, including geotextiles, geogrids, and geomembranes. Combinations of these materials in layered systems are usually called geocomposites.

1. Materials:

a. **Geotextiles:** A geotextile, as defined by ASTM D 4439, is "a permeable geosynthetic comprised solely of textiles." These materials are also known as engineering fabrics. Fabrics are usually created from polymers, most commonly polypropylene, but also potentially including polyester, polyethylene, or nylon (Koerner 1998). Geotextiles are usually classified by their manufacturing process as either woven or non-woven. Both kinds of geosynthetics use a polymer fiber as raw material. Depending on the application, the fibers may be used singly or spun into yarns by wrapping several fibers together, or created by a slit film process. Woven geosynthetics are manufactured by weaving fibers or yarns together in the same way as any form of textile, although generally only fairly simply weaving patterns are used. Non-woven geosynthetics are made by placing fibers in a bed, either in full-length or in short sections. The fibers are then bonded together, either by raising the temperature, applying an adhesive chemical, or by mechanical means (usually punching the bed of fabric with barbed needles, in essence, tangling them into a tight mat).

b. **Geogrids:** Geogrids, as their name suggests, consist of a regular grid of plastic with large openings (called apertures) between the tensile elements. The function of the apertures is to allow the surrounding soil materials to interlock across the plane of the geogrid; hence, the selection of the size of the aperture is partially dependent on the gradation of the material into which it will be placed. The geogrid is manufactured using high-density polymers of higher stiffnesses than are common for geotextiles. These polymers are then punched in a regular pattern and drawn in one or two directions. Alternatively, a weaving process may be used in which the crossing fibers are left wide apart and the junctions between them are reinforced.

c. **Geomembranes:** Geomembranes are used to retard or prevent fluid from penetrating the soil and as such consist of continuous sheets of low permeability materials. These materials are made by forming the polymer into a flat sheet, which may have a roughened surface created to aid in the performance of the membrane by increasing friction with the adjacent soil layer.

Several other kinds of geosynthetic materials may be made by slight variations of these general types. For example, geonets are similar in appearance to geogrids but are manufactured slightly differently so that the individual elements of the geonet are at acute angles to each other. These materials are usually used in drainage applications.

d. **Geocomposites:** Geocomposite materials are often created by combining two or more of the specific types of products described previously to take advantage of multiple benefits. Further, geocomposites may be formed by combining geosynthetics with more traditional geomaterials, the most common example being the geosynthetic clay liner. A geosynthetic clay liner consists of a layer of bentonite sandwiched together with geomembrane or geotextile materials to create a very low permeability barrier.

2. **Applications:** There are six widely recognized functions for geosynthetic applications as shown across the top of Table 6H-1.01. The typical classes of geosynthetic used for each function are also shown. Although the table indicates only primary functions, most geosynthetic applications call for the material to satisfy at least one secondary function as well (e.g., a separation layer under a pavement may also be required to reinforce the subgrade and influence drainage under the pavement).

   Table 6H-1.02 provides a summary of the most commonly used geosynthetic functions for transportation applications. Comparison of Tables 6H-1.01 and 6H-1.02 reveals that the geotextile and geogrid materials are the most commonly used in transportation, although certainly others are sometimes used. This generality is more accurate when only the pavement itself (not including the adjoining fill or cut slopes, retaining walls, abutments, or drainage facilities) is considered. The most common usage for geosynthetics in the United States has historically been for unpaved roads but use in paved, permanent roads is increasing.

   Each of these functional classes, while potentially related by the specific application being proposed, refers to an individual mechanism for the improvement of the soil subgrade. The separation function describes the maintenance of materials of different gradations as separate and distinct materials. In the specific case of the pavement application, separation relates to the maintenance of unbound granular base course materials as distinct from the subgrade (Koerner 1998; Christopher and Holtz 1991).

   These materials may tend to become mixed in service due to pumping of the subgrade into the subbase, or due to localized bearing capacity failures leading to migration of aggregate particles into the subgrade (TRB 1987). This potential behavior has been confirmed in the field, as well as the ability of geosynthetic materials to resist it (Macdonald and Baltzer 1997; McKeen 1976). Once the unbound subbase is mixed with the subgrade, its strength and drainage properties may be detrimentally affected.

Table 6H-1.01: Functions of Geosynthetic Materials

Geosynthetic Materials	**Function**					
	Filtration	***Drainage***	***Separation***	***Reinforcement***	***Fluid Barrier***	***Protection***
Geotextile	x	x	x	x		x
Geogrid			x	x		
Geomembrane					x	
Geonet		x				
Geocomposites:						
Geosynthetic Clay Liner					x	
Thin Film Geotextile Composite					x	
Field Coated Geotextile					x	

Source: Laguros and Miller 1997.

Table 6H-1.02: Transportation Uses of Geosynthetic Materials

Function	Specific Use
Filtration	• Beneath aggregate subbase for paved and unpaved roads and airfields or railroad ballast
Drainage	• Drainage interceptor for horizontal flow • Drain beneath other geosvnthetic systems
Separation (of dissimilar materials)	• Between subgrade and aggregate subbase in paved and unpaved roads and airfields • Between subgrade and ballast for railroads • Between old and new asphalt layers
Reinforcement (of weak materials)	• Over soft soils for unpaved roads, paved roads, airfield, railroads, construction foundations

Source: Koerner 1998

a. Reinforcement Function: The reinforcement function is very similar to the reinforcement process in reinforced concrete elements. The geosynthetic is introduced to provide elements with tensile resistance into the unbound material, which on its own would exhibit very low tensile resistance. The specific improvements imparted to pavement designs include the potential for improved lateral restraint of the subbase and subgrade, modifications of bearing capacity failure surfaces, and tensile load transfer under the wheel load. The lateral restraint arises as the subbase material tends to move outward under load beneath the wheel. The geosynthetic tends to be pulled along as a result of friction or interlock with the aggregate particles, and resists that tendency through its own tensile strength. The particles are therefore held in place as well. Bearing capacity surfaces may be forced to remain above the geosynthetic, in the stronger base course. Finally, the tendency of the subbase to bend under the wheel loads introduces tensile stress at the subbase/subgrade interface, which may be taken by the geosynthetic. Careful consideration must be given to the mobilization behavior of the geosynthetic, which may require fairly large strains to provide the desired reinforcement.

c. Filtration Function: The filtration function is similar to the separation function, but in this case the reason for mixing or migration of particles is the seepage forces induced by water flowing through the unbound material. The function of the filter is to provide a means to allow water to flow through unbound material without excessive loss of soil due to seepage forces, and without clogging (Koerner 1998). Zonal filters may offer the same protection, but may be less convenient or practical to install. The drainage function is related to the filtration function, in that once again the desired behavior is the movement of water out of or through the unbound material with sufficient maintenance of the fine particles in place. The difference arises in the focus and intent; filtration applications tend to be predicated on the maintenance of the soil, while drainage applications tend to attach more importance to the quantity of flow to be maintained or the desired reduction in pore water pressure. Further, the drainage function may be carried out by designing for drainage along the plane of the geotextile itself, rather than through surrounding unbound material.

The specific function to be provided by the geosynthetic in transportation applications is a function of the soil conditions. Table 6H-1.03 indicates that the following functions most commonly arise as a function of the soil strength.

Table 6H-1.03: Function of the Geosynthetic vs. Subgrade Properties

S_u (kPa)[1]	CBR	Function
60-90	2-3	Filtration, some separation
30-60	1-2	Filtration, separation, some reinforcement
<30	Below 1	Filtration, separation, reinforcement

[1] S_u (kPa) = undrained shear strength (1 kPa = 20.89 psf)

Source: Holtz et al. 1998

The range of functions potentially served by the geosynthetic thus increases as the subgrade strength decreases. In all cases reported in Table 6H-1.03, the soil conditions are rather poor. Table 6H-1.04 indicates that geosynthetics are most appropriate under the conditions outlined.

Table 6H-1.04: Appropriate Conditions for Geosynthetic Use

Condition	Related Measures
Poor soils	USCS: SC, CL, CH, ML, MH, OH, or PT soils; or AASHTO: A-5, A-6, A-7, or A7-6 soils
Low strength	S_u <13 kPa, CBR <3, or MR <4500 psi
High water table	Within zone of influence of surface soils
High sensitivity	High undisturbed strength compared to remolded strength

Source: Holtz et al. 1998

3. **Design Considerations:** Koerner describes three potential design approaches: design by cost, design by specification, and design by function, to design geosynthetics for engineering application. Additional information on these design approaches can be found in Koerner 1998.

E. Soil Encapsulation

Soil encapsulation is an embankment placement technique that has been used to protect moisture sensitive soils from large variations in moisture content. However, this technique is rarely used to improve the foundations of higher-volume roadways. It is more commonly used as a foundation or subbase layer for low-volume roadways, where the import of higher-quality embankment materials is restricted from a cost standpoint. The concept of soil encapsulation is to keep the fine-grained soils at or slightly below optimum moisture content, where the strength of these soils can support heavier trucks and traffic. See Section 6D-1 - Embankment Construction, for placement of unsuitable soils within embankment sections.

F. Moisture Conditioning

Table 6H-1.05 shows the relationship between optimum moisture content and density/strength of Iowa soils. For gaining maximum dry density and better compressive strength of soil, the water content should be kept at or around optimum moisture content. The SUDAS Specifications require a moisture content between optimum and 4% above optimum moisture for prepared subgrades.

According to ASTM D 698 Method A, a wide range of maximum densities and optimum moisture contents were determined. Table 6H-1.05 shows the typical relationships between optimum moisture contents and density/strength of some Iowa soils.

Table 6H-1.05: Typical Optimum Moisture Contents and Density/Strengths

Soil	Optimum Moisture Content (%)	Maximum Dry Unit Weight γ_d, (pcf)	Unconfined Compressive Strength at Optimum Moisture Content (psi)
Paleosol	17.0	106.7	48
Alluvium	19.8	102.6	44
Glacial Till	12.5	118.4	44
Le Grand Loess	17.2	106.1	44
Turin Loess	16.6	105.2	33

Source: White, et al. 2005

G. Granular Subbases

Granular subbases are used as a substitute for subgrade materials in regions having poor soils (i.e., high moisture content fine-grained soils) when the subgrade is not treated with another chemical or mechanical stabilizer. The granular subbase provides additional load bearing strength directly below the pavement, reduces the stress applied to the subgrade, provides drainage for the pavement system, and provides a uniform, stable construction platform. See Section 6F-1 - Pavement Subbase Design and Construction, for more information.

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CHAPTER 7
Erosion and Sediment Control

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General Information

A. Purpose

In an effort to protect the nation's waters from pollution, the Environmental Protection Agency (EPA) has developed a system of regulations, under the Clean Water Act, entitled the National Pollutant Discharge Elimination System (NPDES). These regulations require the owner of a site to obtain a permit prior to construction, and ensure that proper steps are taken during construction to help prevent erosion and prevent sediment from leaving construction sites.

The purpose of this chapter is to explain the erosion and sedimentation process, and describe methods that may be used to protect natural resources by reducing both. In addition, the steps required to comply with the EPA's NPDES regulatory requirements will be explained.

B. Background

Approximately 40% of the nation's waterways have been identified as impaired, meaning that they do not meet the water quality standards for their intended use. In Iowa, the most common reason for a waterway to be designated as impaired is due to high levels of suspended sediment. One of the main sources of suspended sediment found in waterways is construction site stormwater runoff.

Sediment in runoff from construction sites is a direct result of the erosion created when the site is stripped of its stabilizing vegetation. According to the U.S. Environmental Protection Agency (EPA), sediment rates in stormwater runoff from construction sites are typically 10 to 20 times greater than for agricultural lands. In urban areas, stormwater runoff is quickly intercepted by the storm sewer and does not have a chance to travel over vegetated areas where suspended sediment can be removed. The sediment in this runoff eventually reaches streams, ponds, lakes, and rivers. High levels of suspended sediment can quickly form large silt deposits, filling in these waterways.

Silt deposits can cause damage to public and private property. Silt deposits often flow onto adjacent property, causing damage; flow onto public streets, creating mud and dust; and clog storm sewers and ditches. The financial impact caused by this sediment is substantial. Waterways must be dredged to remove the deposits, streets swept, ditches and storm sewers cleaned, and property damaged by sediment must be repaired or replaced. While the financial impacts are significant, the loss of natural resources is just as important.

High levels of sediment in storm runoff can impact an ecosystem. Sediment is often contaminated with other pollutants such as phosphorous, nitrates, pesticides, and heavy metals. In high enough concentrations, these pollutants become poisonous. In addition, the suspended sediment filters out sunlight, killing off aquatic vegetation. Many species of animals depend on this vegetation for habitat and nourishment. If enough vegetation is destroyed, the levels of dissolved oxygen in the water can drop to levels where aquatic wildlife cannot survive. Waterfowl and other animals, dependent on the fish and vegetation for sustenance, may die or leave the area.

C. Definitions

Aggregate: Crushed rock or gravel screened to different sizes for various uses in construction projects.

Annual Plant: A plant that completes its life cycle and dies in one year or less.

Apron: A floor or lining to protect a surface from erosion. Normally at the inlet or outlet of a storm conduit.

Berm: A raised area that breaks the continuity of a slope.

Best Management Practices: Schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or reduce the pollution of waters of the United States. BMPs also include treatment requirements, operating procedures, and practices to control plant site runoff, spillage or leaks, sludge or waste disposal, or drainage from raw material storage.

Channel: A stream that conveys continuous or intermittent water; a ditch or channel excavated for the flow of water.

Channel Stabilization: Erosion prevention and stabilization of velocity distribution in a channel using jetties, drops, revetments, vegetation, and other measures.

Clay (Soils): (1) A mineral soil separate consisting of particles less than $6.6x10^{-6}$ feet (0.002 mm) in equivalent diameter. (2) A soil textural class. (3) A fine-grained soil that has a high plasticity index in relation to the liquid limits.

Clean Water Act (CWA): The Federal Water Pollution Control Act.

Clod: A compact, coherent mass of soil 2 inches or larger, produced artificially, usually by digging, etc., especially when these operations are performed on soils that are either too wet or too dry for normal soil movement.

Code of Federal Regulations (CFR): A codification of the final rules published daily in the Federal Register. Title 40 of the CFR contains the environmental regulations.

Compaction: The process by which the soil grains are rearranged to decrease void space and bring the grains into closer contact with one another and thereby increase the weight of solid material per cubic foot.

Construction Site: A site or common plan of development or sale on which construction activity, including clearing, grading, and excavating, results in soil disturbance. A construction site is considered one site if all areas of the site are contiguous with one another and one entity owns all areas of the site.

Contour: An imaginary line on the surface of the ground connecting points of the same elevation.

Cover: (1) Vegetation or other material providing protection. (2) Ground and soils: any vegetation producing a protective mat on or just above the soil surface. (3) Stream: generally trees, large shrubs, grasses, and forbs that shade and otherwise protect the stream from erosion, temperature elevation, or sloughing of banks. (4) Vegetation: all plants of all sizes and species found on an area, regardless of whether they have forage or other value. (5) Artificial: any material (natural or synthetic) that is spread or rolled out over the ground to protect the surface from erosion.

Detention Basin (Pond): A structure barrier built to divert part or all of the runoff water from a land area and to release the water under a controlled condition.

Drainage: The removal of excess surface water or groundwater from a land area.
Erosion: (1) The wearing away of the land surface by running water, wind, ice, or other geological agents, including such processes as gravitational creep. (2) Detachment and movement of soil or rock fragments by water, wind, ice, or gravity.

Filter Strip: Strip of vegetation above ponds, diversion structures, or other elements to retard flow of runoff water and thereby reduce sediment flow.

Final Stabilization: Period when all soil-disturbing activities at the site have been completed and a uniform perennial vegetative cover with a density of 70% for the area has been established, or equivalent stabilization measures have been employed, or which has been returned to agricultural production.

Gabion: A rectangular or cylindrical wire mesh cage filled with rock and used as a protecting apron, revetment, retaining wall, etc., against erosion.

General Permit: An NPDES permit issued under 40 CFR 122.28 that authorizes a category of discharges under the CWA within a geographical area. A general permit is not specifically tailored for an individual discharger.

Grade: (1) The slope of a road, channel, or natural ground, or any surface prepared for the support of construction such as paving. (2) To finish the surface of a roadbed, top of embankment, or bottom of excavation.

Grass: A member of the botanical family Gramineae characterized by blade-like leaves arranged on the culm or stem in two ranks.

Grassed Channel (Waterway): A natural or constructed waterway, usually broad and shallow, covered with erosion-resistant grasses, used to conduct surface water from land.

Ground Cover: Grasses or other plants grown to keep soil from being blown or washed away.

Gully: A channel or miniature valley cut by concentrated runoff, through which water commonly flows only during and immediately after heavy rains, or during the melting of snow. The gullies may be branching or linear, rather long, narrow, and of uniform width. The difference between a gully and rill is the depth. A gully is sufficiently deep that it would not be obliterated by tillage operations. A rill of lesser depth can be smoothed by regular tillage equipment.

Infiltration: The gradual downward flow of water from the surface through soil to ground water and water table reservoirs.

Large Construction Activity: As defined in 40 CFR 122.26(b)(14)(x), a large construction activity includes clearing, grading, and excavating, resulting in a land disturbance that will disturb five acres or more of land, or will disturb fewer than five acres of total land area, but is part of a larger common plan of development or sale that will ultimately disturb five acres or more. Large construction activity does not include routine maintenance that is performed to maintain the original line and grade, hydraulic capacity, or original purpose of the site.

Legume: A member of the Leguminosae family, one of the most important and widely distributed plant families. Leaves are alternate, have stipples, and are usually compound. Most legumes are nitrogen-fixing plants.

Loess: Soil material transported and deposited by wind and consisting predominantly of silt-sized particles.

Mulch: A natural or artificial layer of plant residue, or other materials such as straw, leaves, bark, sand, or gravel on the soil surface, to protect the soil and plant roots from the effects of raindrops, soil crusting, freezing, evaporation, etc.

Municipal Separate Storm Sewer System (MS4): A conveyance or system of conveyances (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, man-made channels, or storm drains) meeting the following criteria:

a. Owned and operated by a state, city, town, borough, county, parish, district, association, or other public body (created by or pursuant to state law) having jurisdiction over disposal of sewage, industrial wastes, stormwater, or other wastes, including special districts under state law such as a sewer district, flood control district or drainage district, or similar entity, or an Indian tribe or an authorized Indian tribal organization, or a designated and approved management agency under section 208 of the Clean Water Act (CWA) that discharges to waters of the United States.
b. Designed or used for collecting or conveying stormwater
c. Which is not a combined sewer
d. Which is not part of a publicly owned treatment works (POTW)

National Pollutant Discharge Elimination System (NPDES): National program under Section 402 of the Clean Water Act for regulation of discharges of pollutants from point sources to waters of the United States. Discharges are illegal unless authorized by an NPDES permit.

Nurse Crop: Seeding of a short-life crop with a permanent species to aid in erosion control until the permanent species are established.

Organic Matter: Decomposition products of plant and animal materials, such as litter, leaves, and manure.

Perennial Plant: A plant that normally lives three years or longer.

Permeability, Soil: The quality of a soil horizon that enables water or air to move through it. The permeability of a soil may be limited by the presence of one nearly impermeable horizon even though the others are permeable.

Permitting Authority: The United States EPA, a Regional Administrator of the EPA, or an authorized representative. Under the Clean Water Act, most states are authorized to implement the NPDES permit program.

pH: A measure of hydrogen ion concentration. Values range from 0 to 14; a pH of 7.0 is neutral. All pH values below 7.0 are acidic, and all above 7.0 are alkalinic.

Planting Season: The period of the year when planting or transplanting is considered advisable from the standpoint of successful establishment.

Point Source: Any discernible, confined, and discrete conveyance, including but not limited to, any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock concentrated animal feeding operation (CAFO), landfill leachate collection system, vessel, or other floating craft from which pollutants are or may be discharged. This term does not include return flows from irrigated agriculture or agricultural stormwater runoff.

Pollutant: Dredged spoil, solid waste, incinerator residue, filter backwash, sewage, garbage, sewage sludge, munitions, chemical wastes, biological materials, radioactive materials (except those regulated under the Atomic Energy Act of 1954, as amended (42 U.S.C. 2011 et seq.)), heat, wrecked or discarded equipment, rock, sand, cellar dirt, and industrial, municipal, and agricultural waste discharged into water.

Qualified Personnel: Those individuals capable enough and knowledgeable enough to perform the required functions adequately well to ensure compliance with the relevant permit conditions and requirements of the Iowa Administrative Code.

Receiving Water: The "Water of the United States" as defined in 40 CFR 122.2 into which the regulated stormwater discharges.

Revetment: Facing of rip rap, or other material, either permanent or temporary, placed along the edge of a stream to stabilize the bank and protect it from the erosive action of the stream.

Rip Rap: Broken rock, cobbles, or boulders placed as revetment on earth surfaces such as the face of a dam or the bank of a stream, for the protection against the action of water or waves.

Runoff: That portion of the precipitation on a drainage area that is discharged from the area. Includes surface runoff and groundwater runoff.

Section 401 Certification: A requirement of Section 401(a) of the Clean Water Act that all federally issued permits be certified by the state in which the discharge occurs. The state certifies that the proposed permit will comply with state water quality standards and other state requirements.

Sediment: Solid material, both mineral and organic, that is in suspension, is being transported, or has been moved from its site of origin by air, water, gravity, or ice.

Seed: The fertilized and ripened ovule of a seed plant that is capable, under suitable conditions, of independently developing into a plant similar to the one that produced it.

Seedbed: The soil prepared by natural or artificial means to promote the germination of seed and the growth of seedlings.

Small Construction Activity: Clearing, grading, and excavating resulting in a land disturbance that will disturb one acre or more and fewer than five acres of total land area, but is part of a larger common plan of development or sale that will ultimately disturb five or fewer acres. Small construction activity does not include routine maintenance that is performed to maintain the original line and grade, hydraulic capacity, or original purpose of the site.

Sod: A closely knit ground cover growth, primarily of grasses, held together by its roots.

Soil Amendment: Any material, such as compost, lime, gypsum, sawdust, or synthetic conditioner that is worked into the soil to make it more productive. This term is used most commonly for added materials other than fertilizer.

Soil Horizon: A layer of soil, approximately parallel to the soil surface, with distinct characteristics produced by soil-forming processes.

Soil Structure: The combination or arrangement of primary soil particles into secondary particles or units. The secondary particles are characterized by size, shape, and degree of distinctness into classes, types, and grades.

Stormwater: Storm runoff, snow melt runoff, and surface runoff and drainage.

Stormwater Discharge Related Activities: Activities that cause, contribute to, or result in stormwater point source pollutant discharges, including excavation, site development, grading, and other surface disturbance activities; and measures to control stormwater, including the siting, construction, and operation of BMPs to control, reduce, or prevent stormwater pollution.

Stubble: The base portion of plants remaining after the top portion has been harvested.

Tacking: The process of binding mulch fibers together by adding a sprayed chemical compound.

Topsoil: The unconsolidated earthy material that exists in its natural state and is or can be made favorable to the growth of desirable vegetation. Usually the A-horizon of soils with developed profiles.

Uncontaminated Groundwater: Water that is potable for humans, meets the narrative water quality standards in subrule 567-61.3(2) of the Iowa Administrative Code, contains no more than half the listed concentration of any pollutants in subrule 567-61.3(3) of the IAC, has a pH of 6.5-9.0, and is located in soil or rock strata.

Vegetation: Plants in general or all plant life in the area.

Waters of the United States: All waters that are currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters subject to the ebb and flow of the tide. Waters of the United States include all interstate waters and intrastate lakes, rivers, streams (including intermittent streams), mudflats, sand flats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds.

Weed: An undesired uncultivated plant.

Wetlands: Areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Regulatory Requirements

A. National Pollutant Discharge Elimination System (NPDES)

The Clean Water Act established a set of requirements called the National Pollutant Discharge Elimination System (NPDES). The NPDES regulates stormwater discharges associated with industrial activities, municipal storm sewer systems, and construction sites. The purpose of these regulations is to reduce pollution of the nation's waterways. At the present time there are no specific loss monitoring requirements. Uses of Best Management Practices (BMP) identified in an approved Stormwater Pollution Prevention Plan (SWPPP) have been identified as the means and methods to meet the NPDES requirements. On-going discussions indicate that in the future where NPDES authorities determine that construction discharges have the reasonable potential to cause or contribute to a water quality standard excursion, numeric effluent limitations may be imposed. In the future, specific emphasis will be placed on containing soil erosion and minimizing soil compaction.

The intent of this section is to describe the regulations and permitting requirements of the NPDES as they relate to construction sites. Refer to Chapter 2 - Stormwater for additional information.

B. NPDES Construction Site Permitting

1. **Permit Requirements:** For construction projects, an NPDES permit from the Iowa DNR is required for any site that disturbs and exposes one acre of land or more. A permit is also required for projects that will disturb one or more acres as part of a common plan of development, even if there will not be one acre of disturbed ground exposed at any given time. In addition to the Iowa DNR, many local agencies also have a permit process. It is necessary to check with the Jurisdictional Engineer to determine what, if any, information is needed for the local agency permit.

   An example of a common plan of development would be a property owner who has two acres of land that he plans to divide up into four half-acre lots. Even though each half-acre lot will be graded and sold off individually, an NPDES permit is required because the grading of the individual lot is part of an overall plan to grade and develop two acres of land.

   Additional information regarding projects that require an NPDES permit can be obtained from the Iowa DNR's website.

2. **Permitting Process:** For most construction projects, coverage under the NPDES program will be obtained from the Iowa DNR through General Permit No. 2. The steps required to obtain coverage under this permit are as follows:

   a. **Prepare a Stormwater Pollution Prevention Plan:** A Stormwater Pollution Prevention Plan (SWPPP) describes the site and identifies potential sources of pollution. The SWPPP also provides a description of the practices that will be implemented to mitigate erosion and sediment loss from the site. The SWPPP must be prepared prior to submittal of the Notice of Intent. Detailed information on the required SWPPP content is provided later in this section.

b. **Publish a Public Notice:** Arrange for publication of a public notice of stormwater discharge that states the applicant's intention to file a Notice of Intent for coverage under the General Permit No. 2. This notice must be published for at least one day in the two newspapers with the largest circulation in the area of the discharge. A link to Iowa DNR for a copy of a typical public notice is contained in the Appendix.

c. **Notice of Intent:** Complete and sign a "Notice of Intent for NPDES Coverage Under General Permit" form. Note that there are specific restrictions on which individuals are authorized to sign the Notice of Intent (NOI). The Notice of Intent must be signed by an authorized individual (see Part VI.G of the NPDES permit for a list of individuals authorized to sign the permit). Also note that that the form contains an area to fill in information for a contact person. This is the person to whom all future correspondence will be sent. This person does not need to be the owner or other authorized signatory, but should be a person who will be involved with the project for the duration of the permitting period. A link to Iowa DNR for a Notice of Intent is contained in the Appendix.

   Acceptable proof of publication consists of an affidavit from the publisher or a newspaper clipping of the NOI that includes the date of publication and newspaper name.

   Construction may not be initiated until the Iowa DNR issues a construction authorization.

d. **Notice of Discontinuation:** The final step in the NPDES General Permit No. 2 process is to file a Notice of Discontinuation (NOD) with the Iowa DNR. The NOD ends the coverage of the site under the permit, relieving the permittees from the responsibilities of the permit and the possibility of enforcement actions against the permittees for violating the requirements of the permit.

   An NOD should be filed with the Iowa DNR within 30 days after the site reaches final stabilization. Final stabilization means that all soil-disturbing activities are completed, and that a permanent vegetative cover with a density of 70% or greater has been established over the entire site. It should be noted that the 70% requirement does not refer to the percent of the site that has been vegetated (i.e. 7 out of 10 acres). In order to file a Notice of Discontinuation, 100% of the disturbed areas of the site must be vegetated. The density of the vegetation across the site must be at least 70%. The NRCS Line-Transect method can be used to determine vegetation density if actual measurements are required.

   Like the Notice of Intent, the Notice of Discontinuation must be signed by an authorized individual and must contain a specific certification statement. A link to Iowa DNR for a Notice of Discontinuation is provided in the Appendix.

e. **Local Requirements:** As part of the NPDES regulations, some communities are required to review SWPPPs for land-disturbing activities that occur within their communities. Other communities may have elected to pass erosion and sediment control ordinances that must be adhered to. The designer should check with the local jurisdiction to determine if local requirements exist.

3. **Compliance with NPDES General Permit No. 2:** Once a Notice of Intent has been filed, activities at the site must comply with the requirements of NPDES General Permit No. 2. These requirements include:

   a. Implement pollution prevention practices as detailed on the SWPPP.

   b. Maintain the SWPPP and keep it current by noting significant changes.

c. Inspecting the site and pollution prevention measures at the required intervals.

d. Contractors and subcontractors, identified in the SWPPP, are required to sign on as co-permittees.

e. Note changes of ownership or transfer of the permit responsibilities.

f. Maintain copies of information on site.

g. Retain records for the required period.

C. Stormwater Pollution Prevention Plans (SWPPP)

1. **Purpose:** The NPDES General Permit No. 2 requires that a Stormwater Pollution Prevention Plan (SWPPP) be developed. The practices described in the SWPPP designed to reduce contamination of stormwater that can be attributed to activities on a construction site. Construction creates the potential for contamination of stormwater from many different sources. Grading removes protective vegetation, rock, pavement, and other ground cover, exposing the soil to the elements. This unprotected soil can erode and be carried off by stormwater runoff to lakes and streams. In addition, construction often involves the use of toxic or hazardous materials such as petroleum products, pesticides and herbicides, and building materials such as asphalt, sealants, and concrete, which may pollute stormwater running off of the site.

   The SWPPP must clearly identify all potential sources of stormwater pollution and describe the methods to be used to reduce or remove contaminants from stormwater runoff.

   The SWPPP is not intended to be a static document; rather it must be updated as necessary to account for changing site conditions that have a significant impact on the potential for stormwater contamination. The SWPPP must also be revised if the current plan proves to be ineffective at significantly minimizing pollutants.

2. **Preparation of a SWPPP:** The individual preparing the SWPPP should have a thorough understanding of the project and the probable sequence of construction operations.

   The process of preparing a SWPPP should begin by reviewing the existing site, and identifying the work required to complete the desired improvements. Next, the project should be broken down into major components or phases (e.g. clearing, grading, utility work, paving, home building, etc.). The specific phasing may vary for each project, depending on the scope of the work. On large projects with multiple areas that will be completed in stages, each stage of construction should be broken down separately.

   Next, a system of erosion and sediment controls should be designed for each phase of construction. The system of controls should take into account the anticipated condition of the site during each stage. For example, at the end of the grading phase, it is likely that the entire site will be stripped and highly vulnerable to erosion; temporary seeding and/or other stabilization practices may be the major control employed at this stage. At the end of the utility phase, the site may now have storm sewer and other drainage structures installed. This creates a direct route for sediment-laden runoff to easily leave the site. Implementing sediment retention may be an important control at this stage.

   An individual erosion or sediment control practice should not be utilized as the sole method of protection. Each phase of construction should incorporate multiple erosion and sediment control practices. Utilizing a variety of both erosion control and sediment control practices is an effective and efficient method of preventing stormwater pollution.

Once the phasing has been determined, and the methods of protection have been selected, a SWPPP can be developed. The following section summarizes the elements of a SWPPP that are required by General Permit No. 2.

3. **Required Content of the SWPPP:** Part IV of the Iowa DNR NPDES General Permit No. 2 contains a description of the specific items that must be included within the SWPPP. A summary of those items is provided below.

   a. **Site Description:** The first step in preparing a SWPPP is to provide a detailed description of the site. This description must include the following items:
      1) The nature of the construction activity (e.g. roadway construction, utility construction, single family residential construction, etc.) and major soil-disturbing activities (i.e. clearing, grading, utility work, paving, home building, etc.).
      2) An estimate of the total area of the project site and the area that is expected to be disturbed by construction.
      3) An estimate of the runoff coefficient for the site after construction (See Chapter 2 - Stormwater for determination of runoff coefficients).
      4) A summary of available information describing the existing soil and soil properties (e.g. type, depth, infiltration, erodibility, etc.).
      5) Information describing the quality of the stormwater runoff currently discharged from the site (required only if data exists, it is not necessary to collect and analyze runoff).
      6) The name of the receiving waters and ultimate receiving waters of runoff from the site. If the site drains into a municipal storm sewer system, identify the system, and indicate the receiving waters to which the system discharges.
      7) A site map that includes limits of soil-disturbing activities, existing drainage patterns, drainage areas for each discharge location (including off-site drainage), proposed grading, surface waters and wetlands, and locations where stormwater is discharged to surface water.
      8) Approximate slopes after major grading activities.
      9) The location of structural and nonstructural controls.
      10) The location of areas where stabilization practices are expected to occur.

   b. **Controls:** The plan needs to show what erosion and sediment controls and stormwater management practices will be used to reduce or eliminate contamination of stormwater by pollutants.
      1) Sequence: List the anticipated sequence of major construction activities and clearly describe the order for implementation of the control measures. It is not necessary to list anticipated dates for completion of the various stages of construction and implementation of practices; rather the SWPPP should indicate the stage of construction at which individual control measures are to be installed.
      2) Stabilizing Practices:
      - Describe the temporary and permanent stabilizing practices (protection of existing vegetation, surface roughening, seeding, mulching, compost blankets, Rolled Erosion Control Products (RECPs), sod, vegetative buffer strips, etc.).
      - Note that areas not subject to construction activity for 21 days or more must have stabilizing measures initiated within 14 days after construction activity has ceased.

      3) Structural Practices:
      - Describe any structural practices that will be used to divert flows away from disturbed areas, store runoff, limit erosion, or remove suspended particles from

runoff (silt fence, filter socks, diversion structures, sediment traps, check dams, slope drains, level spreaders, inlet protection, rip rap, sediment basins, etc.).

- For sites with more than 10 acres disturbed at one time, which drain to a common location, a sediment basin providing 3,600 cubic feet of storage per acre drained is required where attainable. When sediment basins of the size required are not attainable, other methods of sediment control that provide an equivalent level of protection are required.
- For disturbed drainage areas smaller than 10 acres, a sediment basin or sediment control along the sideslope and downslope boundaries of the construction area is required. The sediment basin should provide 3,600 cubic feet of storage per acre drained.
- Unless infeasible, the following measures should be implemented at all sites: utilize outlet structures that withdraw water from the surface when discharging from basins, provide and maintain natural buffers around surface waters, and direct stormwater to vegetated areas to both increase sediment removal and maximize stormwater infiltration.
- According to General Permit No. 2 Part IV.D.2.A.(2).(c), the permittee(s) shall minimize soil compaction and, unless infeasible, preserve topsoil. "Infeasible" shall mean not technologically possible, or not economically practicable and achievable in light of the best industry practices. "Unless infeasible, preserve topsoil" shall mean that, unless infeasible, topsoil from any areas of the site where the surface of the ground for the permitted construction activities is disturbed, shall remain within the area covered by the applicable General Permit No. 2 authorization. Minimizing soil compaction is not required where the intended function of a specific area of the site dictates that it be compacted. Preserving topsoil is not required where the intended function of a specific area of the site dictates that the topsoil be disturbed or removed. The permittee(s) shall control stormwater volume and velocity to minimize soil erosion in order to minimize pollutant discharges and shall control stormwater discharges, including both peak flowrates and total stormwater volume, to minimize channel and streambank erosion and scour in the immediate vicinity of discharge points. An affidavit signed by the permittee(s) may be submitted to demonstrate compliance.
- For construction activity that is part of a larger common plan of development, such as a housing or commercial development project, in which a new owner agrees in writing to be solely responsible for compliance with the provisions of this permit for the property that has been transferred or in which the new owner has obtained authorization under this permit for a lot or lots (as specified in subrule 567-64.6(6) of the Iowa Administrative Code), the topsoil preservation requirements described above must be met no later than at the time the lot or lots have reached final stabilization as described in this permit.
- In residential and commercial developments, a plat is considered a project. For other large areas that have been authorized for multiple construction sites, including those to be started at a future date such as those located at industrial facilities, military installations, and universities, a new construction project not yet surveyed and platted out is considered a project. This stipulation is intended to be interpreted as requiring the topsoil preservation requirements on development plats and construction activities on other extended areas that may have several construction projects allowed under the same authorization to be implemented on those projects not yet surveyed and platted out prior to October 1, 2012, even if other plats and construction activities in the same development or other extended area were authorized prior to October 1, 2012.

4) Stormwater Management:

- Describe the features that will be installed during construction to control pollutants in stormwater after construction operations are completed.
- Pollutant removal features may include detention/retention ponds, vegetated swales, and infiltration practices.
- Post-construction erosion control features may include channel protection/lining and velocity dissipation at outlets.

5) Other Controls:

- Note in the SWPPP that any waste materials from the site must be properly disposed of.
- Describe practices for preventing hazardous materials that are stored on the site from contaminating stormwater.
- Describe a method to limit the off-site tracking of sediment by vehicles.
- Define construction boundaries to limit the disturbance to the smallest area possible.
- Identify areas to be preserved or left as open space.

6) State and Local Requirements:

- List additional state or local regulations that apply to the project. Note that some local jurisdictions may have an erosion and sediment control ordinance. The requirements of this ordinance must be listed in the SWPPP.
- List any applicable procedures or requirements specified on plans approved by state or local officials.
- Section 161A.64 of the Code of Iowa requires that prior to performing any “land-disturbing” activity (not including agricultural activities), a signed affidavit must be filed with the local Soil and Water Conservation District stating that the project will not exceed the soil loss limits stated. It should be noted that this requirement is not a condition of the NPDES General Permit No. 2.

c. Maintenance: The SWPPP must describe the maintenance procedures required to keep the controls functioning in an effective manner. For each type of erosion or sediment control practice utilized, a description of the proper methods for maintenance must be provided. In addition, maintenance should include removal of sediment from streets, ditches, or other off-site areas.

d. Inspections: The SWPPP must describe the inspection requirements of General Permit No. 2. Inspections are required every 7 calendar days. Check local agency regulations for permit inspection and reporting requirements. The inspections must include the following:

1) Inspect disturbed areas and areas used for storage of materials for evidence of pollutants leaving the site and/or entering the drainage system.
2) Inspect erosion and sediment control measures identified in the SWPPP to ensure they are functioning correctly.
3) Inspect discharge locations to ascertain if the current control measures are effective in preventing significant impacts to the receiving waters.
4) Inspect locations where vehicles enter/exit the construction site for signs of sediment tracking.
5) Prepare an inspection report that lists the date, the name of the inspector, and the inspector’s qualifications. The report must summarize the inspection and note any maintenance of the controls or changes to the SWPPP that are required.
6) Implement required maintenance or changes to the SWPPP identified during the inspection within seven calendar days following the inspection.

The Project Engineer should note that SUDAS Specifications Section 9040 provides for two bid items related to the SWPPP. The first relates to the Contractor preparing the SWPPP. The second bid item involves management of the SWPPP, which includes the actions necessary to comply with the General Permit No. 2, conduct regular inspections, documentation, updates to the SWPPP, and filing of the Notice of Discontinuation.

e. Non-stormwater Discharges: Various non-stormwater related flows are allowed to be discharged into the stormwater system, provided that they are not contaminated by detergents or spills/leaks of toxic/hazardous materials. Allowable non-stormwater discharges include flows from fire hydrant and potable waterline flushing, vehicle washing, external building washdown that does not use detergents, pavement washwater where spills or leaks of toxic or hazardous materials have not occurred, air conditioning condensate, springs, uncontaminated groundwater, and footing drains. When there is a possibility for these types of discharges on the site, they must be identified in the SWPPP and include a description of the measures that will be implemented to prevent these flows from becoming contaminated by hazardous materials or sediment.

f. Contractors: The SWPPP must clearly identify all of the contractors or subcontractors that will implement each measure in the plan. Each contractor or subcontractor identified is required to sign a certification statement making them a co-permittee with the owner and other contractors. The certification must read as follows:

"I certify under penalty of law that I understand the terms and conditions of the general National Pollutant Discharge Elimination System (NPDES) permit that authorizes the stormwater discharges associated with industrial activity from the construction site as part of this certification. Further, by my signature, I understand that I am becoming a co-permittee, along with the owner(s) and other contractors and subcontractors signing such certifications, to the Iowa Department of Natural Resources NPDES General Permit No. 2 for "Storm Water Discharge Associated with Industrial Activity for Construction Activities" at the identified site. As a co-permittee, I understand that I, and my company, am legally required under the Clean Water Act and the Code of Iowa, to ensure compliance with the terms and conditions of the stormwater pollution prevention plan developed under this NPDES permit and the terms of this NPDES permit."

Under most circumstances, the identity of the contractor and any subcontractors implementing the pollution prevention measures will not be known at the time of SWPPP preparation. The SWPPP should provide a blank certification form and a location to identify who will be responsible for implementing each pollution prevention measure. The contractor responsible for maintaining the SWPPP can then complete this information, as it becomes available.

D. Who is Responsible

1. **Property Owner:** Coverage under the NPDES General Permit No. 2 is granted to the property owner. The property owner has the ultimate responsibility for ensuring that the conditions of the permit are met. Enforcement actions associated with non-compliance with the permit are normally directed against the property owner.

2. **Designer:** The project designer typically prepares the initial SWPPP, although the contractor may be required to develop the SWPPP and obtain the NPDES permit if so directed in the contract documents. The designer may continue to review and approve changes to the SWPPP (on behalf of the owner).

3. **Jurisdiction:** On public improvement projects, the Jurisdiction serves as the owner of the site (see requirements for owners above).

   According to Iowa DNR regulations, certain MS4 jurisdictions are required to conduct inspections on public construction projects that require coverage under an NPDES permit. Under most circumstances, these inspections must be conducted utilizing the MS4's own staff. The contractor is not allowed to perform these inspections. The purpose of these inspections is to ensure that contractors are correctly implementing the BMPs identified in the SWPPP and to ensure that the jurisdiction maintains an active role in preventing stormwater contamination from its public improvements projects.

   The inspections by the jurisdiction must be conducted every 7 days. These jurisdictional inspections may also be used to satisfy the inspection requirements of the NPDES General Permit No. 2.

   The preparer of the SWPPP should check with the local jurisdiction for additional review and permitting requirements.

4. **Contractor/Builder:** Contractors and builders that are involved in implementing any of the measures identified for controlling pollution of stormwater runoff must sign on as a co-permittee with the owner. As a co-permittee, the contractor is required to comply with all of the requirements of the NPDES permit.

   In addition, most owners will contractually assign all responsibility for compliance with the NPDES permit to the contractor. Under this situation, any fines levied against the owner will normally be passed along to the contractor.

E. Transfer of Ownership and Responsibilities

On many construction projects, such as private residential subdivisions or commercial developments, it is common for a developer to sell off individual lots before work on the entire subdivision is complete. Coverage under General Permit No. 2 cannot be discontinued for individual portions of a project; the permit requires that the entire project reach final stabilization before a Notice of Discontinuation can be filed, and coverage for the entire site terminated. This creates a situation where the developer and any co-permittees are responsible for compliance with the permit for land they no longer own or have control over.

A provision within the Iowa Administrative Code [567 IAC 64.6(6)(b)] addresses this situation. This provision allows the developer and new property owner to become co-permittees under the NPDES permit. This provision requires that the new owner be notified, in writing, of the existence and location of the permit and the SWPPP and of their responsibility to comply with the permit.

This provision within the Code also allows the new owner to accept sole responsibility for compliance with the permit for the transferred property. This transfer of responsibility requires written acknowledgement by the new owner that they accept responsibility for complying with the permit for the property in question.

A copy of all property transactions, notifications of coverage, and transfer of responsibility agreements must be included with the SWPPP.

The Erosion and Sedimentation Process

Erosion and sedimentation are naturally occurring processes. However, human activities have accelerated these processes well beyond the rate desired by nature. The removal of large volumes of soil from the land, and their deposition in waterways, has destroyed ecosystems and degraded the environment.

In order to minimize and control erosion and sedimentation, it is important to understand the process and cause of each.

A. The Erosion Process

1. **Water:** Erosion from water typically occurs in the following ways:

   a. **Raindrop Splash and Sheet Erosion:** The first step in the erosion process begins as raindrops impact the soil surface. Raindrops typically fall with a velocity of 20 to 30 feet per second. The energy of these impacts is sufficient to displace soil particles as high as two feet vertically. In addition, the impact of rainfall on bare soil can compact the upper layer of soil, creating a hard crust that inhibits plant establishment and infiltration.

      Sheet erosion occurs as runoff travels over the ground, picking up and transporting the particles dislodged by raindrop impacts. The process of sheet erosion is uniform, gradual, and difficult to detect until it develops into rill erosion. If runoff is maintained as sheet flow, the velocity remains low and there is little potential that the flow will remove particles that have not been dislodged by other means (i.e. raindrop splash).

      The method used to prevent erosion from raindrop splash and sheet erosion is stabilization. Stabilizing techniques such as temporary and permanent vegetation, sodding, mulching, compost blankets, and rolled erosion control products absorb the impact of raindrops and protect the ground surface. By protecting the surface, soil particles are not dislodged and transported by sheet flow. Typically, sheet flow does not have sufficient volume or velocity to dislodge soil particles from a bare surface by itself. It is dependent on raindrop impacts to disturb the surface. Therefore, stabilizing a surface protects the ground from both raindrop and sheet erosion.

   b. **Rill Erosion:** Rill erosion occurs as runoff begins to form small concentrated channels. As rill erosion begins, erosion rates increase dramatically due to the resulting higher velocity concentrated flows. Construction sites that show signs of rill erosion need to be re-evaluated and additional erosion control techniques employed. Rilling can be repaired by tilling or disking (filling in the rills and discouraging concentrated flows) and should be repaired as soon as possible in order to prevent gullies from forming.

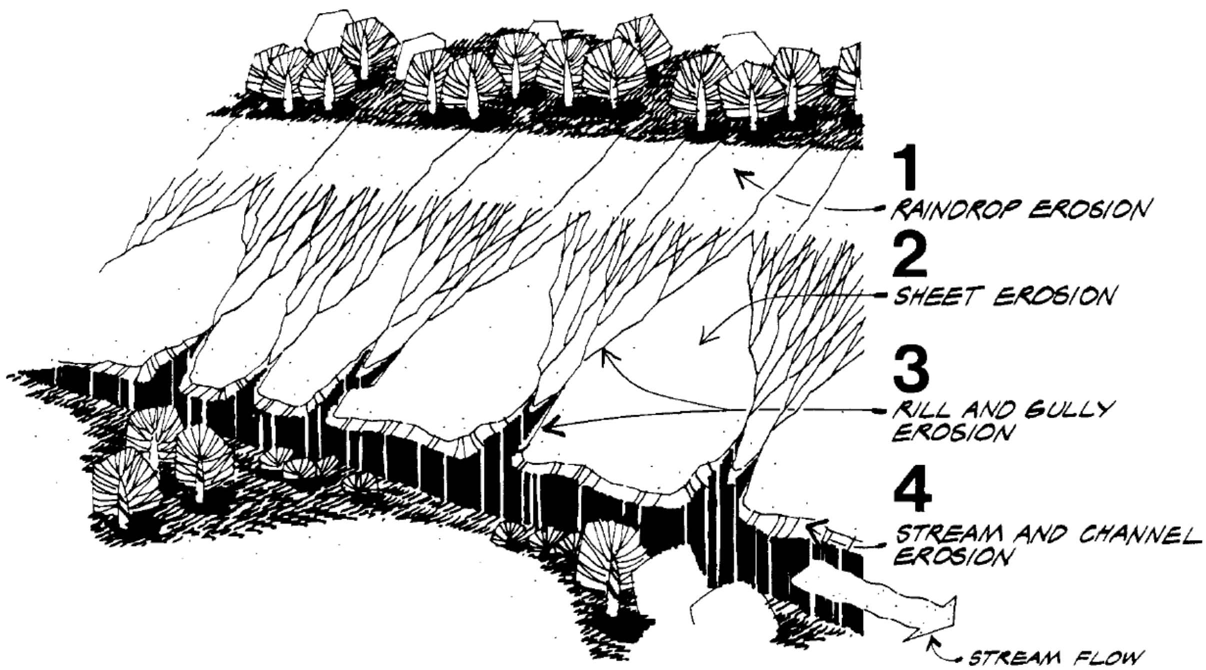
   c. **Gully Erosion:** Gully erosion results from water moving in rills, which concentrates to form larger channels. When rill erosion can no longer be repaired by merely tilling or disking, it is defined as gully erosion. Gullies must typically be repaired with earthmoving equipment. Gully erosion can be prevented by quickly repairing rill erosion and addressing the cause of the rill erosion.

d. **Stream Channel Erosion:** Stream channel erosion consists of both streambed and streambank erosion. Streambed erosion occurs as flows cut into the bottom of the channel, making it deeper. This erosion process will continue until the channel reaches a stable slope. The resulting slope is dependent on the channel materials and flow properties.

As the streambed erodes and the channel deepens, the sides of the channel become unstable and slough off, resulting in streambank erosion. Streambank erosion can also occur as soft materials are eroded from the streambank or at bends in the channel. This type of streambank erosion results in meandering waterways.

One significant cause of both streambed and streambank erosion is the increased frequency, volume, and duration of runoff events that are a result of urban development.

Figure 7C-1.01: Types of Soil Erosion by Water



Source: USDA NRCS, 2002

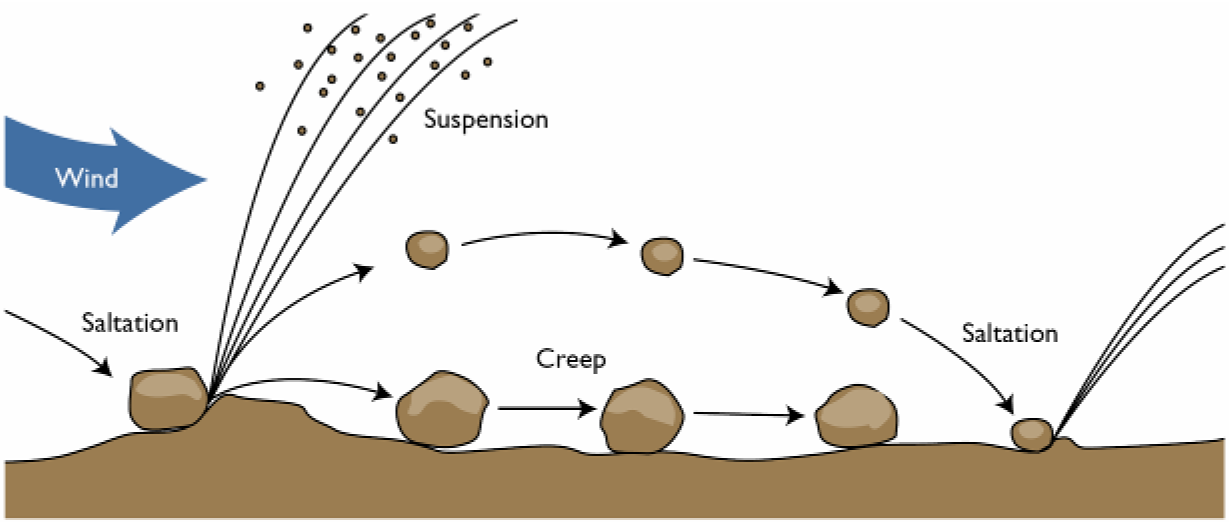
2. **Wind:** Wind can also detach soil particles. Detached soil is moved by wind in one of three ways:

   a. **Suspension:** Very fine silt and clay particles (smaller than 0.002 inches in diameter) may be picked up by the wind and carried in suspension. Suspended dust may be moved great distances, but does not drop out of the air unless rain washes it out or the velocity of the wind is dramatically reduced.

   b. **Saltation:** Fine silts up to medium sand particles (0.002 to 0.02 inches in diameter) move in the wind in a series of steps, rising into the air and falling after a short flight. This movement is called saltation. A vast majority of wind erosion is a result of the saltation process.

   c. **Creep:** Soil particles larger than medium sands (greater than 0.02 inches) cannot be lifted into the wind, but particles up to 0.04 inches (coarse sand) may be pushed along the soil surface by saltating grains or direct wind action. This action is called creep.

Figure 7C-1.02: Movement of Soil Particles by Wind Erosion



Source: McCauley & Jones, 2005

B. Factors Affecting Erosion

The extent of erosion that occurs is dependent on a number of factors including soil erodibility, climate, vegetative cover, topography, and season.

1. **Soil Erodibility:** The erodibility of a soil depends on its texture and physical properties. The characteristics that influence the potential for erosion are those related to the infiltration capacity of the soil and its ability to resist detachment. Soil properties that affect erodibility include texture, organic matter content, soil structure, and permeability.

   In general, soils with a high percentage of fine sand and silt are the most erodible. These particles are easily detached and carried away by rainfall and runoff. As the clay and organic content of a soil increases, the erodibility of the soil tends to decrease.

   Clay particles have the ability to bind together, reducing the potential for detachment by raindrop splash. However, they are also more impermeable, resulting in increased runoff. The increase in runoff increases the erosion potential (especially rill and gully), offsetting some of the benefit that the binding effect has against resisting erosion. The problem with clay particles is that once they have eroded, they are easily transported by water and are very difficult to remove.

   Soils that are high in organic matter content have a more stable texture and increased permeability. This allows the soil to resist detachment and infiltrate more precipitation. Well-draining sands and gravels are the least erodible soils. Soils with high infiltration rates such as these significantly reduce the amount of runoff, thereby reducing the potential for erosion.

   The USDA county soil surveys provide an indication of soil erodibility. This value (K) ranges from 0 to approximately 0.7. Higher values indicate a greater potential for erosion.

2. **Precipitation:** The rate of erosion is directly related to the amount and type of precipitation that occurs. High intensity storms increase particle detachment. In addition, frequent or lengthy storms can saturate the soil, reducing infiltration and increasing runoff. Increased detachment and runoff both contribute to erosion. Erosion risks are high where precipitation is frequent,

intense, or lengthy. In Iowa, the wettest months occur between May and August, when construction activities are at their peak.

3. **Ground Cover:** Ground cover can significantly reduce erosion potential. Vegetative residue, mulch, and compost, as well as the leaves and branches of vegetation, intercept precipitation and shield the ground from raindrop impacts. The roots of vegetation help hold soil particles together and prevent them from becoming detached. Ground cover slows runoff velocity, increases infiltration, and can even filter sediment out of runoff.

4. **Topography:** Areas with long and/or steep slopes increase the potential for erosion. Long slopes increase the potential for runoff to accumulate and develop into erosive concentrated flow near the bottom of the slope. On steep slopes, high-velocity flows can develop quickly and cause significant erosion.

5. **Season:** The potential for erosion varies throughout the year. In winter months when the ground surface is frozen, there is little chance of water erosion. As spring approaches, the surface soils begin to thaw, but the ground below remains frozen. This creates a high potential for erosion. An early spring rain at this time cannot infiltrate into the frozen subsoils. However, the newly thawed surface can be easily washed away, even by a light rain. Erosion potential is also high in the summer months, due to the high-intensity thunderstorms that occur during this period.

C. Sediment Transportation

Once soil particles are detached from the surface and suspended in runoff waters, they will remain there until the velocity of the water is reduced. Flowing waters create turbulence that constantly churns and mixes the flow, holding the particle in suspension. In order for the particles to be removed, the velocity of the flow must be reduced sufficiently to allow the particle to settle out by gravity. This process is discussed in further detail in Section 7D-1 - Design Criteria.

Once sediment reaches a natural waterway or stream, it is nearly impossible to remove. As discussed above, the flowing nature of the stream holds the particles in suspension until the flow velocity is reduced. For natural waterways, this reduction in flow velocity does not normally occur until the waterway empties into a water body. At this point, the sediment settles out and is deposited on the bottom of the pond or lake. Over time, this sediment accumulates, forming large deposits and can eventually fill in a water body completely. Sediment is the largest pollutant (by volume) in stormwater runoff. The resulting deposits can destroy ecosystems and are difficult and expensive to remove.

Design Criteria

A. Introduction

Erosion and sediment control should be an integral part of every construction project. Preventing sediment from leaving construction sites is a major advancement toward improving water quality. The first step in erosion and sediment control for a construction project should begin with proper design. In order to effectively design erosion and sediment control measures, a distinction must be made between erosion control and sediment control; and the role of each defined.

The primary method of protecting a site should be preventing erosion. Erosion control measures protect the ground surface and prevent soil particles from being detached by the force of raindrop impact and concentrated flows. Sediment control practices focus on the removal of suspended particles from runoff after erosion has occurred. No sediment control structure is 100% effective, and removal of fine soil particles, which are very common in Iowa, is difficult. The best way to improve the efficiency of sediment control structures is to prevent erosion in the first place.

Sediment control practices are generally more expensive and less effective than providing proper erosion control. While sediment control structures can remove significant amounts of sediment from stormwater runoff, and should be implemented as part of the overall erosion and sediment control plan, they should be considered secondary to erosion control for the reasons described above.

Figure 7D-1.01: Sediment in Street Due to Inadequate Erosion and Sediment Control During Construction

Source: USDA NRCS Photo Gallery

B. Erosion Control

The key to successful erosion and sediment control on construction sites is the prevention of erosion. The simplest way to keep sediment from leaving a site is to keep it in place. The following site management methods should be implemented on all sites to help prevent erosion from occurring:

1. **Limit Exposed Area:** Existing well-vegetated areas are usually stable and nearly erosion-proof. The simplest and cheapest way to prevent erosion on a site is to prevent the existing vegetation from being disturbed. Obviously, this cannot be done for areas that must be graded and some ground must be exposed. However, by carefully planning the construction, controlling staging and equipment storage areas, and marking construction limits, the exposed area can be minimized.

2. **Limit Exposure Time:** Leave existing vegetation in place as long as construction operations allow to reduce the amount of time that a disturbed surface is exposed. If possible, stage construction so that one area is stabilized before grading activities begin on another area. After areas are disturbed, they should be stabilized as soon as possible. The NPDES permit contains specific requirements for initiating stabilization procedures once construction activities are completed or temporarily suspended. Stabilization activities may include temporary or permanent seeding, sodding, rolled erosion control products, turf reinforcement mats, compost blankets, or mulching.

3. **Divert Runoff:** Sheet or concentrated flow over a disturbed area can cause severe erosion. For sites that receive upland runoff, diversion should be constructed to protect bare slopes until vegetation or stabilization is established. Methods of diverting runoff away from or over disturbed areas include diversion structures (berms and swales), slope drains, rock chutes, and flumes. Diverted runoff must be discharged to a stable outlet. A level spreader can be used to convert concentrated diverted flows to sheet flow before they are released onto stable ground.

4. **Limit Velocity:** As runoff travels down a bare slope, its velocity increases. Limiting slope lengths will help prevent high-velocity flows. Where it is not practical to reduce the height of a slope by grading, the slope length can effectively be broken up into several smaller slopes by installing silt fence, filter berms, filter socks, and wattles. In ditches and channels, check dams should be used.

5. **Protect Concentrated Flow Areas:** Concentrated flows will occur on most sites. As sheet flows converge and the volume increases, the flow eventually becomes concentrated and provisions must be made to prevent erosion. Grass channels can carry some concentrated flow. Rolled erosion control products and turf reinforcement mats can provide additional reinforcement when required. At discharge points, rock outlet protection or flow transition mats can be provided to dissipate energy and prevent scour at the outlet.

C. Calculating Soil Loss

Regardless of the stabilizing and vegetative practices employed, inevitably some soil erosion will occur. Over the years, a variety of different models have been developed to estimate the amount of erosion that occurs on a given site. The current model utilized by the National Resource Conservation Service (NRCS) is the second revision of the Uniform Soil Loss Equation (USLE) which is called RUSLE2 (Revised Universal Soil Loss Equation). RUSLE2 is a semi-empirical model that considers the erodibility factors discussed in the previous section. The RUSLE2 model utilizes the following equation to determine sediment delivery rate:

$$A = R \times K \times L \times S \times C \times P$$ **Equation 7D-1.01**

Where:

A = Estimated average annual soil loss in tons/acre/year
R = Rainfall-runoff erosivity factor
K = Soil erodibility factor
L = Slope length factor
S = Slope steepness factor
C = Cover management factor
P = Support practice factor

Manually calculating soil loss with the RUSLE2 model is a time-consuming process that requires extensive weather, soils, and other support information. In order to simplify the use of RUSLE2, NRCS has developed a RUSLE2 software program. The RUSLE2 program utilizes the concept described above to estimate soil loss, sediment yield, and sediment characteristics from sheet and rill erosion. This program is available for download from NRCS at: http://fargo.nserl.purdue.edu/rusle2_dataweb/RUSLE2_Index.htm.

While the RUSLE2 model was originally developed to analyze conservation practices on agricultural land, it can also be used to estimate sediment delivery rates from construction sites. This is an especially useful tool for designing erosion and sediment control systems for large sites. It is also useful for estimating sediment delivery rates to both temporary and permanent sediment basins. This information can be used to estimate the required cleanout frequency for sediment control structures, and for identifying sites that are highly susceptible to erosion, so potential problems can be addressed prior to construction.

D. Sediment Removal

1. **Sediment Control Devices:** Eroded soil particles that are suspended in flowing runoff waters will be transported offsite unless they are removed. The simplest and most efficient way to remove suspended particles from runoff is by detaining the runoff to slow the flow velocity; thereby allowing the suspended soil particles to settle out. This is most commonly accomplished with a sediment control device.

   The most important factor in designing a sediment control device is selecting the appropriate size. The ideal situation would be to collect and retain all runoff in a large retention structure, preventing any contaminated water from leaving the site. However, this is not practical in most situations. First, to retain all water onsite would require large storage areas and volumes. In addition, the retained runoff would be required to infiltrate into the ground or evaporate. These processes may not be sufficient to remove all of the runoff before the next storm occurs.

A more practical approach is to size a device to detain the runoff for a sufficient time to remove a significant portion of the suspended material, yet allow the structure to outlet excess runoff, rather than retaining it. Since the device is allowed to drain both during and after the storm event, the size can be reduced, and the danger of being flooded out by a subsequent storm event is also reduced.

2. **Designing Major Sediment Control Devices:** For a major sediment control device such as sediment basin or sediment trap to perform efficiently, it must be large enough to detain the contaminated runoff for a sufficient time to allow suspended particles to settle out, allow a sufficient flow of water through the system to prevent flooding, and be small enough that it is cost-effective to construct. In order to size an efficient basin, an understanding of the physics involved in removing suspended soil particles is required.

   a. **Settling Velocity of Suspended Particles:** Particles suspended within a fluid will settle due to the force of gravity according to Stoke's Law. In summary, Stoke's Law states that a particle suspended within a fluid will fall at a constant vertical velocity, or settling velocity. The settling velocity is reached when the force of gravity acting on the particle equals the fluid resistance acting on the particle. The settling velocity of a suspended particle (assumed to be spherical) falling through water can be expressed as:

$$V_s = \frac{\left[g \times (G_s - 1) \times d^2\right]}{(18 \times \mu)}$$

Equation 7D-1.02

Where:

V_s = Settling velocity (ft/sec)
g = Acceleration of gravity (32.2 ft/sec^2)
μ = Kinematic viscosity (ft^2/sec^2)
G_s = Specific gravity of a particle
d = Diameter of a particle (ft)

   b. **Soil Types and Properties:** The size required for a sediment control structure to be effective depends greatly on the properties of the suspended soil particles that must be removed. Soil particles settle at different rates based upon their diameter and specific gravity. Larger particles will settle out according to Stoke's Law, as described above. However, very small particles, such as colloidal clay particles and fine silts have extremely slow settling velocities.

   Capturing these small particles with a sediment control device may be impractical due to the extremely large structure size required to provide the long detention time required. Clay particles in particular, may never settle and remain suspended indefinitely due to Brownian Movement, which is a result of negatively charged particles repelling each other.

   A sediment control device is designed around a design-size particle. The device is designed to remove 100% of soil particles that are design-size or larger. The design-size particle selected should be based upon the smallest soil particles that are present on the site to be disturbed.

   Based upon the practical limitations discussed above, design-size particle selected to size the structure may normally be limited to medium silts or larger. For sites with fine silts and clay, which are smaller than the size used to design the structure, only a partial removal of these suspended fines can be expected. Because of this, additional efforts to prevent erosion should be utilized. The following table lists common settling velocities for various soil types.

Table 7D-1.01: Typical Soil Particle Settling Velocities

Particle	Diameter (ft)	Settling Velocity @ 60˚ F (ft/sec)
Fine Silt	$3.3x10^{-5}$	$2.62x10^{-4}$
Medium Silt	$6.6\ x10^{-5}$	$1.02\ x10^{-3}$
	$9.8\ x10^{-5}$	$2.26\ x10^{-3}$
Coarse Silt	$1.3\ x10^{-4}$	$4.00\ x10^{-3}$
	$1.6\ x10^{-4}$	$6.27\ x10^{-3}$
	$2.0\ x10^{-4}$	$9.02\ x10^{-3}$
Very Fine Sand	$2.3\ x10^{-4}$	0.012
	$2.6\ x10^{-4}$	0.016
	$3.0\ x10^{-4}$	0.020
	$3.3\ x10^{-4}$	0.025
	$3.6\ x10^{-4}$	0.030
	$3.9\ x10^{-4}$	0.036
Fine Sand	$4.3\ x10^{-4}$	0.042
	$4.6\ x10^{-4}$	0.049
	$4.9\ x10^{-4}$	0.056
	$5.2\ x10^{-4}$	0.064
	$5.6\ x10^{-4}$	0.073
	$5.9\ x10^{-4}$	0.081
	$6.2\ x10^{-4}$	0.091
	$6.6\ x10^{-4}$	0.100

Source: Adapted from Fifield, 2001

c. Major Sediment Control Device Sizing: Soil particles are held in suspension by the turbulence associated with high flow velocities. In order to force suspended particles to settle out at a desired location, it is necessary to reduce the velocity of the runoff. Sediment control devices achieve this by increasing the cross-sectional area of the flow.

Based upon the settling velocity of the design-size soil particle and the outflow rate from the structure, the required surface area of the device can be calculated with the following equation:

$$SA = (1.2) \times \frac{(100 \times Q_{out})}{V_s}$$ **Equation 7D-1.03**

Where:

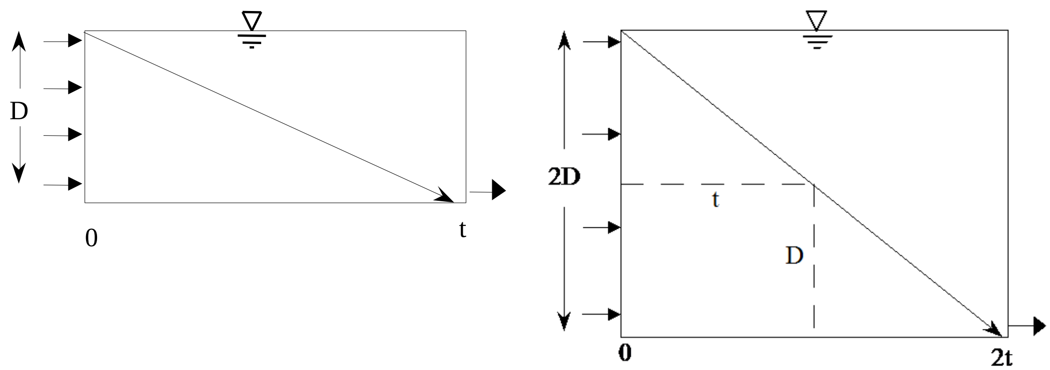
SA = Surface area of sediment control device (ft^2)
Q_{out} = Discharge (ft^3/s)
V_s = Settling velocity of design particle (ft/sec)

The discharge rate from the device is the peak release rate for a 2 year, 24 hour storm. This rate is dependent on the drawdown time and outlet configuration. Refer to the information in Sections 7E-12 and 7E-13 on sediment basins and sediment traps for determining the configuration of the release structure and the drawdown time.

The equation above includes a safety factor of 1.2 as recommended by the EPA. This factor increases the minimum surface area by 20% to compensate for disturbances in uniform flow caused by wind, rain, wave action, and turbulence at the outlet structure.

The above equation for determining the size of a sediment control device is independent of depth. The reason the size is independent of depth can best be explained by the figures below. Particles that reach the bottom of the device prior to the overflow point are considered captured and should remain within the device. In the figures below, as the depth of the device is doubled (assuming the surface area remains constant), the suspended particles must travel twice as far to reach the bottom of the device. However, since the volume of storage is twice that of the first figure, the flow velocity through the device is only half that of the first and the particles have twice as much time to settle.

Figure 7D-1.02: Example of the Relationship Between Settling Time and Structure Volume



While theory may suggest that any depth is sufficient, field experience has shown that a minimum depth of 2 feet is required to account for actual conditions. This depth helps eliminate dead zones and short-circuiting, where inflows simply pass straight through the device without spreading out, reducing their velocity, and dropping the suspended sediment. This minimum depth also provides sufficient volume for a deposition zone, reducing cleanout frequency. Additional depth should be provided near the upstream end of the device. This provides an area for heavier particles to be trapped while maintaining the deposition area for smaller sized particles. Permanent sediment control devices should have a minimum depth greater than 2 feet in order to reduce the cleanout frequency.

Based upon field experience and practical limitations, a minimum depth of 2 feet can be applied to Equation 7D-1.04 to determine the minimum storage volume required.

$$SV = (2.4) \times \left(\frac{Q_{out}}{V_s} \right)$$ **Equation 7D-1.04**

Where:

SV = Storage volume (ft^3) (volume of dry storage)
Q_{out} = Discharge (ft^3/s) (peak discharge for a 2-yr, 24-hr storm)
V_s = Settling velocity of design particle (ft/sec)

d. **Device Shape:** The shape of the sediment control device is also important. The longer the flow path is for a particle through a device, the better the chances are that it will be captured. In addition, longer devices provide more area for deposition away from the turbulence of the inlet and outlet. A length to width ratio of 10:1 is recommended. The minimum length to width ratio should be 2:1.

3. **Major Sediment Control Device Requirements:** While the discussion above provides a background on the concept and theory behind designing a sediment control device, the EPA has established its own minimum standards that must be met. The following summary of recommended design standards meet or exceed the EPA's regulations and should be followed for sediment basin and sediment trap design.

   Sediment basins are required for disturbed areas greater than 10 acres, which drain to a common location. Sediment basins must be sized to provide a minimum storage volume of 3,600 cf of storage per acre drained. The storage requirement does not apply to flows from undisturbed areas or stabilized areas that have been diverted around the sediment basin.

   For disturbed areas greater than 10 acres where a sediment basin designed according to the guidelines above is not feasible, smaller sediment basins or sediment traps should be used in conjunction with other erosion and sediment control practices as required to provide equivalent protection.

   The storage volume provided for a sediment basin or sediment trap should be split equally between wet and dry storage. Wet storage is that volume which is below the embankment area and has a permanent pool. Dry storage is the volume that is detained by the release structure, but eventually released.

   The following additional criteria should be provided for sediment control structures:

   a. A minimum length to width ratio of 2:1 (10:1 desirable) should be provided.

   b. A minimum depth of 2 feet from bottom of basin to overflow elevation (deeper structure recommended to reduce cleanout frequency).

   c. Side slopes 2:1 or flatter.

4. **Minor Sediment Control Devices:** For areas where a major sediment control device such as a sediment basin or sediment trap, are not required or cannot be utilized, minor sediment control devices and measures should be provided. These measures provide the last line of defense against releasing sediment-laden stormwater runoff from a construction site. Minor sediment control devices that remove sediment from flow include vegetative filter strips, filter berms, filter socks, silt fence, and inlet protection.

   Other measures that control sediment include stabilized construction entrances, which help prevent track out into streets; flocculents, which help remove suspended particles from standing water; and flotation silt curtains, which are used for construction within or near a water body.

Design Information for Erosion and Sediment Control Measures

A. General

The following sections provide design information for a variety of erosion and sediment control measures. Each section describes the measure, how to properly design and implement it, and the benefits that it provides. Each measure's benefits are shown on the first page and a rating (high, medium, or low) is given for each; a summary of the individual measures and their benefits is shown in Table 7E-1.01. The benefits have been divided into five categories that directly affect erosion or sediment transportation. The following are descriptions of each of the benefits shown in Table 7E-1.01.

B. Flow Control

Flow control refers to the ability of a practice to reduce flow velocity (either sheet or concentrated flow). Reducing flow velocity helps reduce erosion and transportation of sediment. Controlling velocity is important on long or steep slopes. High-velocity flows can quickly cause severe erosion.

C. Erosion Control

Erosion control is the measure's ability to stabilize the surface and prevent soil particles from becoming displaced. Erosion control should be utilized on all disturbed surfaces. Preventing erosion from taking place is the simplest and most cost-effective method of keeping sediment from leaving a site.

D. Sediment Control

Sediment control is the ability of a practice to remove suspended soil particles from runoff after erosion has taken place. Sediment control measures are the last line of protection against releasing sediment laden runoff into water bodies or waterways.

E. Runoff Reduction

Runoff reduction is the ability to reduce the volume of runoff from a site. Reducing the volume from an area also reduces the potential for both erosion and sediment transportation. These methods utilize absorption or increase the potential for infiltration of stormwater into the soil.

F. Flow Diversion

Flow diversion consists of routing upland runoff around disturbed areas. By reducing the amount of runoff over a disturbed area, the potential for erosion and sediment transportation are also reduced.

G. Selecting Control Measures

The following table may be used to select a system of both erosion control and sediment control measures. No single measure should be relied upon as the sole method of erosion control and sediment control.

Table 7E-1.01: Summary of Erosion and Sediment Control Measures and Benefits

Section	Control Measure	Benefits				
		Flow Control (Velocity)	**Erosion Control (Stabilization)**	**Sediment Control (Removal)**	**Runoff Reduction (Volume)**	**Flow Diversion**
Vegetative and Soil Stabilization Erosion Control Measures						
7E-2	Compost Blanket	M	M	L	M	
7E-5	Temporary Rolled Erosion Control Products	L	H			
7E-16	Dust Control		M			
7E-17	Erosion Control Mulching	L	M	L	L	
7E-18	Turf Reinforcement Mats	L	H			
7E-19	Surface Roughening	L	L		L	
7E-22	Temporary Erosion Control Seeding	M	H	M	L	
7E-23	Grass Channel	L	H	L	L	
7E-24	Permanent Seeding	M	H	M	M	
7E-25	Sodding	M	H	M	M	
7E-26	Vegetative Filter Strip	L	L	M	L	
Structural Erosion Control Measures						
7E-7	Check Dams	H		L		
7E-8	Temporary Earth Diversion Structures					H
7E-9	Level Spreaders	H				M
7E-10	Rip Rap	H	H			
7E-11	Temporary Pipe Slope Drains					H
7E-21	Flow Transition Mats	L	H			
7E-27	Rock Chutes and Flumes	M	H			
Sediment Control Measures						
7E-3	Filter Berms	L		L		L
7E-4	Filter Socks	L		L		L
7E-6	Wattles	L		L		
7E-12	Sediment Basin	H		H	L	
7E-13	Sediment Traps	H		H	L	
7E-14	Silt Fences	L		M		M
7E-15	Stabilized Construction Entrance			L		
7E-20	Inlet Protection			L		
7E-28	Flocculents			H		
7E-29	Flotation Silt Curtain			M		

Compost Blanket

Source: Soil-Tek

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: A 1 to 4 inch surface application of compost/mulch or a blend of both to protect areas with erosive potential.

Typical Uses: Used to protect bare soil surfaces from raindrop impact, prevent/reduce sediment loss, reduce surface water runoff, and promote seed growth for establishment of ground cover.

Advantages:

- Immediately protects 100% of the ground surface upon application.
- Conforms to any terrain.
- Reduces rill erosion and the volume of stormwater runoff.
- Allows for natural infiltration and percolation of water into underlying soil.
- Can be combined with seed and placed in a one-step process with pneumatic blower truck.
- Can be used when projects have begun too late in the growing season to establish erosion control vegetation.
- Has high water retention properties, thereby reducing watering requirements during dry weather periods.
- Can be used in areas with poor quality soils (low organic matter) that do not support vigorous growth of vegetation.

Limitations:

- Not suitable for areas of concentrated water flow unless used in conjunction with other control measures that slow velocities.
- Erosion control benefit of compost is eliminated if material is incorporated (tilled) into soil.
- Susceptible to wind erosion.

Longevity: One year; longer if seeding combined

SUDAS Specifications: Refer to Section 9040, 2.01 and 3.05

A. Description/Uses

A compost blanket consists of a layer of compost/mulch or a blend of both placed on denuded areas to help prevent initiation of runoff and erosion. Apply compost blanket to a depth of 2 to 4 inches, depending on slope steepness. When a pneumatic blower truck is utilized, a compost blanket can be installed and seeded simultaneously for permanent vegetation establishment.

A compost blanket is both an erosion control and stormwater quality practice. Compost blankets stabilize the soil, prevent splash, sheet, and rill erosion, and remove suspended soil particles and contaminants from water moving offsite and into adjacent waterways or stormwater conveyance systems.

B. Design Considerations

Compost quality and screen size is important. Coarse compost tends to provide more protection than fine material. The coarser compost includes particles which are large enough to prevent them from being washed away or displaced by the rainfall. Fine compost particles can be dislodged and washed away, eliminating any potential protection.

For full erosion control benefits, compost should not be incorporated (tilled) into underlying soil.

In order to prevent water from sheeting between the compost blanket material and soil surface on a slope, a minimum 3-foot wide band of blanket material should be placed behind the top of the slope. Alternatively, a compost berm or filter sock may be placed at the top of the slope.

Compost can be seeded for temporary and/or permanent vegetation during or immediately after installation. For vegetated compost blankets (pneumatic seeding), a maximum blanket depth of 2 inches is recommended. Deeper compost depths can prevent the roots of the vegetation from growing down into the underlying soil. This process is important in developing long term slope stability.

With turf or sod, compost can replace topsoil requirements; 1 inch of compost is equivalent to 3 inches of topsoil.

C. Application

Application rates should be between 2 to 3 inches in depth (270 to 405 cubic yards per acre) with greater depths for steeper slopes.

Table 7E-2.01: Compost Blanket Thickness on Slopes

Slope	Compost Thickness (inches)
2:1 (see comments below)	4
3:1	3
4:1	2

Compost blankets may be used to stabilize steep slopes (up to 2:1) if additional measures are provided. For these severe applications, the slope length should be reduced through the installation of silt fence, filter berms or filter socks. A maximum spacing of 25 feet between slope reduction practices should be provided. In addition, lightweight mulch control netting should be placed under the compost and anchored into place. These additional practices help stabilize the blanket and prevent the material from sliding down steep grades.

D. Maintenance

The disturbed ground under the blanket should be checked in spots for failure. Common failures, due to concentration of water flows or the improper type of compost used, will result in splash, sheet or rill erosion of the underlying soil. Damage should be repaired immediately to prevent further erosion.

E. Time of Year

Compost blankets are effective on a year-round basis. Unlike other erosion control measures, installation is possible when the ground is wet or frozen, especially if a pneumatic blower truck is utilized for placement.

F. Regional Location

The availability of compost blankets are affected by regional location, due to adequate supplies of compost and composting facilities.

Filter Berms

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control			
Sediment Control	■		
Runoff Reduction			
Flow Diversion	■		

Description: A filter berm is a windrow-shaped (triangular) structure with a specified ‘filter material’ that normally is a blend of composted materials or other organic products, used to slow flow velocity, capture and degrade chemical pollutants, and trap sediment.

Typical Uses: Perimeter control, slope length reduction, flow diversion for small drainage areas, environmentally sensitive areas such as wetlands and waterways, at the edge of gravel parking lots, and general areas under construction.

Advantages:
- Less likely to obstruct wildlife movement and migration then other practices.
- Does not always need to be removed, thereby eliminating removal and disposal costs.
- Can be installed year-round in difficult soil conditions such as frozen or wet ground, on hard compacted soils, near pavements, and in wooded areas.

Limitations:
- Not suitable for areas of concentrated water flow or below culvert outlet aprons.
- Availability of suitable filter materials may be limited.
- Equipment operators may drive over berms, damaging the product.

Longevity: Six months

SUDAS Specifications: Refer to Section 9040, 2.03 and 3.06

A. Description/Uses

A filter berm typically consists of a three-dimensional matrix of biologically active stable composted organic material with various sized particles formed in a continuous windrow fashion (triangular) that slows and filters water to capture sediment and degrade pollutants. Its natural permeability allows water to seep through it while capturing sediment in its pore space and behind its mass, slowing water velocity and absorbing water pollutants, such as hydrocarbons, nutrients, and bacteria.

B. Design Considerations

1. **Materials:** The key to achieving the proper balance between sediment removal and flow-through rate is using a filter material with the proper particle size. Filter material with a high percentage of fine particles will clog and create a barrier to flow. This will cause water to pond and the pressure may cause the installation to fail. Alternatively, filter material with particles that are too large will allow runoff to pass through the barrier with little or no resistance, eliminating the velocity reduction and sediment trapping benefits of the barrier. Refer to SUDAS Specifications Section 9040 for proper filter material size.

2. **General Guidelines:** Filter berms should maintain a 2:1 base to height ratio to ensure berm stability, with a minimum berm size of 1 foot high by 2 feet wide.

3. **Slope Control:** When installed on slopes, filter berms should be installed along the contour of the slope, perpendicular to sheet flow. The beginning and end of the installation should point slightly up the slope, creating a “J” shape at each end to contain runoff and prevent it from flowing around the ends of the berm. Allowable slope length for compost filter berms is dependent upon the grade of the slope as shown in Table 7E-3.01. For slopes that receive runoff from above, a filter berm should be placed at the top of the slope to control the velocity of the flow running onto the slope, and to spread the runoff out into sheet flow. On steep or excessively long slopes a number of filter berms may be placed at regular intervals down the slope.

4. **Sediment Control:** Filter berms remove sediment both by filtering, and by ponding water behind them. When used for sediment control, filter berms should be located to maximize the storage volume created behind the berm.

   A common location to place filter berms for sediment control is at the toe of a slope. When used for this application, the berm should be located as far away from the toe of the slope as practical to ensure that a large storage volume is available for runoff and sediment.

C. Application

Compost filter berms should be spaced according to Table 7E-3.01.

Table 7E-3.01: Maximum Filter Berm Spacing

Slope	Maximum Spacing (feet)	Compost Berm Size Height x Width (feet)
0% to 2%	125	1 x 2
2% to 5%	75	1 x 2
5% to 10%	50	1 x 2

As mentioned previously, the material properties of the filter material are a significant factor in the performance of the berm. The wood chip product typically used as a filter material may not be readily available in all areas. This may limit the utilization of filter berms as an economical sediment control option in some areas.

D. Maintenance

Accumulated sediment should be removed, or a new berm installed, when it reaches approximately one-half of the berm height. If concentrated flows are bypassing or breaching the berm, it must be expanded, enlarged, or augmented with additional erosion and sediment control practices. Additional filter material should be added as required to maintain the dimensions of the berm. Any damage should be repaired immediately.

Filter Socks

Source: Soil-Tek

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control			
Sediment Control	■		
Runoff Reduction			
Flow Diversion	■		

Description: A filter sock is a tubular mesh sock filled with a specified 'filter material' that normally is a blend of composted materials or similar organic products, used to slow flow velocity, capture and degrade chemical pollutants, and trap sediment. They are most effective when designed to provide comprehensive water and sediment control throughout a construction site and if used in conjunction with other erosion control practices.

Typical Uses: Perimeter control, inlet protection, slope length reduction, flow diversion for small drainage areas, environmentally sensitive areas such as wetlands and waterways, at the edge of gravel parking lots, and general areas under construction.

Advantages:
- Less likely to obstruct wildlife movement and migration than other practices.
- Does not always need to be removed, thereby eliminating removal and disposal costs.
- Can be installed year-round in difficult soil conditions such as frozen or wet ground, on hard compacted soils, near pavements, and in wooded areas, as long as stakes can be driven.
- Relatively low cost.

Limitations:
- Not suitable for areas of concentrated water flow, low points of concentrated runoff or below culvert outlet aprons.
- Availability of suitable sock filtering materials and equipment may be limited.
- Equipment operators may drive over socks, damaging the product.
- Often used improperly as the sole method of sediment control.
- Uneven ground may cause leakage under socks.

Longevity: Until sediment accumulates to one-half the height of the sock

SUDAS Specifications: Refer to Section 9040, 2.04 and 3.07

A. Description/Uses

A filter sock typically consists of a three-dimensional matrix of certified, composted organic material and/or other organic matter to create a filter medium. These various sized particles enclosed in a tubular mesh material slows and filters water to capture sediment and degrade pollutants. Its natural permeability allows water to seep through it while capturing sediment in its pore space and behind its mass, slowing water velocity, and absorbing water pollutants, such as hydrocarbons, nutrients, and bacteria.

The filter socks are typically constructed by filling a mesh tube with organic filter material, although other materials, such as crushed rock or gravel may be used. The sock is filled by blowing the material into the tube with a special pneumatic blower truck or similar device. Hand filling is not an acceptable means to fill the tube as the material is not compacted in the sock.

B. Design Considerations

1. **Materials:** Several types of materials can be utilized for filter material in the sock. The key to achieving the proper balance between sediment removal and flow-through rate is using a material with the proper particle size. Filter material with a high percentage of fine particles will clog and create a barrier to flow. This will cause water to pond and the pressure could cause the installation to fail. Alternatively, filter materials with particles that are too large will allow flows to pass through the barrier with little or no resistance, eliminating the velocity reduction and sediment trapping benefits of the barrier. Refer to SUDAS Specifications Section 9040 for proper filter material size.

   Filter material normally consists of wood chips or mulch that is screened to remove some of the fines and produce the desired gradation. Crushed stone or gravel is an ideal material to use when the sock will be used on a paved street for inlet protection, or other areas where the sock cannot be staked to hold it in place. The additional weight of the stone helps prevent the sock from moving. Socks can be filled with a fine compost material for applications where the sock is to be vegetated and remain as a permanent feature. This material should only be used in areas where ponding water is acceptable since it has a low flow-through rate, and will quickly plug with sediment.

   The mesh sock used to contain the compost is designed to photo-degrade over time (approximately 18 months).

2. **General Guidelines:** When installed on slopes, filter socks should be installed along the contour of the slope, perpendicular to flow, and staked at 10 foot intervals. The beginning and end of the installation should point slightly up the slope, creating a “J” shape at each end to contain runoff and prevent it from flowing around the ends of the sock. Individual section of filter sock should be limited to 200 foot lengths. This limits the impact if a failure occurs, and prevents large volumes of water from accumulating and flowing to one end of the installation, which may cause undermining or damage to the sock.

   Once installed, additional loose compost (or soil from the site) should be placed at the bottom of the sock, along the leading edge, filling the seam between the ground surface and the sock.

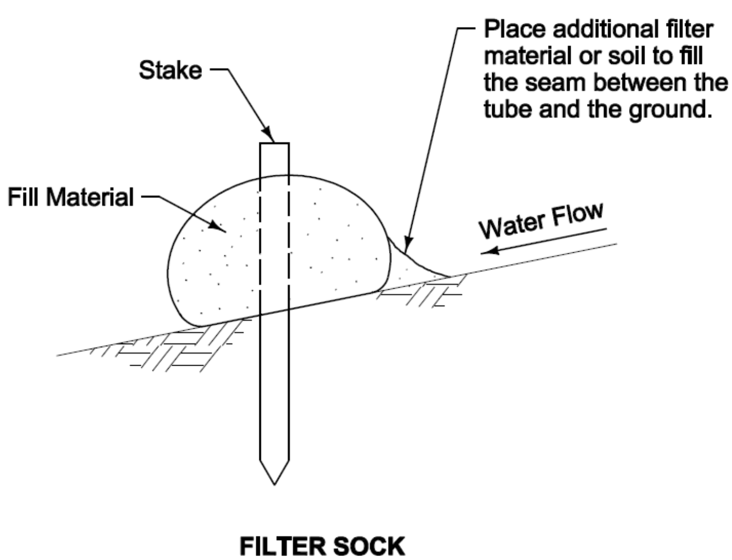
3. **Slope Control:** Filter socks can be installed at regular intervals on long slopes to reduce the effective slope length, and limit the velocity of runoff flowing down the slope. The design layout of filter socks will help prevent concentrated flows from developing, which can cause rill and gully erosion. As a secondary benefit, filter socks installed on slopes can remove suspended sediment from runoff that results from any erosion that has occurred. Allowable slope length for filter socks is dependent upon the size of the sock and the grade of the slope as shown in Table 7E-4.01. For slopes that receive runoff from above, a sock should be placed at the top of the slope to control the velocity of the flow running onto the slope, and to spread the runoff out into sheet flow. On steep or excessively long slopes a number of socks may be placed at regular intervals down the slope.

4. **Sediment Control:** Filter socks remove sediment both by filtering, and by ponding water behind them. When used for sediment control, filter socks should be located to maximize the storage volume created behind the sock.

   A common location to place filter socks for sediment control is at the toe of a slope. When used for this application, the sock should be located as far away from the toe of the slope as practical to ensure that a large storage volume is available for runoff and sediment.

5. **Inlet Protection:** Filter socks may also be used to provide inlet protection. The drainage area to a filter sock around an intake should not exceed 1/4 acre for every 100 feet of sock unless used in conjunction with other erosion and sediment control practices. Filter socks used for inlet protection should be staked at regular intervals, not exceeding 6 to 8 feet, to prevent movement of the sock. For protection of curb inlets in pavement, the length of sock recommended above is not practical. Using short sections of filter socks, such as those for curb intakes in pavement, should be done with caution. Because the length of filter sock is short, it is only able to filter a small volume of runoff. This increases the chances that significant ponding will occur, possibly dislodging the sock, or that the flows will simply bypass or overtop the sock, eliminating any treatment potential. For additional information on inlet protection, refer to Section 7E-20.

Figure 7E-4.01: Typical Filter Sock Installation
(From SUDAS Specifications Figure 9040.2)



C. Application

Filter socks, placed on slopes, should be spaced according to Table 7E-4.01.

Table 7E-4.01: Maximum Filter Sock Spacing

Slope	Sock Diameter			
	8”	***12”***	***18”***	***24”***
2%	85’	100’	100’	100’
5%	50’	75’	100’	100’
10%	40’	50’	85’	100’
5:1	35’	40’	55’	60’
4:1	30’	40’	50’	50’
3:1	30’	35’	40’	40’

As mentioned previously, the material properties of the filter are a significant factor in the performance of the sock. The wood chip product typically used as a filter material may not be readily available in all areas. This may limit the utilization of filter socks as an economical sediment control option in some areas.

D. Maintenance

Accumulated sediment should be removed, or a new sock installed, when it reaches approximately one-half of the sock diameter. If sheet flows are bypassing or breaching the sock during design storm events, it must be repaired immediately and better secured, expanded, enlarged, or augmented with additional erosion and sediment control practices.

Temporary Rolled Erosion Control Products (RECP)

Source: North American Green - SC150

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Temporary RECPs consist of prefabricated blankets or netting which are formed from both natural and synthetic materials.

Typical Uses: Temporary RECPs are used as a temporary surface stabilizing measure and to aid in the establishment of vegetation. RECPs are typically used on steep slopes and in vegetated channels.

Advantages:

- Numerous manufacturers, each with a number of different products, allow for the selection of a product that meets the individual characteristics of each site.
- Stabilizes disturbed slope and protects surface from erosive forces of raindrop impact.
- Promotes growth of vegetation.
- Most products degrade over time, eliminating potential maintenance issue.

Limitations:

- With numerous products available, appropriate product selection can be difficult.
- Various products and manufacturers have different design and construction standards. Designer must rely on manufacturer's data.
- RECPs are temporary and do not provide permanent stabilization.
- Permanent stabilization and protection is dependent on the establishment of vegetation.

Longevity: Varies based upon product specified (3 months to 36 months)

SUDAS Specifications: Refer to Section 9040, 2.05 and 3.08

A. Description/Uses

Temporary rolled erosion control products (RECP) consist of netting or blanket materials that are used to stabilize disturbed surfaces and promote the establishment of vegetation. RECPs may also be used to stabilize the surface of channels until vegetation can be established, for low to moderate flow conditions.

They are manufactured from a wide variety of different materials including coconut fiber (coir), jute, nylon, polypropylene, PVC, straw, hay, or wood excelsior. These materials may be used individually, or in combination to form nets or blankets.

The products function by protecting the ground surface from the impact of raindrops and stabilize the surface until vegetation can be established. RECPs also promote the growth of vegetation by helping to keep seed in place, and by maintaining a consistent temperature and moisture content in the soil.

RECPs are not intended to provide long-term or permanent stabilization of slopes or channels. Their role is to protect the surface until the vegetation can establish itself and become the permanent stabilizing feature. In fact, most RECPs are either biodegradable or photodegradable and will decompose over a period of time.

B. Design Considerations

RECPs are produced by a number of manufacturers, and are available in a wide variety of different configurations. Competing products from different manufacturers can have completely different material compositions and construction, but be intended to serve the same purpose. Given the wide variety of RECPs available, product selection and specification can be difficult. Fortunately, the Erosion Control Technology Council (ECTC) has developed a uniform product selection guide for RECPs. The ECTC is an organization representing suppliers and manufacturers of rolled erosion control products. A list of member organizations is available on their website (www.ectc.org).

Table 7E-5.01 follows the guidelines of the ECTC and classifies products based upon longevity and product description. RECP longevity is divided into 4 categories ranging from 3 months to 3 years. RECPs are further classified by their general material properties and construction. These classifications include: mulch control nets, open weave textiles, and erosion control blankets.

Mulch control nets (MCN) are used in conjunction with loose mulches. The MCN is applied over the loose mulch to stabilize and hold it in place. MCNs are used as an intermediate application where loose mulch may not be stable, but an open weave textile or erosion control blanket is not necessary.

Open weave textiles (OWT) consist of natural or synthetic yarns that are woven into a 2-D matrix. OWT are similar to mulch control nets, but have higher strength and a more tightly woven construction, allowing them to provide erosion protection with or without the use of an underlying loose mulch layer.

While available, the use of mulch control nets and open weave textiles as rolled erosion control products is fairly uncommon. Erosion control blankets (ECB) are the most commonly used RECP. ECB are constructed of natural and synthetic materials that are glued, woven, or structurally bound with a netting or mesh. The most common of these products are made from straw, wood excelsior or coconut fiber attached to/between netting. Wide varieties of erosion control blankets are available.

ECTC also established recommendations on the appropriate use/performance for each product classification. RECP selection and design should follow the product classification and recommendation shown in Table 7E-5.01.

For slope applications, the designer should select a product from Table 7E-5.01 that has the desired longevity and is rated for the proposed slopes.

For channel applications, the channel lining should be analyzed for the 10 year storm in the permanent vegetated state (ignoring the RECP) as described in Section 7E-23 (Grass Channel). The RECP should also be analyzed for shear stress. This analysis should be for the unvegetated state, representing the situation immediately after installation. Since it is considered a temporary measure, stabilizing the channel only until vegetation is established, the RECP does not need to be analyzed for a 10 year event as the vegetation does. Analyses of the RECP's shear strength for a 2 year event is adequate.

Proper installation of RECPs is critical. Prior to placing a RECP, the ground should be prepared and the area should be seeded and fertilized. It is imperative that seeding occur prior to placement of the RECP to ensure proper contact between seed and soil. Some manufacturers can embed the specified seed mixture into the product during the manufacturing process (if this process is used, follow the manufacturer's recommended installation specifications). After seeding, the appropriate RECP may be placed and anchored with stakes or staples. The manufacturer will provide specifications for the pattern and spacing of anchor stakes or staples, overlap between rolls (typically 6 inches), and any additional product requirements. It is important that the stakes or staples be properly installed to prevent "tenting" of the product as the vegetation begins to grow and push up on the matting. This can create an unsightly situation and the product can become entangled in mowing equipment.

At the tops of slopes and at the entrance to a channel, the leading edge of the RECP should be trenched into the ground, approximately 6 inches, anchored in place with stakes or staples, and backfilled. This prevents runoff from lifting the leading edge, and flowing between the ground and the RECP. Subsequent segments of RECPs should have their upstream edges trenched in, and the downstream edge should slightly overlap the next section to prevent water from flowing under the product.

Table 7E-5.01: Typical Rolled Erosion Control Product Properties and Uses

Type	Product Description	Material Composition	Slope Applications	Channel Applications
			Max. Grade	Permissible Shear Stress [1,2]
ULTRA SHORT-TERM - Typical 3 Month Functional Longevity				
1.A	Mulch Control Nets	A photodegradable synthetic mesh or woven biodegradable natural fiber netting	5:1 (H:V)	0.25 lbs/ft^2
1.B	Netless Rolled Erosion Control Blankets	Natural and/or polymer fibers mechanically interlocked and/or chemically adhered together to form a RECP	4:1 (H:V)	0.5 lbs/ft^2
1.C	Single-net Erosion Control Blankets and Open Weave Textiles	Processed degradable natural and/or polymer fibers mechanically bound together by a single rapidly degrading, synthetic or natural fiber netting or an open weave textile of processed rapidly degrading natural or polymer yarns or twines woven into a continuous matrix.	3:1 (H:V)	1.5 lbs/ft^2
1.D	Double-net Erosion Control Blankets	Processed degradable natural and/or polymer fibers mechanically bound together between two rapidly degrading, synthetic or natural fiber nettings.	2:1 (H:V)	1.75 lbs/ft^2
SHORT-TERM - Typical 12 Month Functional Longevity				
2.A	Mulch Control Nets	A photodegradable synthetic mesh or woven biodegradable natural fiber netting	5:1 (H:V)	0.25 lbs/ft^2
2.B	Netless Rolled Erosion Control Blankets	Natural and/or polymer fibers mechanically interlocked and/or chemically adhered together to form a RECP	4:1 (H:V)	0.5 lbs/ft^2
2.C	Single-net Erosion Control Blankets and Open Weave Textiles	An erosion control blanket composed of processed degradable natural or polymer fibers mechanically bound together by a single degradable synthetic or natural fiber netting to form a continuous matrix or an open weave textile composed of processed degradable natural or polymer yarns or twines woven into a continuous matrix.	3:1 (H:V)	1.5 lbs/ft^2
2.D	Double-net Erosion Control Blankets	Processed degradable natural and/or polymer fibers mechanically bound together between two degradable synthetic or natural fiber nettings.	2:1 (H:V)	1.75 lbs/ft^2
EXTENDED-TERM - Typical 24 Month Functional Longevity				
3.A	Mulch Control Nets	A slow degrading synthetic mesh or woven natural fiber netting	5:1 (H:V)	0.25 lbs/ft^2
3.B	Erosion Control Blankets and Open Weave Textiles	An erosion control blanket composed of processed slow degrading natural or polymer fibers mechanically bound together between two slow degrading synthetic or natural fiber nettings to form a continuous matrix or an open weave textile composed of processed slow degrading natural or polymer yarns or twines woven into a continuous matrix.	1.5:1 (H:V)	2.0 lbs/ft^2
LONG-TERM - Typical 36 Month Functional Longevity				
4	Erosion Control Blankets and Open Weave Textiles	An erosion control blanket composed of processed slow degrading natural or polymer fibers mechanically bound together between two slow degrading synthetic or natural fiber nettings to form a continuous matrix or an open weave textile composed of processed slow degrading natural or polymer yarns or twines woven into a continuous matrix.	1:1 (H:V)	2.25 lbs/ft^2

[1] Refer to Section 7E-18 - Turf Reinforcement Mats for additional information on determining shear stress in a channel

[2] Minimum shear stress RECP (unvegetated) can sustain without physical damage or excess erosion (0.5 inch soil loss during 30 minute flow event)

Source: Lancaster and Austin, 2004

C. Application

Rolled erosion control products should be used on bare ground that is highly susceptible to erosion, such as slope and channels, and in locations where establishing vegetation may otherwise be difficult.

There are a wide variety of RECPs available. Table 7E-5.01 shows the recommended applications for slopes and channels for each type of product. A manufacturer or supplier can provide further assistance in selecting an appropriate RECP. For channel applications, products that contain straw are not recommended due to the likelihood that the concentrated flow will dislodge the straw from the binding material, creating the potential for clogging problems downstream.

D. Maintenance

Once installed, there is little maintenance that needs to be done to RECPs. If the RECPs are vegetated, the vegetation should be watered as needed (refer to Section 7E-24). Until the vegetation is fully established, the surface should be inspected for signs of rill or gully erosion below the matting. Any signs of erosion, tearing of the product, or areas where the product is no longer anchored firmly to the ground should be repaired.

E. Time of Year

Seeding and placement of RECPs should be completed well within the annual seeding window. While RECPs provide some stabilization of the channel or slope surface until the vegetation is established, the vegetation ultimately provides stabilization of the surface. The vegetation needs time to establish so it can resist flows from winter snowmelt and spring rains.

F. Design Example

Due to difficulty establishing vegetation, and concerns with channel erosion, assume that a RECP is proposed for the design example from Section 7E-23 (Grass Channel).

Find the shear stress in the bare channel after the RECP has been installed. Determine if the RECP is sufficient to temporarily stabilize the channel, until the vegetation can become established.

The manufacturer states that the RECP can withstand a shear stress (without vegetation) of 2.0 lbs/ft^2. In addition, the manufacturer states that for depths between 0.5 feet and 2 feet, the Manning coefficient for the RECP varies from 0.05 to 0.018 respectively. The coefficient used for the analysis should be interpolated based upon the depth.

Assume a flow depth of 1.5 feet. Interpolating, the Manning coefficient is 0.029.

Trial 1 - Assume a depth of 1.5 feet. Interpolating, the Manning coefficient is 0.029.

Area, A=13.5; Wetted Perimeter, P=15.4; Hydraulic Radius, R=0.88
From Manning's Equation, Q=50 cfs. This is too high. Try a lower depth.

Trial 2 - Try 1.0 feet. n= 0.039.

A=8.4; P=12.1; R=0.67; Q=19.5 cfs Too low. Try higher depth.

Trial 3 - Try 1.1 feet. n=.0372

A=8.1; P=12.1; R=0.67; Q=25.07 cfs. Say 24 cfs. OK

Find the shear stress on the bare RECP liner.

$$\tau_{max} = \gamma \times d \times S = 62.4 \times 1.1 \times 0.01 = 0.69 \text{ lbs/ft}^2$$

0.69 lbs/ft^2 is less than the allowable value of 2.0 lbs/ft^2. The RECP liner should adequately protect the channel until vegetation is established.

Wattles

Source: Clackamas County, 2000

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control			
Sediment Control	■		
Runoff Reduction			
Flow Diversion	■		

Description: Wattles are a sediment and stormwater velocity control device. They are tubes of straw, rice straw, or coconut husk encased in ultraviolet (UV) degradable plastic netting or 100% biodegradable burlap material. Wattles help stabilize slopes by breaking up the length, and by slowing and spreading overland water flow.

Typical Uses: Wattles may be suitable along the toe, top, face, and at grade breaks of exposed and erodible slopes to shorten slope length and spread runoff as sheet flow; at the end of a downward slope where it transitions to a steeper slope; along sidewalks and curbs to prevent sediment from washing into gutters; around storm drains and drop inlets; down-slope of exposed soil areas; and around temporary material spoil and stockpiles, such as topsoil and for streambank (sensitive area) protection.

Advantages:
- Lightweight, easy to stake, and may be installed quickly.
- Removal is not necessary, as wattles are typically left in place permanently to biodegrade and/or photo-degrade.
- Wattles come in a variety of diameters and lengths.

Limitations:
- They are difficult to move once saturated.
- Wattles are temporary, lasting only one or two seasons.
- If not properly staked and trenched in, wattles can be transported by high flows.
- Wattles have a very limited sediment capture zone.
- Wattles should not be used on slopes subject to creep or slumping.

Longevity: Varies, 3 to 6 months or until sediment accumulates to one-half the height of the wattle

SUDAS Specifications: Refer to Section 9040, 2.06 and 3.09

A. Description/Uses

Wattles are formed by filling tubular netting with fibrous organic material such as straw, rice straw, or coconut fiber inside of a mesh sock. Alternatively, a wattle may be constructed by tightly rolling a straw/coconut erosion control blanket to form a multi-layer roll.

The completed wattle consists of a long, flexible tube that may be installed along the contours of slopes or at the base of slopes to help reduce soil erosion and retain sediment. Wattles can be highly effective when they are used in combination with other surface soil erosion/re-vegetation practices, such as surface roughening, straw mulching, erosion control blankets, and hydraulic mulching. When wattles are placed at the toe and on the face of slopes, they intercept runoff, reduce its flow velocity, release the runoff as sheet flow, and provide removal of sediment from the runoff. By interrupting the length of a slope, wattles can also reduce erosion.

B. Design Considerations

Wattles should be used to intercept and control sheet flow and should not be used for situations with concentrated flows greater than 1/2 cfs.

Installation of wattles begins by constructing a shallow trench, 2 to 4 inches deep, and shaped to accept the wattle, along the contour of the slope. All debris (rocks and clods) that would prevent close contact between the wattle and soil should be removed. The wattle is placed in the trench, and excavated material from the trench is packed tightly along the base of the wattle, on the uphill side. The wattle should be secured with 1 inch by 1 inch wooden stakes. The stakes should be placed at a 4 foot spacing and driven in perpendicular to the slope through the center of the wattle leaving less than 2 inches of stake exposed above the wattle. The terminating ends of each wattle installation should be turned uphill a minimum of 6 inches to prevent runoff from flowing around the ends of the wattle.

When practical, the wattles may be left in place. Over time, they will break down, decay, and eventually disappear completely. When wattles are removed, any trenches, depressions, or other ground disturbances caused by the removal of the wattle should be backfilled and repaired with the excess sediment captured by the wattle, prior to spreading the straw or other final erosion control protection.

1. **Flat Ground Application:** Install along sidewalks and behind curbs, fitting tightly against the concrete before backfilling, then backfill the wattle to create a trench.

2. **Storm Drain Inlet Protection:** Wattles placed along the back of curb should be offset, as required to go around structures such as curb intakes that project behind the back of curb. At these locations, the wattle should be placed behind the structure (not over it) and shaped to direct water around either side of the structure to prevent ponding. At area intake locations, a shallow trench should be constructed 1 to 2 feet away from the edge of the intake. The wattle should be placed in the trench and firmly staked in place.

3. **Slope Application:**

   a. Wattles should be installed on the contour.

   b. Wattles should be installed from the bottom of the slope up.

4. **Materials:** Wattles can be made from straw, rice straw, coconut husk, or other approved material. The netting consists of biodegradable burlap or high-density polyethylene and ethyl vinyl acetate containing ultraviolet inhibitors.

   Straw should be Certified Weed Free Forage, by a manufacturer whose principle business is wattle manufacturing. Coir (coconut fiber) can be in bristle and mattress form, and should be obtained from freshwater cured coconut husk.

C. Application

Wattles are available in a variety of diameters ranging from 9 inches to 20 inches. The most common sizes are 9 and 12 inch wattles. The allowable spacing for these diameters is given in Table 7E-6.01.

Table 7E-6.01: Recommended Wattle Spacing by Slope

Slope	**Spacing Intervals (feet)**	
	9" Diameter	***12" Diameter***
< 4:1	20	40
2:1 to 4:1	15	30
2:1 or greater	10	20

For soft, loamy soils, the spacing interval should be decreased. For hard, rocky soils, the spacing interval shown in Table 7E-6.01 may be increased.

For highly erosive soils, and for slopes 2:1 or greater, an additional row of wooden stakes should be provided on the downhill side of the wattle.

D. Maintenance

Repair or replace split, torn, unraveling, or slumping wattles.

If the wattle is used as a sediment capture device, or as an erosion control device to maintain sheet flows, sediment that accumulates in the wattle must be periodically removed when accumulation reaches one-half the designated sediment storage depth, usually one-half the distance between the top of the fiber roll and the adjacent ground in order to maintain effectiveness.

If wattles are used for reduction of slope length, sediment removal should not be required as long as the system continues to control the grade. Additional sediment control practices are required to be used in conjunction with this type of application.

Check Dams

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Check dams, sometimes called ditch checks, consist of a vertical barrier constructed across swales, ditches, and waterways. These structures are most commonly constructed of erosion stone, although silt fence and manufactured devices are also used. Straw bales were used at one time, however, due to their high rate of failure and low level of effectiveness, their use is severely limited.

Typical Uses: Check dams are used to control the velocity of concentrated runoff in ditches and swales, and to prevent gully erosion until the channel can be stabilized. The structures may also provide some sediment removal benefits, however this is not their primary function.

Advantages:
- Highly effective at reducing flow velocities in channels.
- Simple to construct.
- Low maintenance.

Limitations:
- Steep slopes require short spacing between check dams.
- Sediment removal practices are still required.
- Straw bales are ineffective and prone to failure.
- Removal difficulties if not permanent

Longevity: Rock check dams - 1 year; may be considered permanent. Manufactured devices and silt fence - 6 months.

SUDAS Specifications: Refer to Section 9040, 2.07 and 3.10

A. Description/Uses

A check dam is a small, temporary obstruction in a ditch or waterway used to prevent erosion by reducing the velocity of flow. A dam placed in the ditch or channel interrupts the flow of water, thereby reducing the velocity. Although some sedimentation may result behind the dam, check dams do not function as sediment trapping devices and should not be designed as such.

Check dams are most commonly constructed of loosely placed erosion stone or rip rap, or from stone-filled gabions.

Silt fence, placed across a ditch or swale, is often used incorrectly under moderate or high flows as a check dam. Silt fence may be used as a check dam; however, it should be limited to applications where the flow rate will be less than 1 cfs. See Section 7E-14 for additional information on using silt fence as a ditch check.

A variety of manufactured devices are also available for installation as ditch checks. One type of manufactured ditch check consists of a 9 to 10 inch tall, triangular-shaped structure constructed from sheets of perforated HDPE (High Density Polyethylene Pipe). Another manufactured product is constructed from a length of triangular-shaped urethane foam. The foam is wrapped in a geotextile fabric for protection.

Gravel bag berms, formed from a pile of gravel-filled bags, may also be used to construct check dams. The bags may be constructed from a variety of porous fabrics, and are filled with clean, poorly-graded gravel. The purpose of the bag is to prevent individual gravel particles from being dislodged, and to allow the gravel barrier to be easily removed or relocated upon completion of the project.

Straw bales were commonly used in the past. However, field experience has shown that this technique is highly ineffective and prone to failures.

B. Design Considerations

Regardless of the type of check dam installed, the concept for controlling the flow is the same. The check dam interferes with the flow in the channel, dissipating the energy of the flowing water, thereby reducing velocity and channel erosion.

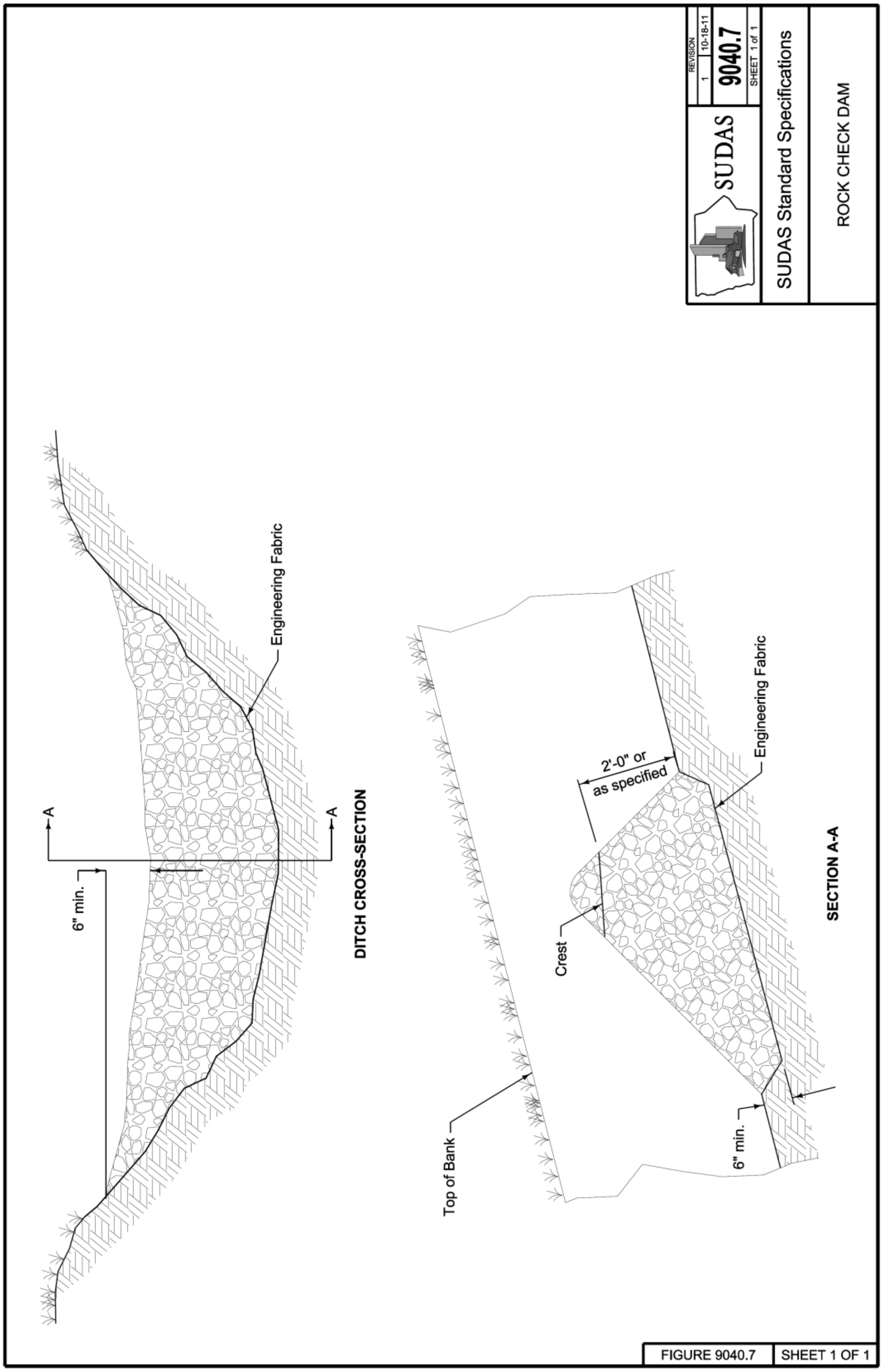
Check dams are not intended to control flows from large drainage areas. Typically, the maximum drainage area to a check dam should be limited to approximately 2 acres.

Check dams should be designed to pass the two-year storm without overtopping the roadway or side slopes of the channel. A weir equation can be used to determine the depth of flow over the structure if necessary.

1. **Rock Check Dams:** Rock check dams should be placed on top of a blanket of engineering fabric to prevent erosion of the underlying surface as water filters through the dam. A typical stone check dam is 2 feet high, with a 4 foot base and 2:1 side slopes. The crest of the check dam should be 6 inches lower than the sides to prevent flows from going around the dam, and eroding the sides of the channel. These dimensions are approximate, and may be modified based upon individual needs and for larger flows. However, heights much greater than 2 feet increase the potential for scour on the downstream side of the dam. For larger check dams, additional channel protection may be required on the downstream side.

The aggregate used should be large enough to prevent the flows from pushing individual stones downstream. A 6 inch erosion stone is normally sufficient.

Figure 7E-7.01: Typical Rock Check Dam
(SUDAS Specifications Figure 9040.7)



2. **Manufactured Devices:** Triangular-shaped manufactured products should be designed and installed according to their manufacturer's recommendations. These products require anchoring to the ground to keep them in place and may require the installation of a blanket of engineering fabric below them.

3. **Gravel Bag Berms:** Gravel bag berms should be placed and spaced in the same manner as rock check dams. The berms should be placed on a layer of engineering fabric, and be limited to a height of 24 inches. The crest of the check dam should be 6 inches lower than the sides to prevent flows from going around the dam, and eroding the sides of the channel.

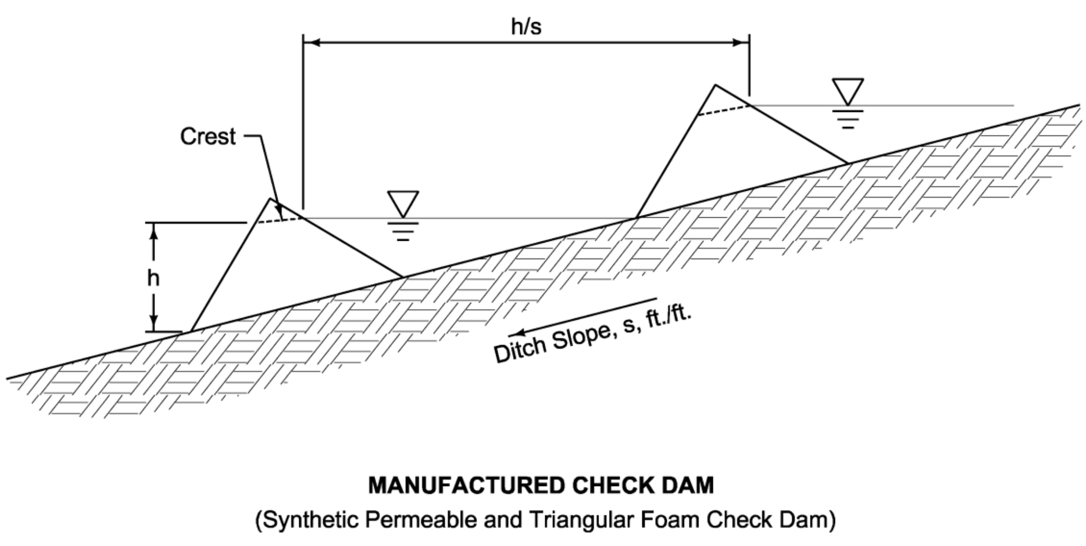
4. **Silt Fence:** Silt fence may be used as a ditch check device for very low flow applications. See Section 7E-14 for additional information on this application.

C. Application

Achieving the proper spacing is the most important aspect of check dam design. The spacing between structures is dependent on the height of the check dam, and the grade of the waterway. In order to protect the channel between the check dams, the devices should be spaced such that the elevation of the toe of the upstream check dam is equal to the elevation of the crest of the downstream check dam. This allows the water between the check dams to pond, resulting in a greatly reduced flow velocity.

As a rule, check dams should not be spaced closer than 20 feet in order to allow for proper maintenance. If slopes and check dam height call for a spacing closer than 20 feet, a Rolled Erosion Control Product or Turf Reinforcement Mat should be considered as an alternative.

Figure 7E-7.02: Typical Check Dam Spacing
(From SUDAS Specifications Figure 9040.6)



D. Maintenance

Check dams should be inspected for damage every seven days and after any 1/2 inch or greater rainfall until final stabilization is achieved. Sediment should be removed when it reaches one-half of the original dam height. Upon final stabilization of the site, the check dams should be removed, including any stone that has been washed downstream, and any bare spots stabilized.

E. Time of Year

Check dams function on a year-round basis.

F. Regional Location

Check dams should be designed to account for the individual characteristics of each site.

Temporary Earth Diversion Structures

Source: Clackamas County, 2000

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Consists of an excavated swale, berm, or combination of the two, constructed in such a manner as to direct water to a desired location.

Typical Uses: Diversion structures are used to intercept surface and shallow subsurface flows and divert this water away from disturbed areas, active gullies, and critically eroding areas. Diversion structures can also be constructed along slopes to reduce the slope length, intercepting and carrying runoff to a stable outlet point or letdown structure.

Advantages:
- Reduces the volume of flow across disturbed areas, thereby reducing the potential for erosion.
- Breaks up concentration of water on long slopes.
- Maintaining a separation between clean water and sediment-laden water allows sediment basins and traps to function more efficiently.
- Easily constructed with equipment found on most construction sites.

Limitations:
- High flow velocities can cause erosion in the diversion structure.
- Diversion structures must be stabilized immediately after installation.

Longevity: One year

SUDAS Specifications: Refer to Section 9040, 3.11

A. Description/Uses

Diversion structures consist of swales or berms that are used to temporarily divert water around an area that is under construction or is being stabilized. Specific applications include perimeter control, diversion away from disturbed slopes, and diversion of sediment-laden water to treatment facilities.

As a perimeter control, temporary swales and/or berms may be constructed above a large disturbed area to divert upstream run-on around the site. This serves several purposes. First, the amount of runoff flowing over the disturbed area is reduced, thereby reducing the erosion potential. Secondly, clean water can be separated from the sediment-laden water and can be passed through or around the site. Sediment-laden water can be directed to a sediment trap or basin for treatment. Separating the upstream runoff from the sediment-laden water allows the designer to reduce the required size of the sediment removal structure, and allows the structure to work more efficiently.

Another specific use of a diversion structure is to keep upstream stormwater off of disturbed slopes or to safely carry it down the slope. This is accomplished by constructing a swale and/or berm at the top of the slope, and conveying it to a letdown structure or stable outlet. On long slopes, they can be placed at regular intervals to trap and divert sheet flow before it concentrates and causes rill and gully erosion.

B. Design Considerations

Diversion structures should be designed to carry peak flows from the 2 year, 24 hour storm. The maximum drainage area conveyed through a diversion structure should be 5 acres.

The depth of the diversion should be based upon the design capacity, plus an additional 4 inches of freeboard. The minimum depth provided should be 18 inches. This may be provided solely by a berm or swale or may be developed with a combination of berm and swale. The shape of the diversion may be parabolic, trapezoidal, or V-shaped, with side slopes of 2:1 or flatter.

The minimum slope of the diversion structure should be sufficient to carry the design flow. The maximum slope of the diversion is limited by the permissible velocities of flows within the structure, as shown in the following table. Since any existing vegetation will likely be destroyed upon construction of the diversion structure, the bare surface situation should be considered for most applications.

Table 7E-8.01: Diversion Structure Slopes by Soil Type

Soil Type	Permissible Velocity (fps)			
	Channel Vegetation			
	Bare	***Poor***	***Fair***	***Good***
Sand, silt, sandy loam, and silty, loam	1.5	1.5	2.0	3.0
Sandy clay, loam, and sandy clay, loam	2.0	2.5	3.0	4.0
Clay	2.5	3.0	4.0	5.0

Source: Smoot, 1999

After construction of the diversion structure, it is important to stabilize the surface immediately with seed and mulch, sod, or other means.

C. Application

Diversion structures should be used around the perimeter of sites to prevent run-on of off-site flows over disturbed ground.

D. Maintenance

The channel should be inspected every seven days and after any 1/2 inch or greater rainfall. Any damage to the vegetated lining should be repaired. All debris should be removed and properly disposed of to provide adequate flow conveyance.

E. Time of Year

When diversion structures are constructed during times when vegetation cannot be established to stabilize the surface, alternative stabilization methods such as sodding or matting may be required.

F. Regional Location

As mentioned above, the allowable velocity within the diversion structure is based upon the soil characteristics of the site. Silty and sandy soils are more prone to erosion than clay soils. However, with the proper design and stabilization methods, diversion structures may be used in all appropriate locations.

Level Spreaders

Source: Umstead Coalition, 2005

BENEFITS

	L	M	H
Flow Control	■	■	■
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion	■	■	

Description: A level spreader is a low-cost method to convert small volumes of concentrated runoff into sheet flow and release it onto an area stabilized by existing vegetation.

Typical Uses: Level spreaders are commonly used at the outlet of a diversion structure or sediment removal structure to convert concentrated flow to uniform sheet flow prior to releasing the runoff onto stabilized downstream slopes. Level spreaders are also used to convey runoff from impervious surfaces, such as parking lots, onto vegetated areas or into detention basins.

Advantages:
- Widely used BMP due to ease of installation and availability of materials.
- Low cost and simple to construct.

Limitations:
- Flows from a level spreader should be limited to clean, diverted runoff, or runoff that has been passed through a sediment removal structure.
- The downstream slope must have existing vegetation and be capable of accepting sheet flow without incurring erosion.
- May require adjustment after freeze-thaw cycle due to heaving.

Longevity: One year

SUDAS Specifications: Refer to Section 9040, 2.08 and 3.12

A. Description/Uses

A level spreader is a device used at the outlets of dikes and berms to convert the concentrated flows to sheet flow prior to discharging the flow onto a vegetated area downstream of the disturbed site.

A level spreader normally consists of a shallow excavation that serves as a stilling basin to allow runoff to pond up and dissipate its kinetic energy. An overflow weir is constructed to release the accumulated runoff. This weir is normally constructed from a 2 by 8 inch pressure-treated wooden timber placed at 0% grade to ensure uniform flow over the weir. For low flow applications, an earthen weir may also be constructed; however, special attention must be paid to ensure that the weir is level. If low points exist, concentrated flows will result and these could cause damage to the weir and the downstream slope.

B. Design Considerations

The grade of the last 20 feet of the diversion structure channel should be 1% or less to slow the velocity of the flow prior to draining into the depression. This will help reduce turbulence and erosion within the depression.

It is imperative that the receiving area downstream of the weir be stabilized sufficiently to receive the flows from the spreader without causing erosion. The receiving area must also be smooth to preserve the sheet flow and prevent the flow from concentrating. The slope of the receiving area should be less than 10%.

For level spreaders constructed from earthen embankments, a layer of erosion control matting should be placed on either side of the weir to provide additional stability to the surface.

C. Application

The length of the weir and depth of the depression required behind the weir are dependent on the anticipated flows over the weir. Select the length and depth of the spreader from Table 7E-9.01 based upon the 10 year peak flow.

Table 7E-9.01: Level Spreader Properties

Flow (cfs)	Min. Depth (feet)	Min. Length (feet)	Material
0-4	0.5	10	Stabilized Earth
5-10	0.5	10	2” x 8” Timber
10-20	0.6	20	2” x 8” Timber
20-30	0.7	30	2” x 8” Timber
30-40	0.8	40	2” x 8” Timber

D. Maintenance

The downstream slope should be inspected for signs of rilling. If rilling occurs, the length of the spreader may need to be increased, or additional stabilizing practices may need to be employed on the slope. If silt accumulates within the depression, it should be cleaned out when it loses one-third of its volume.

After a freeze-thaw cycle, the level spreader should be inspected to ensure that heaving has not occurred. Any displacement should be corrected to ensure that it is completely level.

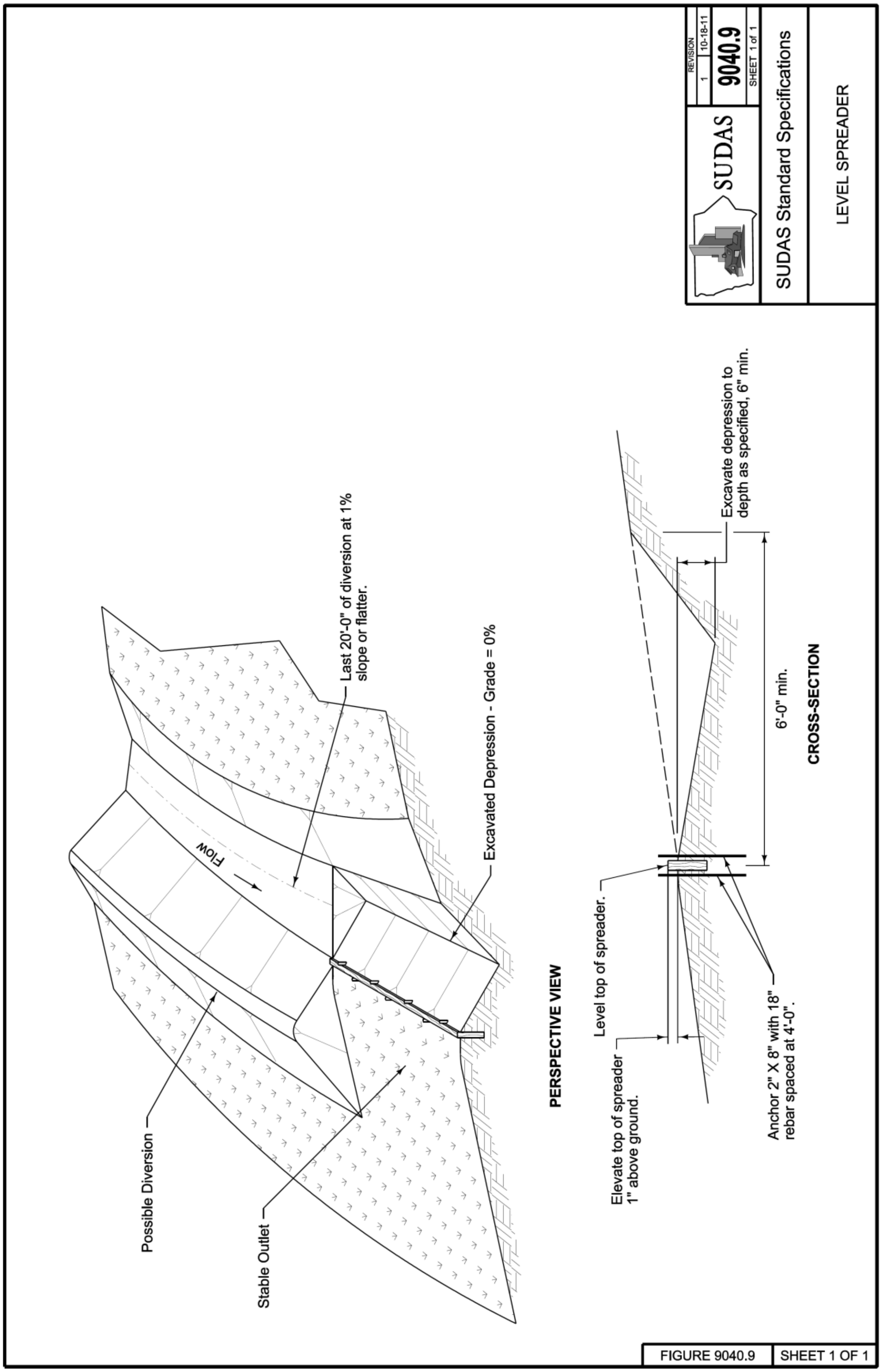
E. Time of Year

Level spreaders will function on a year-round basis.

F. Regional Location

For soils that are highly sensitive to erosion, even when fully vegetated, the length of the spreader may need to be increased beyond that shown in the table.

Figure 7E-9.01: Typical Level Spreader Configuration
(SUDAS Specifications Figure 9040.9)



Rip Rap

Source: Mississippi State University

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Rip rap is a common method of protecting a channel downstream of a storm sewer or culvert outlet from erosion. A layer of crushed stone placed on the bottom and sides of the channel protects the channel and dissipates the energy of the high velocity flow.

Typical Uses: Used at the outlet of storm sewer pipes, roadway and driveway culverts, and at any point concentrated runoff enters a channel.

Advantages:

- Widely used method of erosion protection.
- Materials are readily available in most areas.
- Effective at reducing scour when properly designed and installed.

Limitations:

- Commonly undersized.
- Not aesthetically pleasing.
- May not be adequate for flows from large pipes (>48 inches).
- May be higher cost due to limited availability of stone.

Longevity: Temporary or permanent

SUDAS Specifications: Refer to Section 9040, 2.09 and 3.13

A. Description/Uses

The most common method of protecting a channel at an outlet is to place a layer of crushed stone along the bottom and sides of the channel. The purpose of the stone is to protect the channel until the outlet flow loses sufficient velocity and energy, so that erosion will not occur in the downstream channel. Rip rap is provided by constructing a blanket of crushed stone, to a specified depth at the outlet. The layer of the stone is constructed so that the top is flush with the invert elevation of the outlet pipe. The stone should be placed on a layer of engineering fabric to protect the underlying soil from the erosive action of the churning water.

For larger pipes, or for discharges from pipes with large head pressures, greater protection may be required. Additional protection can be provided by constructing a rock-lined plunge pool, stilling basin, or through the use of concrete energy dissipaters (see Chapter 2 - Stormwater).

B. Design Considerations

The following design information only applies to the design of rock protection at outlets. It does not apply to rock lining of channels or streams. In addition, the design of rock plunge pools or stilling basins, and other types of energy dissipaters is not covered in this section. Refer to the Federal Highway Administration Hydraulic Engineering Circular No. 14 (HEC-14), "Hydraulic Design of Energy Dissipators for Culverts and Channels" for information on designing these structures.

The Iowa DOT Culvert Program (version 2.0) includes three methods of designing rock protection at the outlet of culverts. The methods include HEC-14 rip rap basins, U.S. Army Corps of Engineers scour hole design and U.S. Bureau of Reclamation plunge basin design. This program is available online and can be obtained from the Iowa DOT's Office of Bridges and Structures.

The steps below describe the method of designing rip rap:

1. **Tailwater Depth:** The first step is to find the tailwater depth at the pipe outlet, corresponding to the appropriate design-year storm event for the outlet structure (see Chapter 2 - Stormwater) for design criteria for various structures). Normally, the tailwater depth is found by determining the normal depth in the channel using Manning's equation (see Chapter 2 - Stormwater). If downstream restrictions such as a culvert, dam or channel constriction exist, a more thorough analysis is required.

   If the tailwater is less than half of the discharge flow depth (pipe diameter or box height if flowing full) it is classified as a *minimum tailwater condition.* If the tailwater is greater than or equal to half of the discharge flow depth, it is classified as a *maximum tailwater condition.* The tailwater condition will determine which figure (Figure 7E-10.03 or 7E-10.04) to use to find the necessary rock size and apron dimensions.

   Pipes that outlet onto flat areas without a well-defined channel can be assumed to have a minimum tailwater condition.

   If the tailwater condition cannot be easily determined for a channel, the apron should be designed for the maximum tailwater condition as a conservative approach.

2. **Stone Size:** As the discharge flows over the crushed stone, the flow imposes shear stresses on the individual stones. Since the stones are only held in place by the force of gravity, they must have sufficient mass to prevent them from being dislodged by the force of the flowing water. For rip rap design, the crushed stone material is selected based upon its average, or d_{50}, diameter. The d_{50}

diameter represents the size at which half of the individual stones (by weight) are smaller than the specified diameter.

The d_{50} diameter is determined with Figure 7E-10.03 or 7E-10.04, for the appropriate tailwater condition. This value represents the minimum average diameter of stone necessary to resist the anticipated flows.

a. **Pipes Flowing Full:** The appropriate figure is entered along the x-axis at the design discharge. A vertical line is projected to the curve for the appropriate pipe diameter in the lower set of curves. From this point, a horizontal projection is made to the right, and the minimum d_{50} diameter is read.

b. **Partially Full Pipes and Box Culverts:** Using the depth of flow and velocity at the outlet, the intersection of d and v in the lower portion of the appropriate figure is found. From this point, a horizontal projection is made to the right, and the minimum d_{50} diameter is read.

Most crushed stone used for outlet protection is specified by weight, not by diameter. The following table lists the standard SUDAS and Iowa DOT revetment and erosion stone weights and corresponding d_{50} diameters. These gradations are also shown on Figures 7E-10.03 and 7E-10.04. Alternative gradations may be selected and specified if available from local aggregate suppliers.

Table 7E-10.01: Standard Revetment and Erosion Stone Properties

Standard Classification	d_{50} Weight (lbs)	Average d_{50} Diameter[1] (feet)	Maximum Weight (lbs)	Avg. max. Diameter[1] (feet)
Class A Revetment Stone	125[2]	1.1[2]	400	1.7
Class B Revetment Stone	275	1.5	650	2.0
Class D & E Revetment Stone	90	1.0	250	1.4
Erosion Stone	---	0.5	---	0.75

[1] Diameters based upon an assumed specific gravity of 2.65.
[2] Approximate values for design purposes. Actual d_{50} value is not specified. (d_{75} = 75 lbs).

3. **Apron Length:** A sufficient length of protection must be provided in order to reduce the velocity and energy of the flow to the level anticipated in the downstream channel. This length is dependent on the volume and velocity of the flow at the discharge point. It is also dependent on the tailwater condition of the downstream channel. The length, L_a, is found from Figure 7E-10.03 or 7E-10.04 for the appropriate tailwater condition.

   From the intersection of discharge and pipe diameter, or for velocity and flow depth found in the previous step, a vertical line is projected to the appropriate discharge depth/pipe diameter in the upper set of curves. From this intersection, a horizontal line is projected to the left to determine the minimum length of rock protection required.

4. **Apron Width:** For pipes that discharge into a well-defined channel, the width of the apron should extend to the top of the bank, or at least 1-foot above the maximum tailwater depth, whichever is less, along the entire length of the apron.

   For outlets that discharge onto flat areas, the width of the apron at the upstream end of the culvert should be three times the diameter of the pipe, or equal to the width of the concrete pipe apron if one is provided. The width of the apron at the downstream end should be equal to the length of the apron, L_a, plus the diameter of the pipe, D.

5. **Apron Depth:** The depth of the apron should be equal to one and one-half times the maximum stone diameter (see Table 7E-10.01 for maximum diameter).

   The channel downstream of the rock apron must be analyzed to ensure that existing or proposed channel liner is sufficient and that it will not be eroded under the anticipated flow depths. Methods for analyzing channel liners can be found in Section 7E-23.

C. Application

Outlet protection should be considered at all pipe and culvert outlets. Rip rap is an easily constructed method of protection and is sufficient for many situations.

D. Maintenance

After installation, rock aprons should be inspected regularly. Special attention should be paid to the end of the apron, as it transitions to a natural channel. If scour or erosion is occurring at this junction, the apron should be extended, and additional stabilization methods may be required.

Figure 7E-10.01: Rip Rap Apron for Pipe Outlet into Channel
(SUDAS Specifications Figure 9040.11)

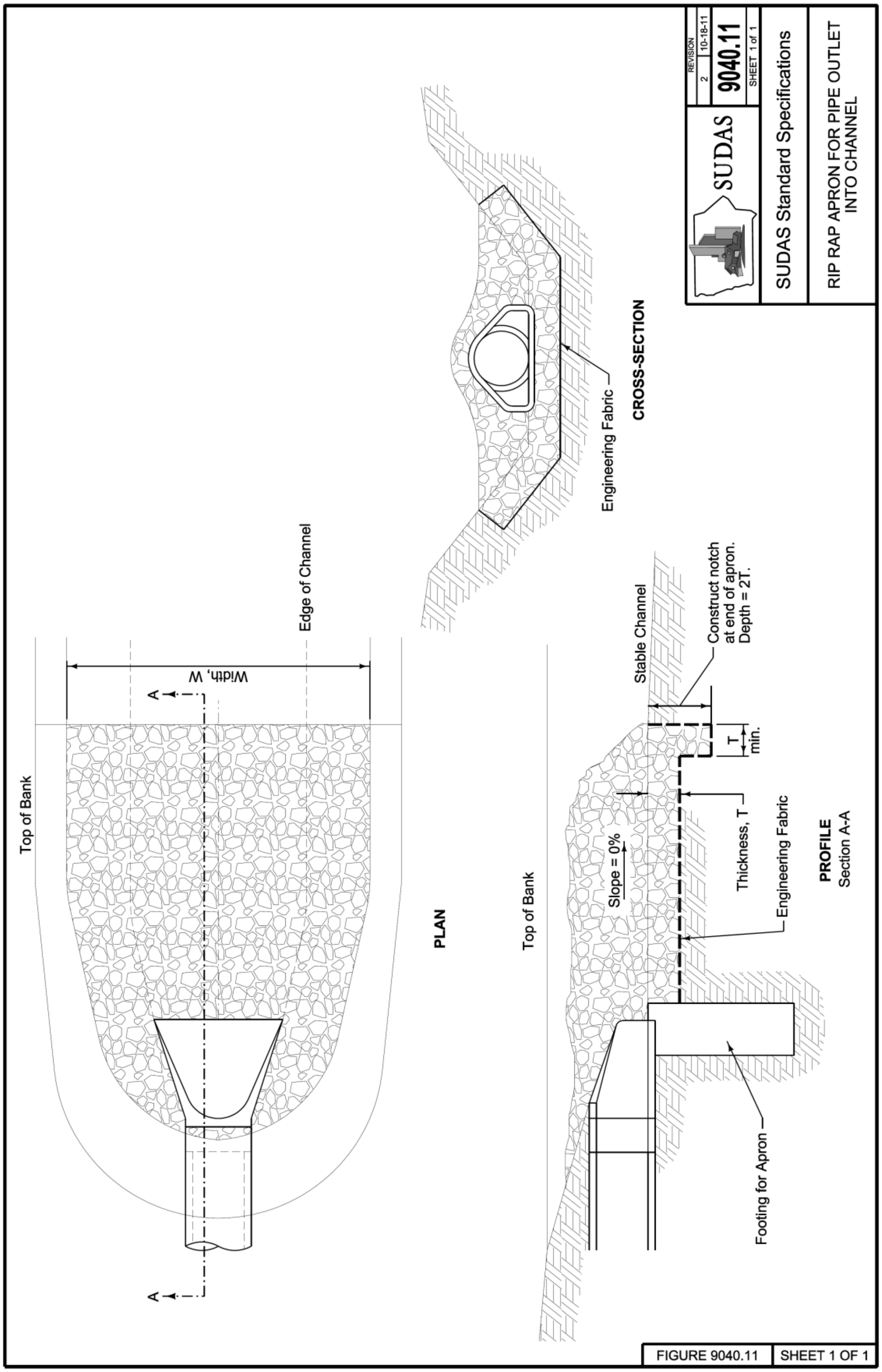


Figure 7E-10.02: Rip Rap Apron for Pipe Outlet onto Flat Ground
(SUDAS Specifications Figure 9040.10)

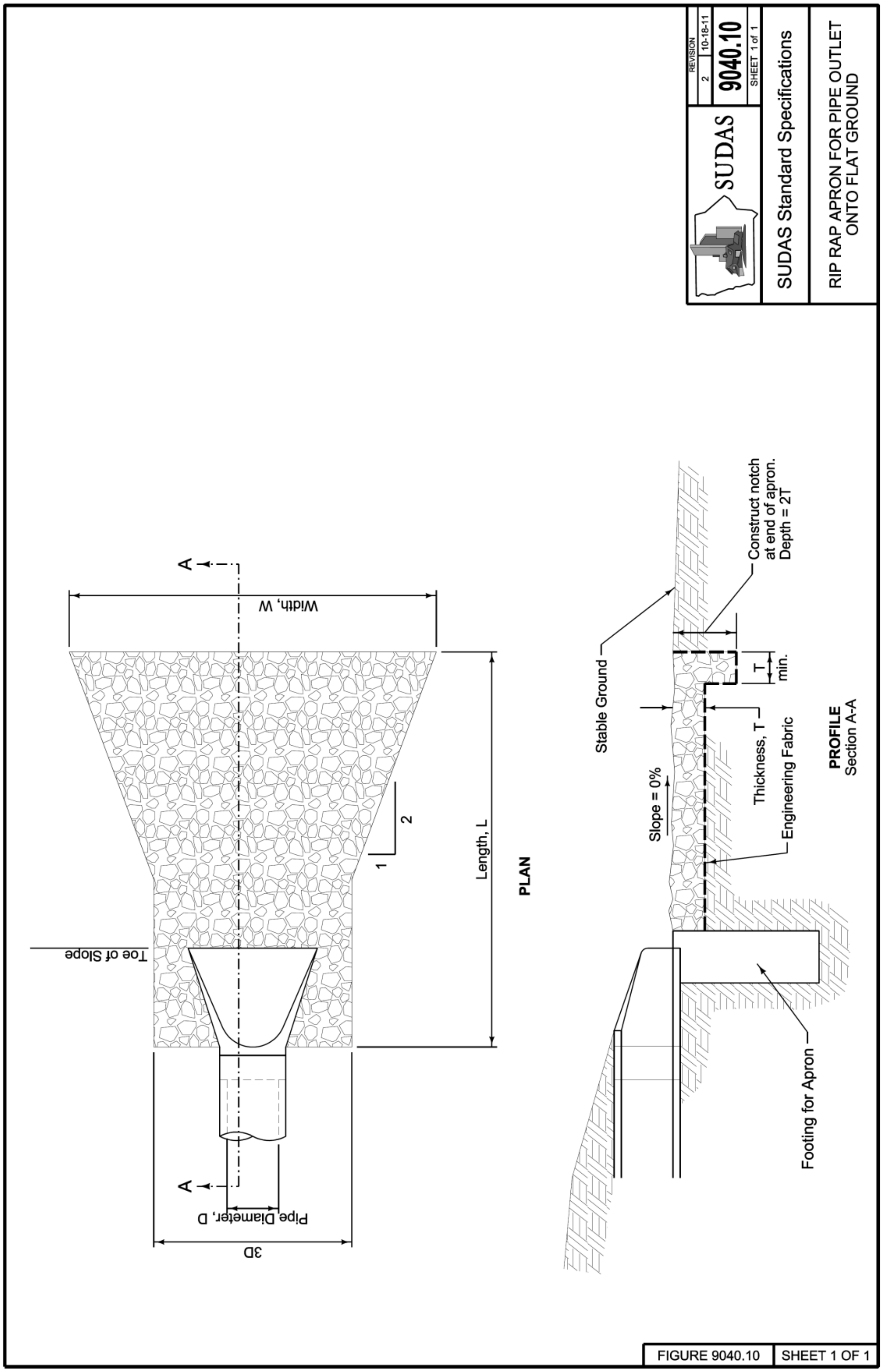
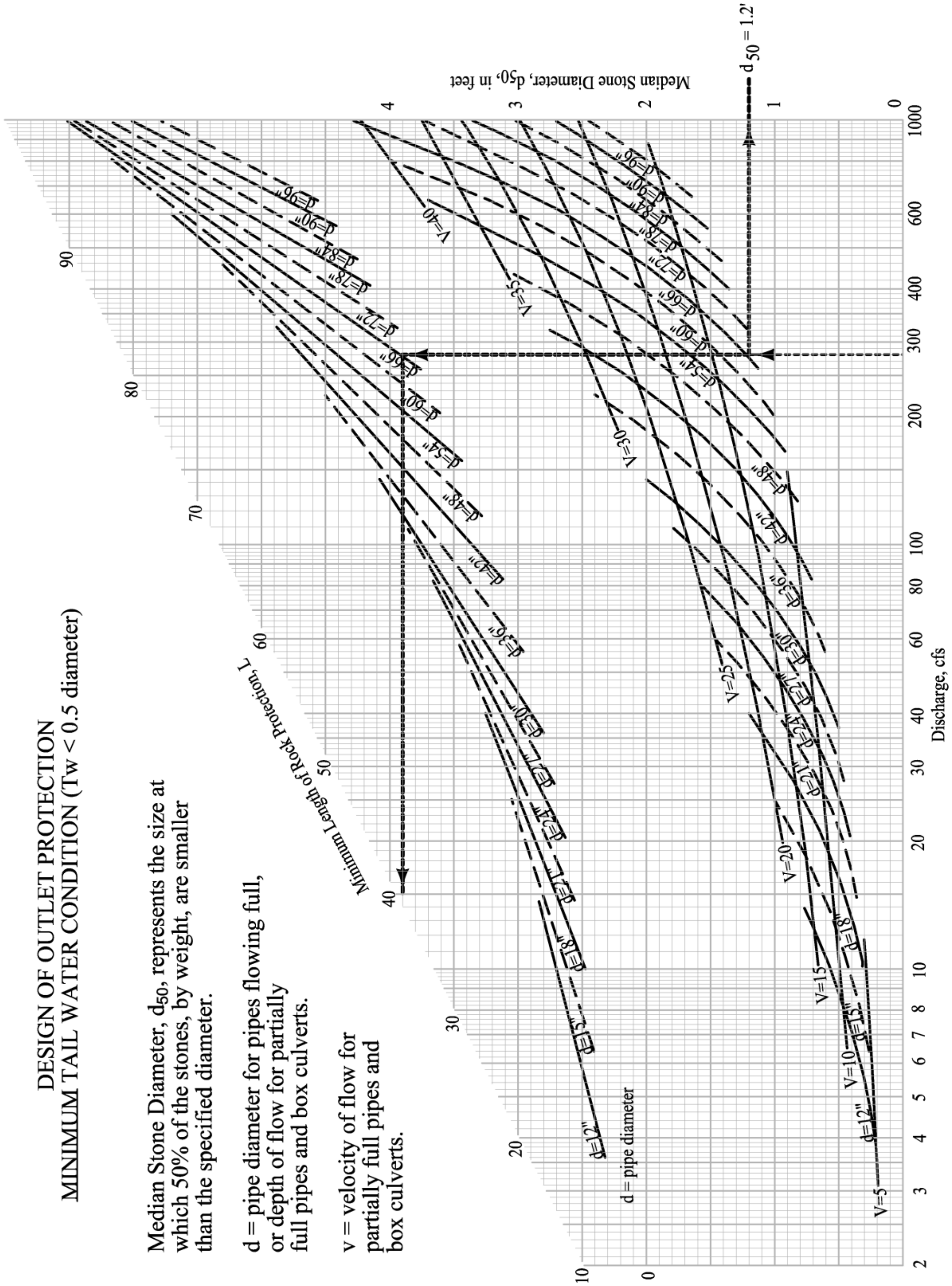
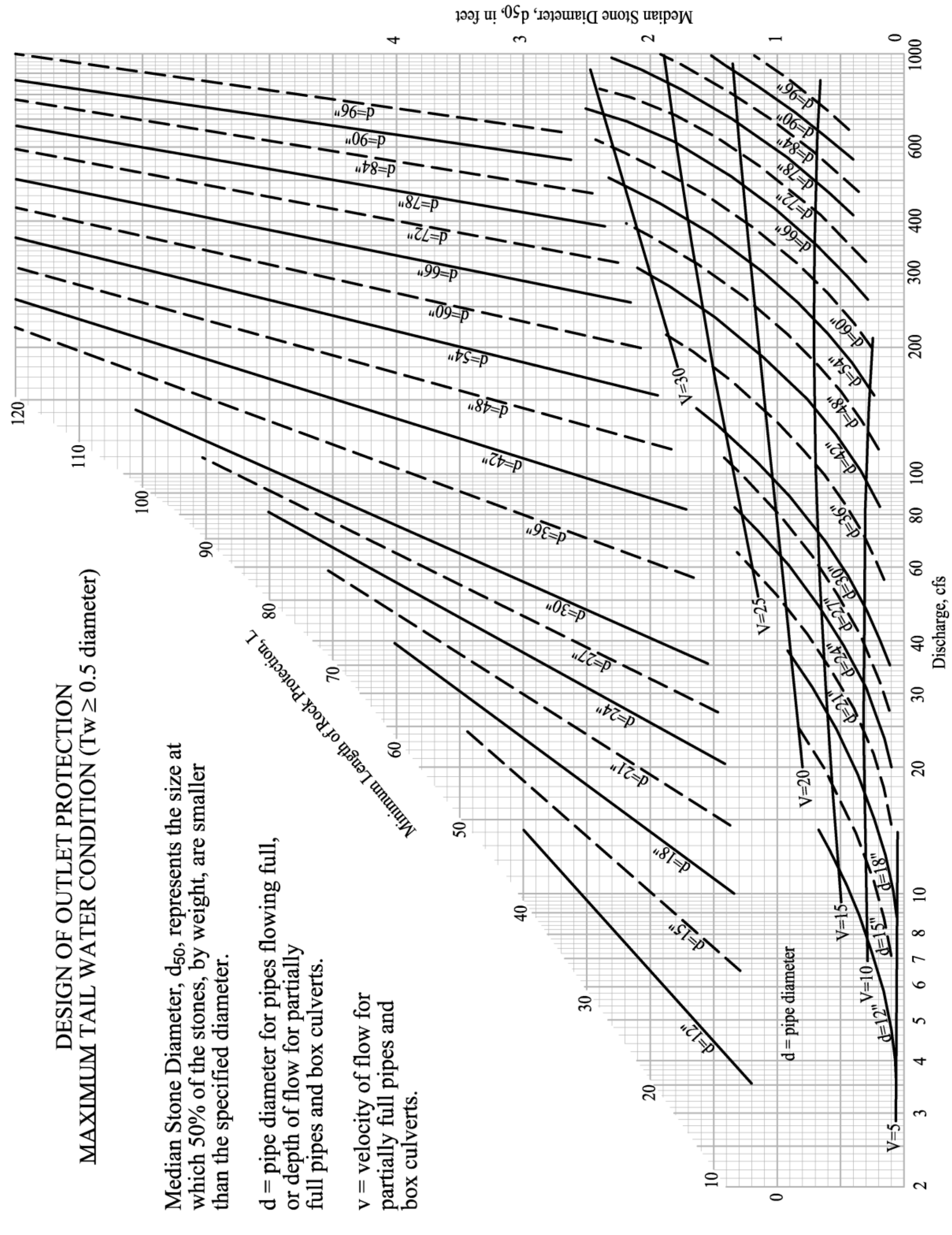


Figure 7E-10.03: Design of Outlet Protection, <u>Minimum</u> Tailwater Condition



Source: USDA NRCS, 2004

Figure 7E-10.04: Design of Outlet Protection, Maximum Tailwater Condition



Source: USDA NRCS, 2004

Temporary Pipe Slope Drains

Source: Mississippi State University

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Temporary slope drains consist of a pipe or tubing, installed from the top to the bottom of a disturbed slope. The drain transports concentrated runoff down the slope to a stabilized outlet, reducing the potential for erosion caused by runoff flowing over the disturbed slope.

Typical Uses: Used to transport concentrated runoff collected by a diversion structure, down a slope to a stable outlet or channel.

Advantages:
- Highly effective method for transporting runoff down a disturbed slope with minimal erosion.
- Easily constructed.
- Materials may be reused.

Limitations:
- Area around drain inlet must be carefully constructed to prevent water from flowing along the pipe, and breaching the diversion.
- The drain outlet must be discharged to a stable area, or outlet protection must be provided.

Longevity: Temporary, until vegetation is established

SUDAS Specifications: Refer to Section 9040, 2.10 and 3.14

A. Description/Uses

Temporary slope drains are constructed of flexible pipe or tubing, running from the top to the bottom of a disturbed slope. Slope drains provide a means of transporting collected runoff from the top of the slope to the bottom of the slope and prevent the erosive potential created by concentrated runoff flowing over the face of a disturbed slope.

Slope drains are commonly used in conjunction with diversion structures. A diversion structure at the top of the slope collects upland runoff and transports it to the desired outlet point. The slope drain provides an outlet for the diversion structure, safely carrying the collected runoff down the slope.

After grading, slopes are highly susceptible to erosion caused by sheet and concentrated flows from upland areas. Stabilizing the slope by seeding can be difficult as runoff over the slope may wash away seed and seedlings. Slope drains are used as a temporary measure to transport runoff down a slope, until the slope can be permanently stabilized. Eliminating flows over the face of a slope reduces erosion and provides newly planted seed an opportunity to establish itself without being washed away.

B. Design Considerations

Temporary slope drains should be sized to carry a two-year storm event. Table 7E-11.01 provides a summary of recommended pipe diameters based upon the contributing drainage area.

Table 7E-11.01: Slope Drain Diameters by Drainage Area

Maximum Drainage Area (acre)	Minimum Pipe Diameter (inches)
0.5	8
1.0	10
1.5	12
2.5	15
4	18
5	21
> 5	Special Design Required

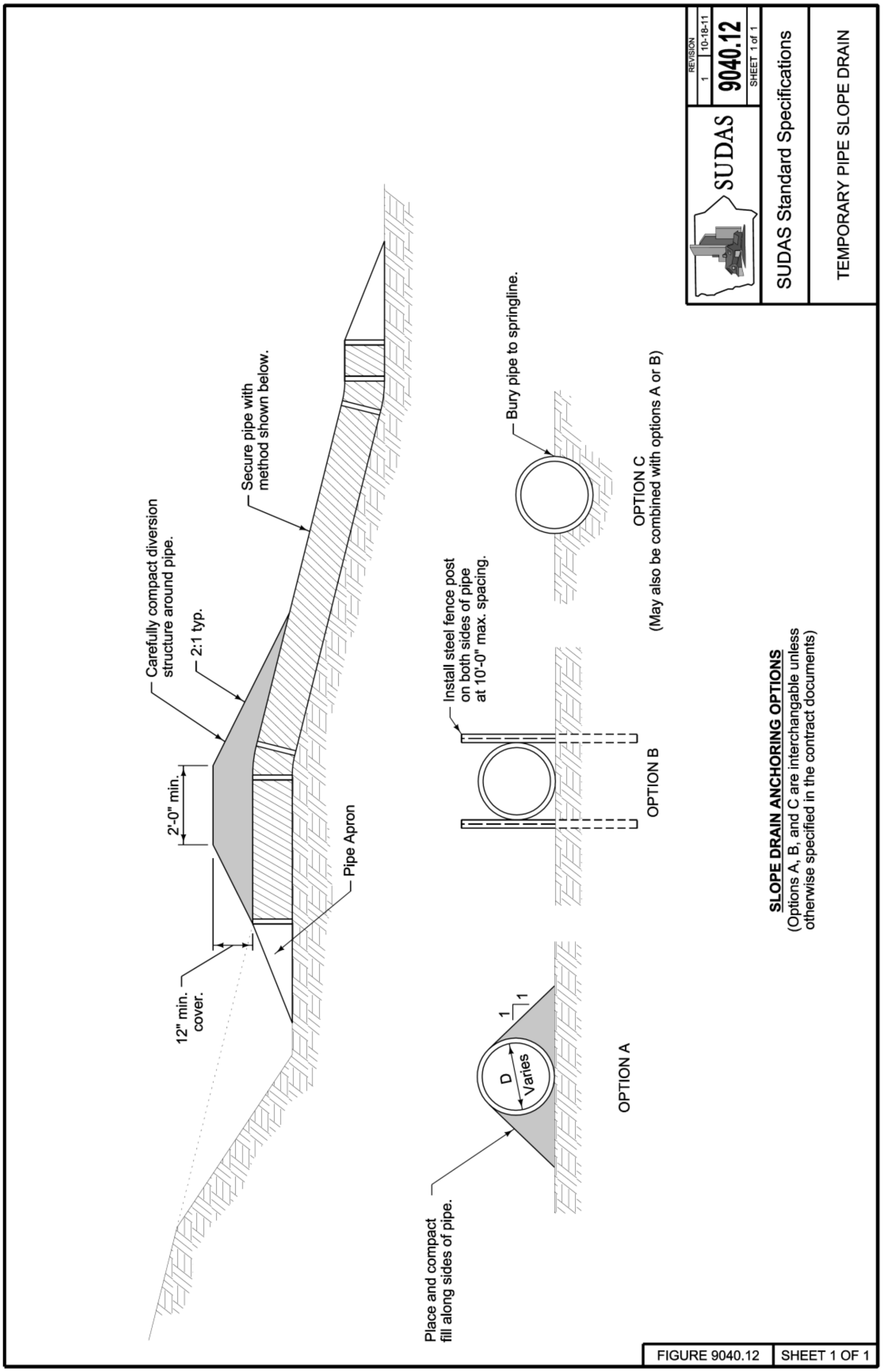
Note: Values assumed a 2 year storm, 15 minute T_c, and a runoff coefficient of 0.5

Slope drains are normally installed in conjunction with diversion structures. The diversion structure should have a height or depth at the pipe inlet of at least 18 inches, or 6 inches greater than the pipe diameter, whichever is larger. The soil under and around the inlet of the pipe should have a low permeability, and be carefully compacted to ensure that seepage does not occur along the pipe-soil interface. The area around the inlet should be graded to ensure that flows are directed toward the pipe inlet.

The slope drain should have a minimum grade of 3%. A metal or flexible apron should be provided at the inlet of the pipe. If the area draining to the diversion and slope drain is disturbed, the slope drain should outlet to a sediment trap or sediment basin. If the upland area is undisturbed, the pipe outlet should bypass any sediment basins or traps, and drain to a stabilized area.

Unless the pipe drains to a stable outlet, protection such as rip rap or a rolled erosion control product may be required at the outlet.

Figure 7E-11.01: Temporary Pipe Slope Drain
(SUDAS Specifications Figure 9040.12)



C. Application

Slope drains should be considered whenever a diversion structure is constructed on a disturbed slope steeper than 3%. When properly incorporated, diversion structures with slope drains provide a method to separate runoff from disturbed and stabilized areas, reducing the size requirements for sediment basins or traps.

D. Maintenance

The slope drain should be inspected for signs of leaking joints, pipe movement, erosion at the inlet and outlet, and seepage through the berm at the inlet.

E. Design Example

Assume the runoff from 7.5 acres of bare ground is intercepted by a diversion structure and carried to the location of a proposed slope drain. Determine the required diameter of the slope drain.

Using the techniques described in Chapter 2 - Stormwater, the following information is determined:

Time of Concentration, T_c = 15 minutes
Rainfall Intensity, I = 3.48 (Region 7)
Runoff Coefficient for bare ground, C = 0.5.

Using this information, the peak runoff is found to be 13.1 cfs by the Rational Method.

The minimum pipe diameter is found with the orifice equation (assume head to top of pipe).

$$Q = (0.6)(A)\sqrt{2gh}$$

Where:

Q	=	Runoff volume, cfs
A	=	Area of pipe opening
g	=	Acceleration of gravity, 32.2 ft/s^2
h	=	Head pressure (h=D/2 for head to top of pipe)

$13.1 = (0.6)\left(\frac{\pi \times D^2}{4}\right)\sqrt{2 \times 32.2 \times \frac{D}{2}}$, Solving for D yields a diameter of 1.9' or 23 inches.

Conclusion: Based upon the analysis, a 24 inch diameter pipe would be selected.

Sediment Basin

BENEFITS

	L	M	H
Flow Control	■	■	■
Erosion Control			
Sediment Control	■	■	■
Runoff Reduction	■		
Flow Diversion			

Description: Sediment basins, like sediment traps, are temporary structures that are used to detain sediment-laden runoff long enough to allow a majority of sediment to settle out. Sediment basins are larger than sediment traps, serving drainage areas between 5 and 100 acres.

Sediment basins use a release structure to control the discharge, and normally have an emergency spillway to release the flow from larger storms. If properly planned, the basins may also serve as permanent stormwater management facilities, such as detention basins or permanent sediment removal structures.

Typical Uses: Used below disturbed areas where the contributing drainage area is greater than 5 acres. Basins require significant space and the appropriate topography for construction.

Advantages:
- Can greatly improve the quality of runoff being released from a site by removing suspended sediment on a large-scale basis.
- May be designed as a permanent structure to provide future detention, or for long-term water quality enhancement.

Limitations:
- Large in both area and volume.
- Use is somewhat dependent on the topography of the land.
- Must be carefully designed to account for large storm events.
- Not to be located within live streams.
- May require protective fencing.

Longevity: 18 months; may be converted to a permanent feature

SUDAS Specifications: Refer to Section 9040, 2.11 and 3.15

A. Description/Uses

Sediment basins, like sediment traps, are temporary structures used to detain runoff so sediment will settle before it is released. Sediment basins are much larger than sediment traps, serving drainage areas up to 100 acres. If properly planned and designed, sediment basins can be converted to permanent stormwater management facilities upon completion of construction.

B. Design Considerations

Adequate storage volume is critical to the performance of the basin. Sediment basins that are undersized will perform at much lower removal efficiency rates. Sediment basin volumes and dimensions should be sized according to the criteria in Section 7D-1.

A sediment basin consists of several components for releasing flows: a principal spillway, a dewatering device, and an emergency spillway. The principal spillway is a structure which passes a given design storm. It also contains a de-watering device that slowly releases the water contained in the temporary dry storage. An emergency spillway may also be provided to safely pass storms larger than the design storm.

1. **Principal Spillway:** The principal spillway consists of a vertical riser pipe connected at the base to a horizontal outlet pipe. The outlet pipe carries water through the embankment and discharges beyond the downstream toe of the embankment.

   The first step in designing a principal spillway is to set the overflow elevation of the riser pipe. The top of the riser should be set at an elevation corresponding to a storage volume of 3,600 cubic feet per acre of disturbed ground. When an emergency spillway is provided, this elevation should be a minimum of 1 foot below the crest of the emergency spillway. If no emergency spillway is used, the top of the riser should be set at least 3 feet below the top of the embankment.

   The next step is to determine the size of the riser and outlet pipes required. These pipes are sized to carry the peak inflow, Q_p, for the design storm. If an emergency spillway will be included, the principal spillway should be designed to handle the peak inflow for a 2-year, 24-hour storm, without exceeding the elevation of the emergency spillway. If an emergency spillway is not included, the principal spillway must be designed to pass the 25-year storm, with at least 2 feet of clearance between the high-water elevation and the top of the embankment. Peak inflow flow rates should be determined according to the methods described in Chapter 2 - Stormwater. The peak rate should account for the lack of vegetation and high runoff potential that is likely to occur during construction.

   The riser size can be determined using the following equations. The flow through the riser should be checked for both weir and orifice flow. The equation, which yields the lowest flow for a given head, is the controlling situation.

   Weir Flow Orifice Flow

   $$Q = 10.5 \times d \times h^{\frac{3}{2}} \qquad Q = 0.6 \times A \times \sqrt{2gh}$$

   Equations 7E-12.01 and 7E-12.02

   Where:

Q	=	Inlet capacity of riser, cfs
d	=	Riser diameter, ft
h	=	Allowable head above top of riser, ft
A	=	Open area of the orifice, ft^2
g	=	Acceleration of gravity, (32.2 ft/s^2)

The allowable head is measured from the top of the riser to the crest of the emergency spillway or to the crest of the embankment if no emergency spillway is provided.

Figure 7E-12.01: Sediment Basin Without Emergency Spillway
(SUDAS Specifications Figure 9040.13)

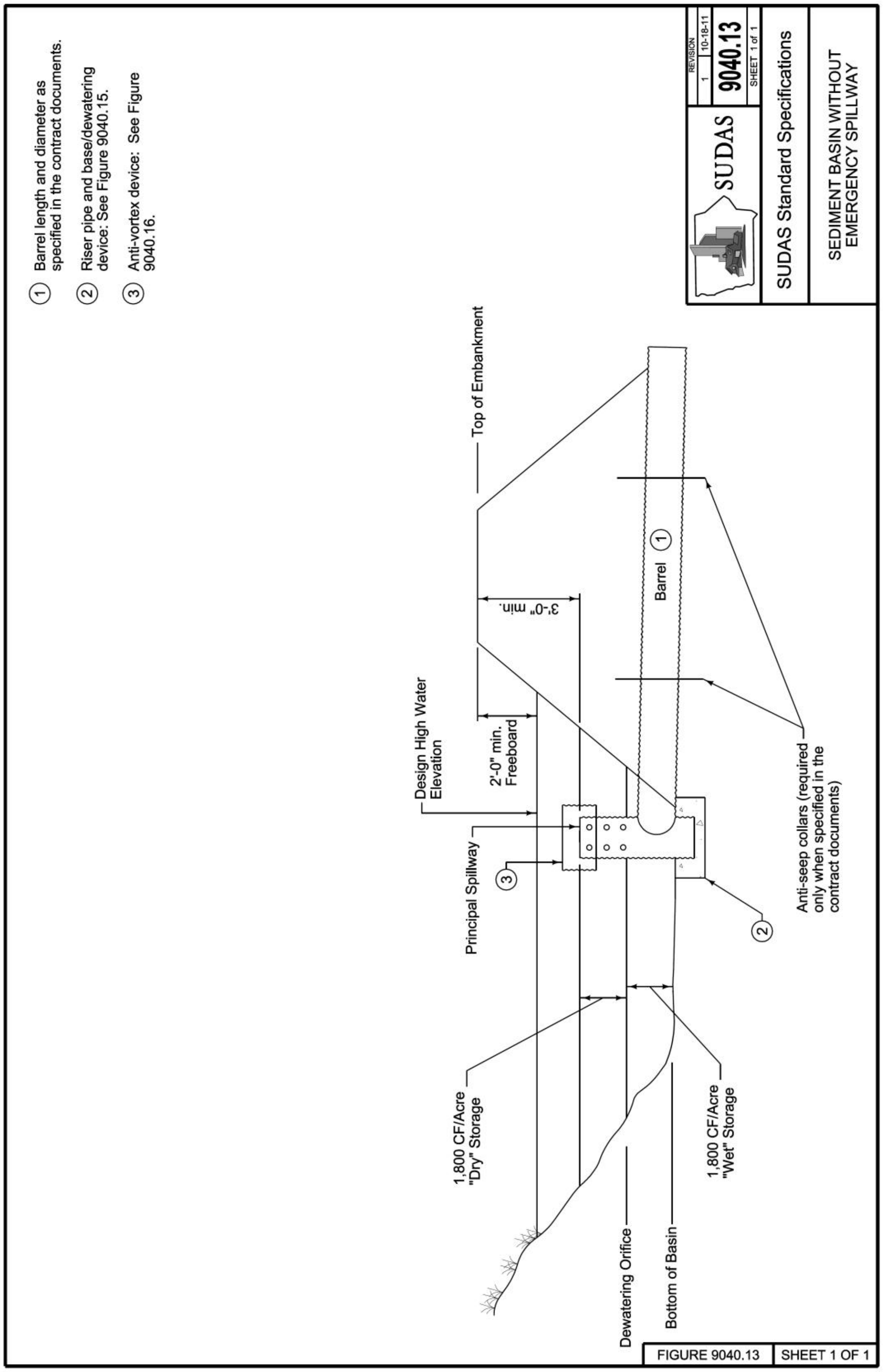
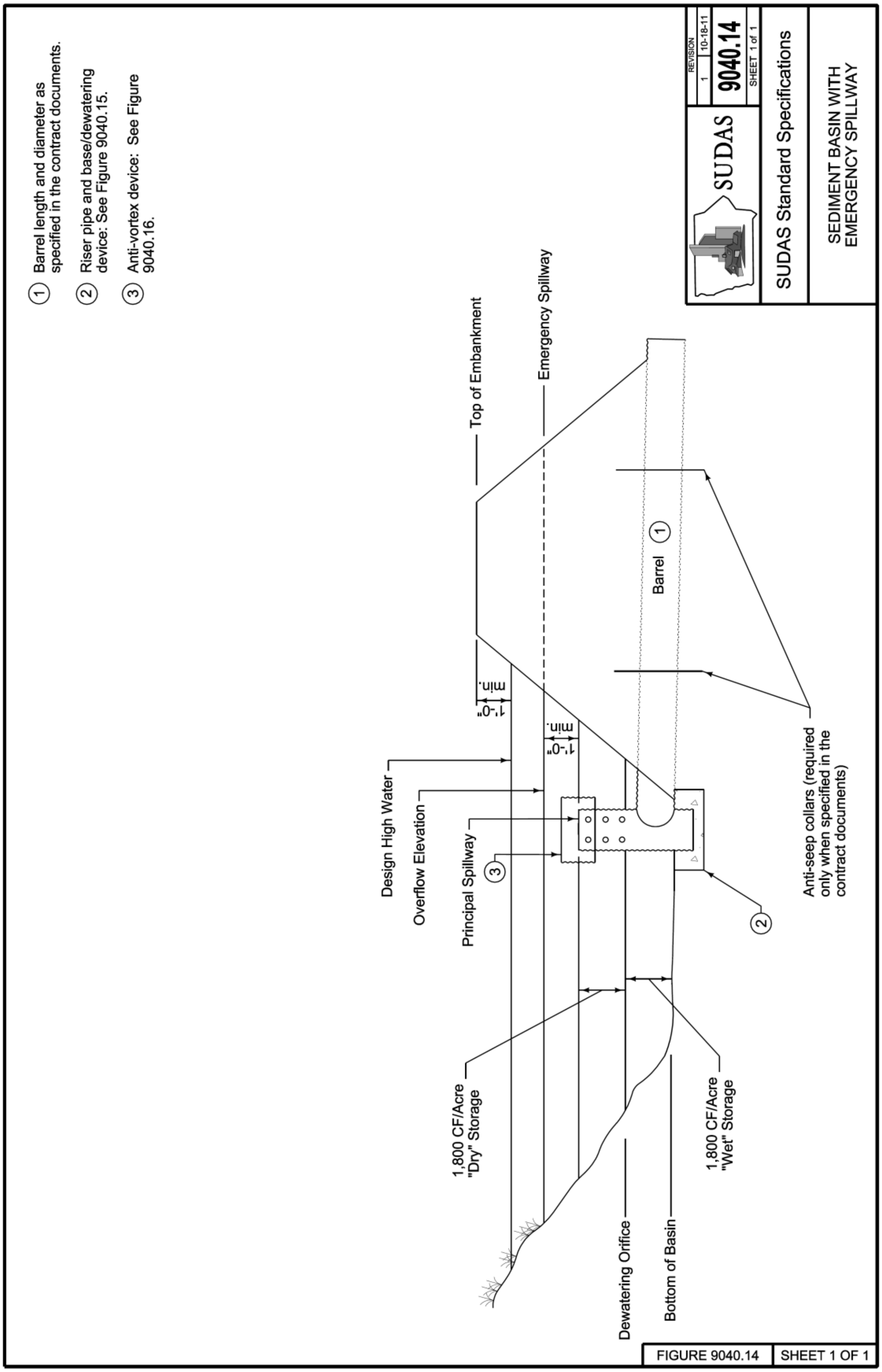


Figure 7E-12.02: Sediment Basin With Emergency Spillway
(SUDAS Specifications Figure 9040.14)



2. **Outlet Barrel:** The size of the outlet barrel is a function of its length and the total head acting on the barrel. This head is the difference in elevation of the centerline of the outlet of the barrel and maximum elevation of the water (design high water). The size of the outlet barrel can be determined using Chapter 2 - Stormwater for culvert design.

3. **Anti-vortex Device:** An anti-vortex device should be installed on top of the riser section to improve flow characteristics of water into the principal spillway, and prevent floating debris from blocking the spillway.

   There are numerous ways to provide protection for concrete pipe including various hoods, grates, and rebar configurations that are part of the project-specific design, and will frequently be part of a permanent structure.

   The design information provided in the following detail and table are for corrugated metal riser pipes.

Figure 7E-12.03: Example Anti-vortex Device
(SUDAS Specifications Figure 9040.16, sheet 1)

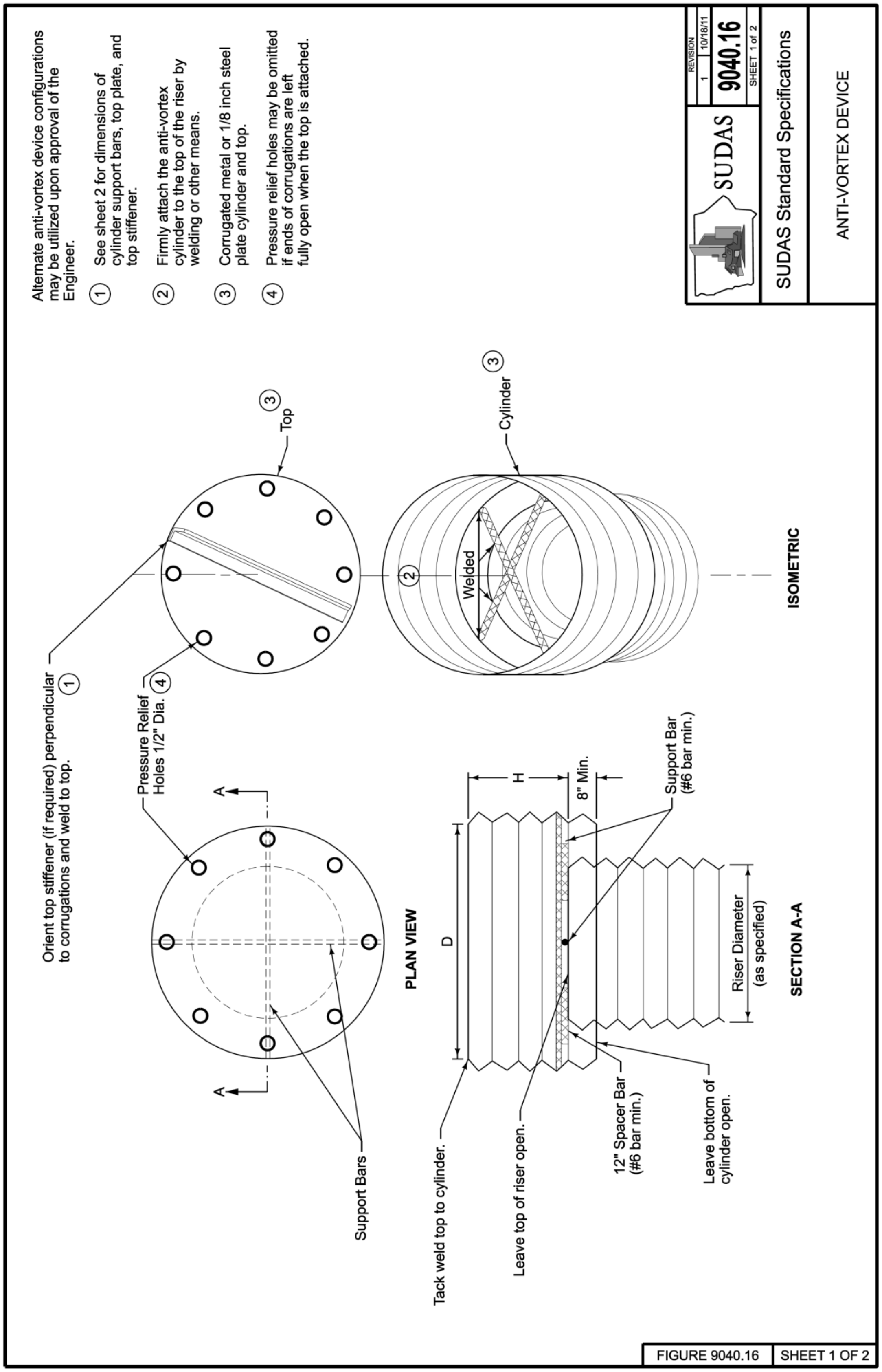


Table 7E-12.01: Design Information for Anti-vortex and Trash Rack Device
(SUDAS Specifications Figure 9040.16, sheet 2)

RISER	CYLINDER				MINIMUM TOP	
Diameter (in.)	Diameter (in.)	Thickness (gage)	Height (H) (in.)	Minimum Size Support Bar	Thickness	Stiffener
12	18	16	6	#6 rebar or 1 1/2" X 3/16" angle	16 ga F & C	----
15	21	16	7	#6 rebar or 1 1/2" X 3/16" angle	16 ga F & C	----
18	27	16	8	#6 rebar or 1 1/2" X 3/16" angle	16 ga F & C	----
21	30	16	11	#6 rebar or 1 1/2" X 3/16" angle	16 ga (C), 14 ga (F)	----
24	36	16	13	#6 rebar or 1 1/2" X 3/16" angle	16 ga (C), 14 ga (F)	----
27	42	16	15	#6 rebar or 1 1/2" X 3/16" angle	16 ga (C), 14 ga (F)	----
36	54	16	17	#8 rebar	14 ga (C), 12 ga (F)	----
42	60	16	19	#8 rebar	14 ga (C), 12 ga (F)	----
48	72	16	21	1 1/4" pipe or 1 1/4" X 1 1/4" X 1/4" angle	14 ga (C), 10 ga (F)	----
54	78	16	25	1 1/4" pipe or 1 1/4" X 1 1/4" X 1/4" angle	14 ga (C), 10 ga (F)	----
60	90	14	29	1 1/2" pipe or 1 1/2" X 1 1/2" X 1/4" angle	12 ga (C), 8 ga (F)	----
66	96	14	33	2" pipe or 2" X 2" X 1/4" angle	12 ga (C), 8 ga (F)	2" X 2" X 1/4" angle
72	102	14	36	2" pipe or 2" X 2" X 1/4" angle	12 ga (C), 8 ga (F)	2 1/2" X 2 1/2" X 1/4" angle
78	114	14	39	2 1/2" pipe or 2" X 2" X 1/4" angle	12 ga (C), 8 ga (F)	2 1/2" X 2 1/2" X 1/4" angle
84	120	12	42	2 1/2" pipe or 2" X 2" X 1/4" angle	12 ga (C), 8 ga (F)	2 1/2" X 2 1/2" X 5/16" angle

Notes:

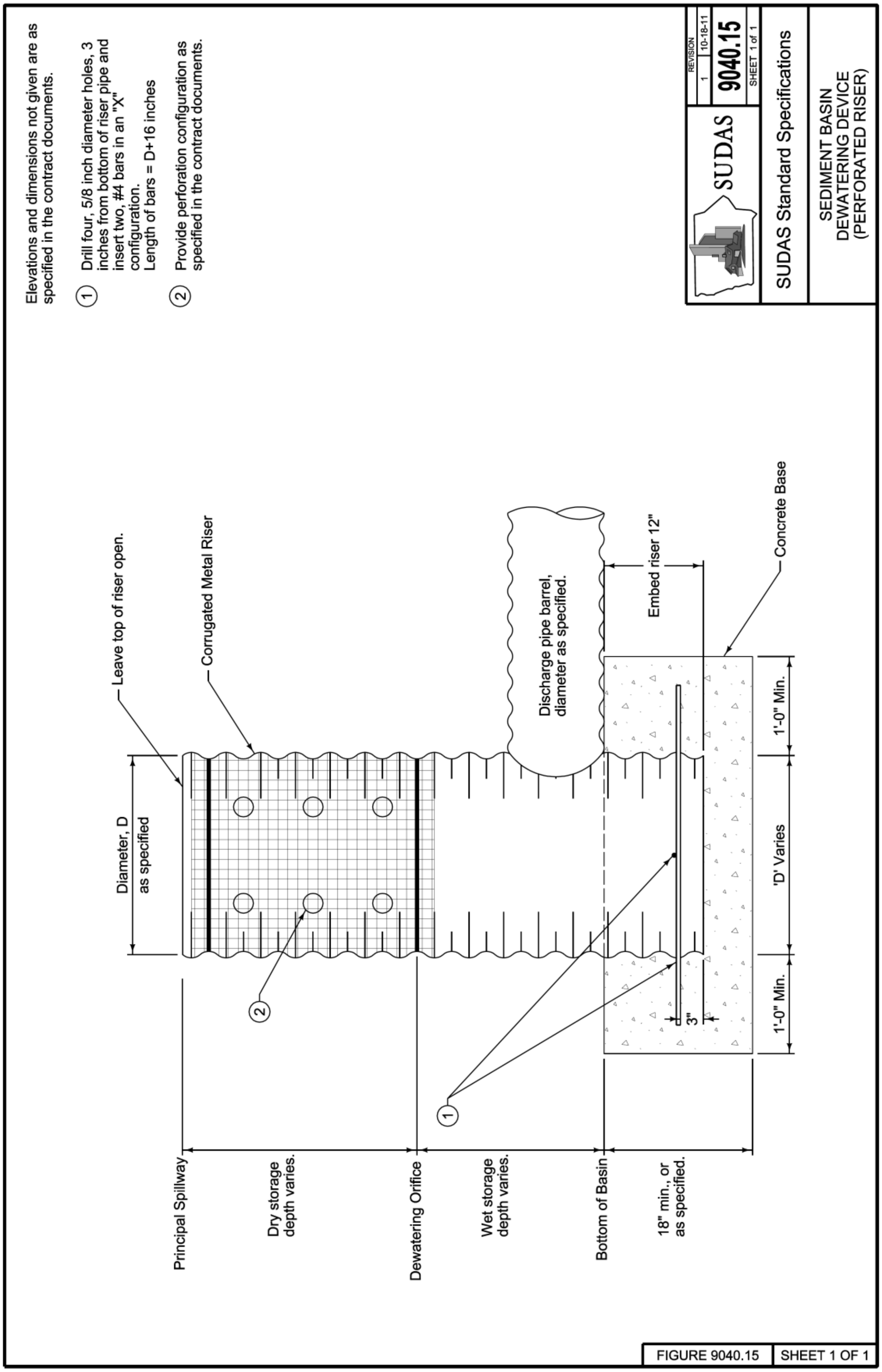
1. The criterion for sizing the cylinder is that the area between the inside of the cylinder and the outside of the riser is equal to or greater than the area inside the riser. Therefore, the above table is invalid for use with concrete pipe risers.
2. C - Corrugated F - Flat.

The riser pipe needs to be firmly attached to a base that has sufficient weight to prevent flotation of the riser. The weight of the base should be designed to be at least 1.25 times greater that the buoyant forces acting on the riser at the design high water elevation.

A base typically consists of a poured concrete footing with embedded anchors to attach to the riser pipe to anchor it in place.

4. **Dewatering Device:** The purpose of the dewatering device is to release the impounded runoff in the dry storage volume of the basin over an extended period of time. This slow dewatering process detains the heavily sediment-laden runoff in the basin for an extended time, allowing sediment to settle out. The dewatering device should be designed to drawdown the runoff in the basin from the crest of the riser to the wet pool elevation over a period of at least 6 hours.

Figure 7E-12.04: Theoretical Discharge Orifice for Design of Perforated Risers (SUDAS Specifications Figure 9040.15)



One common method of dewatering a sediment basin is to perforate the riser section to achieve the desired draw-down of the dry storage volume. Riser pipes with customized perforations to meet individual project requirements can be easily fabricated from a section of corrugated metal pipe. The contractor or supplier can drill holes of the size, quantity, and configuration specified on the plans. The lower row of perforations should be located at the permanent pool elevation (top of the wet storage volume). The upper row should be located a minimum of 3 inches from the top of the pipe (principal spillway elevation).

Dewatering device design begins by determining the average flow rate for a 6 hour drawdown time. Once the average discharge is known, the number and size of perforations required can be determined. To calculate the area of the perforations, a single rectangular orifice that extends from the wet pool elevation to the proposed elevation of the top row of holes (a minimum of 3 inches below the principal spillway) is assumed.

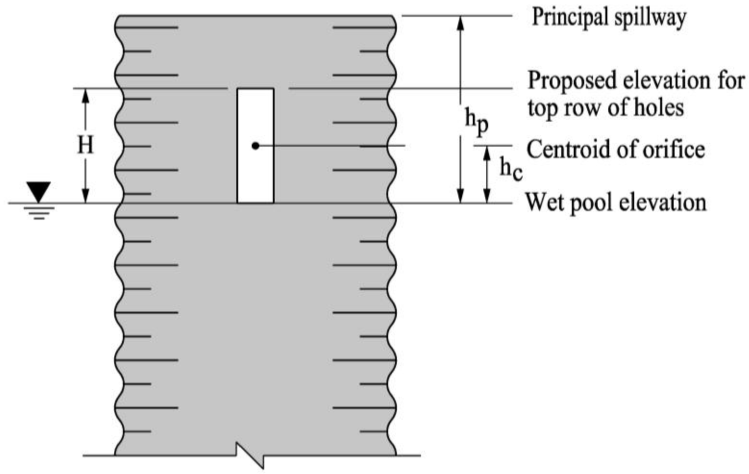
Next, the average head acting on the rectangular orifice as the basin is dewatered is determined. This average head is approximated by the following equation:

$$h_a = \frac{(h_P - h_c)}{2}$$ **Equation 7E-12.03**

Where:

h_a = Average head during dewatering
h_p = Maximum head (between the wet pool and principal spillway)
h_c = Distance between the wet pool elevation and the centroid of the orifice, ft

Figure 7E-12.05: Theoretical Discharge Orifice for Design of Perforated Risers



Once the average head is known, the area of the rectangular orifice is sized according to Equation 7E-12.04 to provide the average flow rate for the 6 hour drawdown. Providing evenly spaced perforations that have a combined open area equal to that of the calculated rectangular orifice, will provide the desired discharge rate for a 6 hour drawdown.

$$A = \frac{Q_a}{0.6 \times \left(2g \times h_a\right)^{1/2}}$$ **Equation 7E-12.04**

Where:

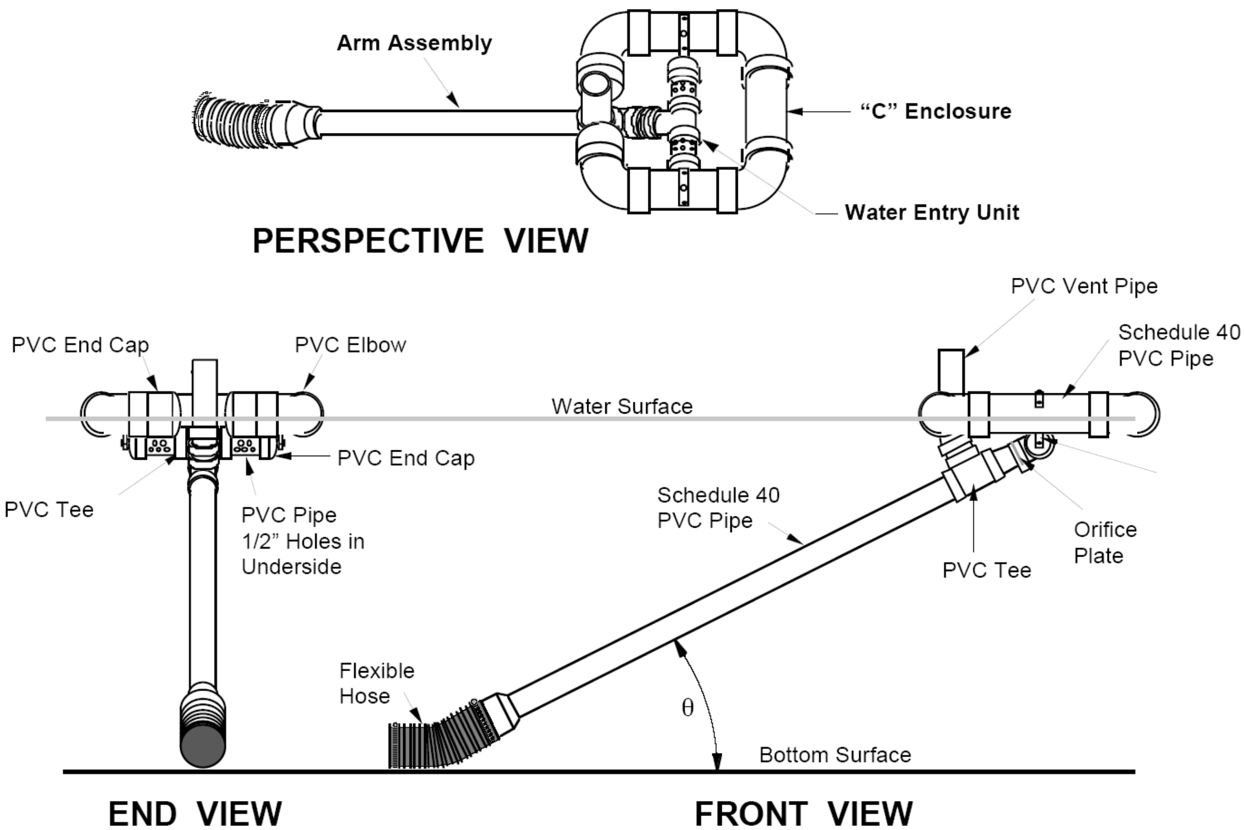
A = Total area of the orifices, sf
h_a = Average head acting on the orifice (Equation 7E-12.03)
Q_a = Average flow rate required for 6-hour drawdown, cfs
Q = S/21,600 sec. (6 hour drawdown only)
S = Dry storage volume required, cf

The number and diameter of the holes is variable. The diameter selected should be a minimum of 1 inch to minimize clogging, and should be a multiple of 1/4 of an inch. The perforation configuration should consist of a minimum of three horizontal rows and two vertical columns of evenly spaced perforations. Selecting a combination of hole diameter and number of holes is a trial and error process. Once the configuration is determined, the required information should be specified on the plans.

An alternative to the traditional riser is to provide a skimmer device that floats on the surface of the water in the basin. The skimmer is made of a straight section of PVC pipe equipped with a float and attached with a flexible coupling to an outlet at the base of the riser. Because the skimmer floats, it rises and falls with the level of the water in the basin and drains only the cleanest top layer of runoff. Sediment removal rates from basins equipped with skimmers have been shown to be significantly more effective than with a perforated riser or orifice.

Skimming devices are normally proprietary. Discharge information should be obtained from the manufacturer.

Figure 7E-12.06: Example Skimmer for Drawdown of Wet Storage



Source: Penn State University

5. **Emergency Spillway:** An emergency spillway acts as an overflow device for a sediment basin by safely passing the large, less frequent storms through the basin without damage to the embankment. It also acts in case of an emergency such as excessive sedimentation or damage to the riser that prevents flow through the principal spillway. The emergency spillway should consist of an open channel constructed adjacent to the embankment over undisturbed material, not fill. This channel should be stabilized with matting, seeding, or sodding.

   Where conditions will not allow the construction of an emergency spillway on undisturbed material, the spillway may be constructed on top of the embankment and protected with non-erodible material such as erosion stone.

   An evaluation of site and downstream conditions must be made to determine the feasibility of, and justification for, the incorporation of an emergency spillway. In some cases, the site topography does not allow a spillway to be constructed in undisturbed material, and the temporary nature of the facility may not warrant the cost of disturbing more acreage to construct and armor an emergency spillway. The principal spillway should then be sized to convey a 25 year storm event, providing 2 feet of freeboard between the design high water elevation and the top of the embankment. If the facility is designed to be permanent, the added expense of constructing and armoring an emergency spillway may be justified.

   When an emergency spillway is required, it should be designed to safely pass the 25 year design storm with a minimum of one-foot clearance between the high water elevation and the top of the basin embankment. Since the principal spillway is only designed to carry the 2 year event, the emergency spillway must carry the remainder of the 25 year event.

$$Q_e = Q_{25} - Q_p \qquad \textbf{Equation 7E-12.05}$$

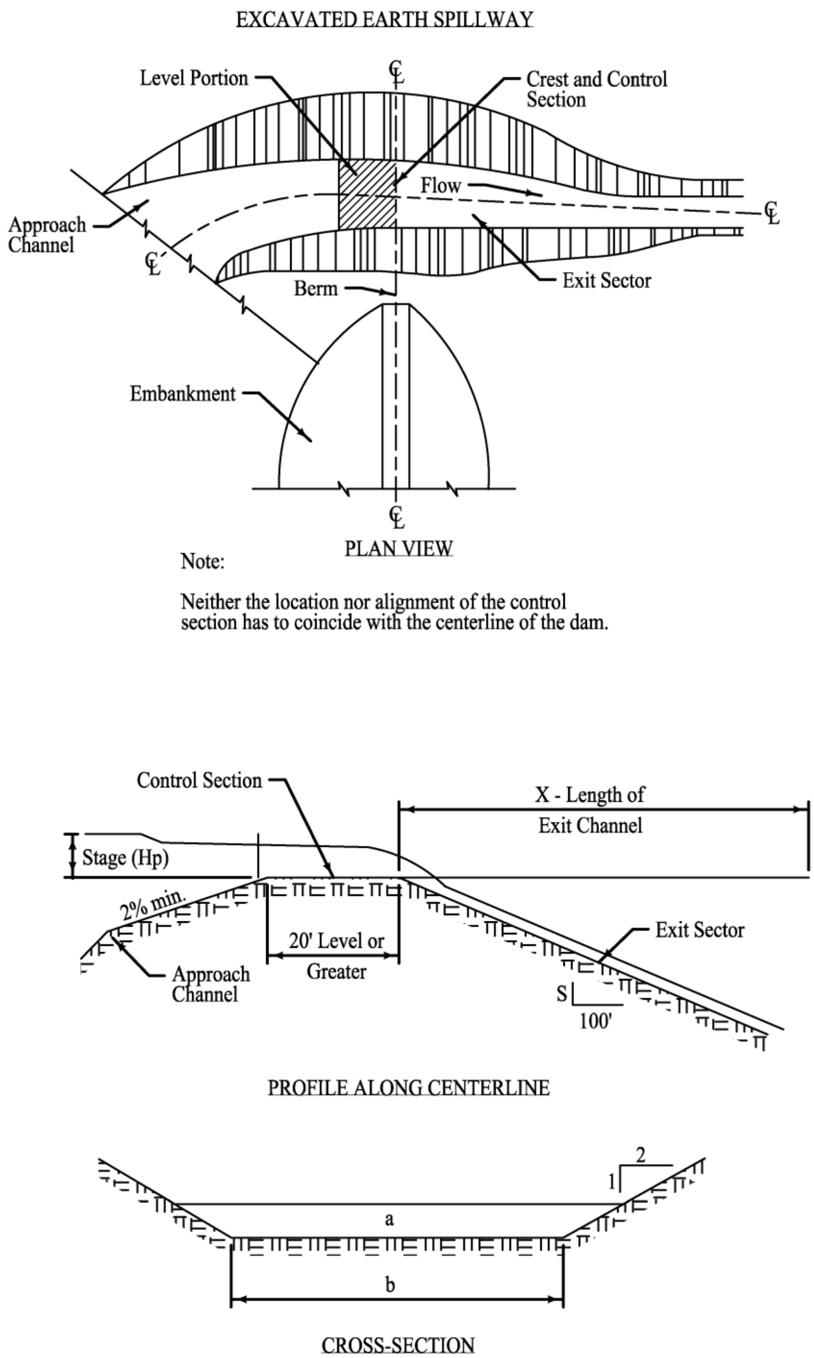
Where:

Q_e = Required emergency spillway capacity, cfs
Q_{25} = 25-year, 24 hour peak flow, cfs
Q_p = Principal spillway capacity at high water elevation, cfs

Based upon the flow requirements, Table 7E-12.01 can be used to determine the minimum width of the emergency spillway (b), the minimum slope of the exit channel (S), and the minimum length of the exist channel (X).

A control section at least 20 feet in length should be provided in order to determine the hydraulic characteristics of the spillway, according to Table 7E-12.01. The control section should be a level portion of the spillway channel at the highest elevation in the channel. If the length and slope of the exit channel indicated in Table 7E-12.01 cannot be provided, alternative methods of evaluating the spillway must be conducted.

Figure 7E-12.07: Typical Sediment Basin Emergency Spillway



Source: Roberts, 1995

Table 7E-12.02: Design Data for Earthen Emergency Spillways

Stage (H_p) in feet	Spillway Variables	Bottom Width (b) in feet																
		8	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**24**	**26**	**28**	**30**	**32**	**34**	**36**	**38**	**40**
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	2	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	4
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	61	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
	S	3.1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.5	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5	5	5	5	5	5	5	5	5	5	5	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	246	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	91	92	92	92	92	92	92	92	92	92	92

Data to the right of the heavy vertical lines should be used with caution, as the resulting sections will be either poorly proportioned, or have velocities in excess of 6 feet per second

Q=Spillway Capacity, cfs
V=Velocity, fps
S=Minimum downstream embankment slope, %
X=Minimum length of the exit channel, ft

Source: Roberts, 1995

In addition to checking the capacity of the spillway, the discharge velocity should also be considered. The allowable velocity for vegetated channels or channels lined with a turf reinforcement mat should be carefully analyzed. See Section 7E-23 and 7E-18 for information on permissible velocities. For non-erodible linings such as concrete or rip rap, design velocities may be increased.

5. **Anti-seep Collars:** Anti-seep collars help prevent water from flowing along the interface between the outlet barrel and the embankment. This movement of water can, over time, destabilize the embankment, causing it to wash out or burst.

   Anti-seep collars are not normally required for sediment basins. However, when the height of the embankment exceeds 10 feet, or the embankment material has a low silt-clay content, anti-seep collars should be used. Anti-seep collars should be used on all structures that may be converted to permanent features.

   The first step in designing anti-seep collars is to determine the length of the barrel within the saturated zone. The length of the saturated zone is determined with the following:

$$L_S = Y(Z+4)\left(1+\frac{S}{0.25-S}\right)$$ **Equation 7E-12.06**

   Where:

L_s	=	Length of the barrel within saturated zone, ft
Y	=	Depth of water at principal spillway crest, ft
Z		Slope of upstream face of embankment, Z ft H: 1 ft V.
S	=	Slope of the barrel in ft per ft

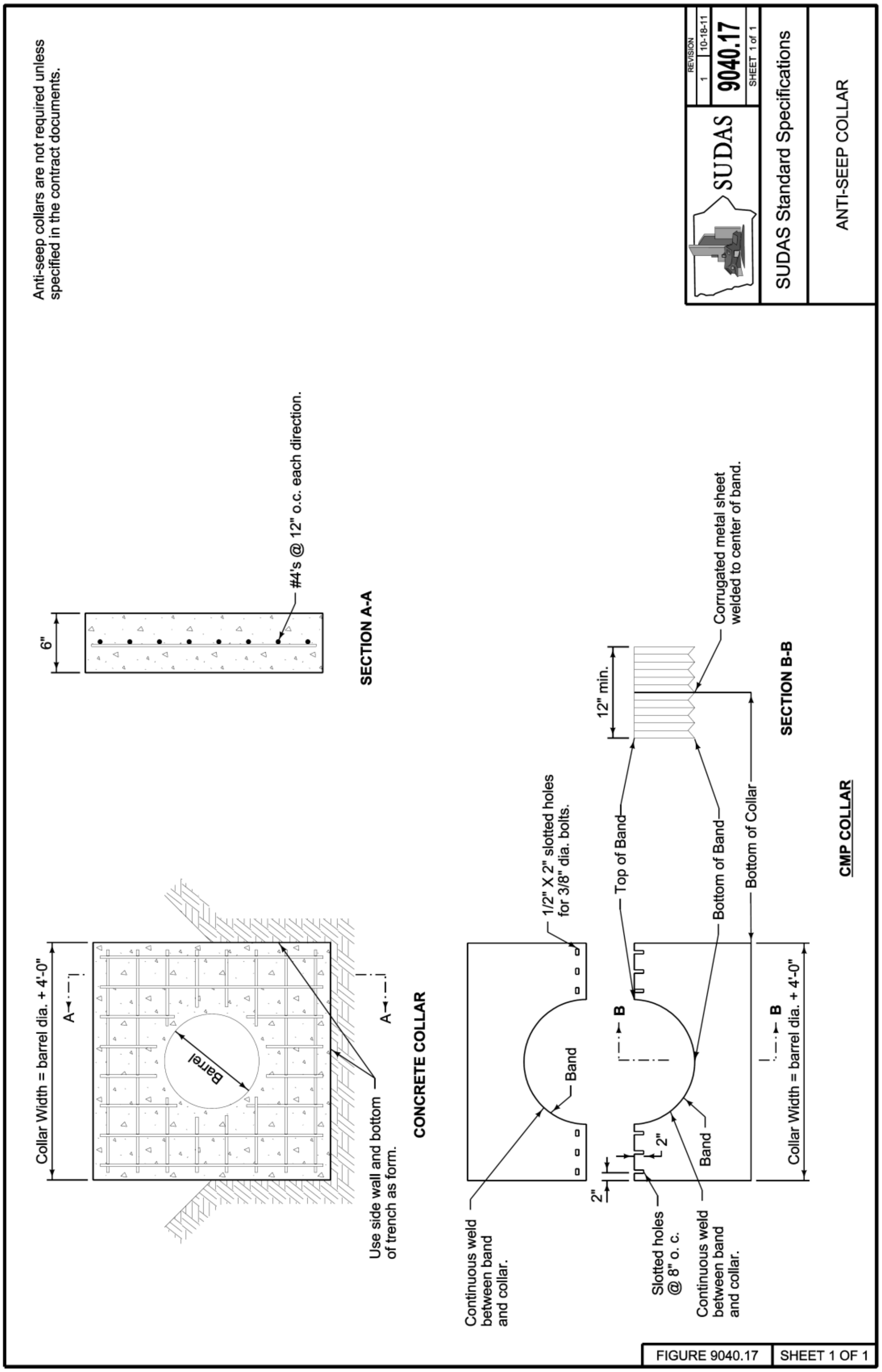
   An increase in the seepage length along the barrel of 10% should be provided. Determine the length required to achieve this by multiplying L_s by 10% ($0.10L_S$). This increase in length represents the total collar projection. This can be provided for by one or multiple collars.

   Choose a collar size that is at least 4 feet larger than the barrel diameter (2 feet in all directions). Calculate the collar projection by subtracting the pipe diameter from the collar size. Then determine the number of collars required by dividing the seepage length increase ($0.10L_S$) by the collar projection. To reduce the number of collars required, the collar size can be increased. Alternatively, providing more collars can decrease the collar size.

   Collars should be placed at a maximum spacing of 14 times the minimum projection above the pipe, and a minimum spacing of 5 times the minimum projection. All collars should be located within the saturated zone. If spacing will not allow this, at least one collar should be located within the saturated zone.

   Alternative methods of controlling seepage, such as a filter diaphragm may also be acceptable. A filter diaphragm consists of a layer of porous material running perpendicular to the outlet barrel which intercepts and controls water movement and fines migration within the embankment.

Figure 7E-12.08: Anti-seep Collar
(SUDAS Specifications Figure 9040.17)



Source: Adapted from Virginia DCR, 1999

7. **Safety Fence:** Depending on the depth, location, and local ordinances, a safety fence and appropriate signing may be required around the sediment basin.

8. **Additional Considerations:** Sediment basins which are more than 10 feet high, or which have storage capacities in excess of 10 acre-feet may require review and approval from the Iowa DNR per IAC 567-71.3. A vast majority of temporary sediment basins will not fall under these regulations. Basins that are intended to become permanent features are more likely to require this review.

C. Application

Sediment basin volumes and dimensions should be sized according to the criteria in Section 7D-1. Sediment basins are normally required for disturbed drainage areas of 10 acres or greater.

D. Maintenance

Maintenance and cleanout frequencies for sediment basins depend greatly on the amount of precipitation and sediment load arriving at the basin. During inspections, the embankment should be reviewed for signs of seepage, settlement, or slumping. These problems should be repaired immediately. Sediment should be removed from the basin when it accumulates to one-half of the wet storage volume.

During sediment cleanout, trash should be removed from the basin, and the dewatering device and riser pipe should be checked and cleared of any accumulated debris.

E. Design Example

Assume a construction site has 12 acres of disturbed ground which drains to a common location. In addition, 8 acres of off-site area drains through the construction site. Due to site restrictions, the 8 acres of off-site drainage cannot be routed around the site. Design a temporary sediment basin, with and emergency spillway, to handle and treat the runoff from the 20 acre site.

Solution:

1. **Basin Volume:** The Iowa DNR NPDES General Permit No. 2 requires a minimum storage volume of 3,600 cubic feet of storage per acre drained.

   Therefore: 20 acres x 3,600 cf = 72,000 cf.

   According to Section 7D-1, D, 3, this volume should be split equally between wet and dry storage (36,000 cf each).

   For the remaining calculations, assume that a basin has been sized and laid out to provide the following elevations:

   Elevation A (Bottom of Basin) = 100
   Elevation B (Wet Storage) = 103.0
   Elevation C (Dry Storage) 105.0
   Elevation D (Invert of emergency spillway) = 106.5
   Elevation E (Top of embankment) = 108.5

2. **Size the Principal Spillway (Riser):** From TR-55, using the methods described in Chapter 2 - Stormwater, assume the peak inflow from the 2 year, 24 hour storm is 41 cfs.

   In order to determine the required diameter of the principal spillway, the available head elevation above the spillway must be determined. From the elevation information provided above, the principal spillway is at elevation 105.0, and the invert of the emergency spillway is at elevation 106.5. Based upon this, the allowable head is 1.5 feet (106.5-105.0).

   The diameter of principal spillway (riser) is found by trial and error process, with the weir and orifice equations:

   Try a 24 inch diameter riser: (d=2 ft, A=3.14 ft^2)

   Weir Flow

   $$\mathrm{Q} = 10.5 \times \mathrm{d} \times \mathrm{h}^{\frac{3}{2}}$$

   $$Q = 10.5 \times 2 \times 1.5^{\frac{3}{2}} = 39 \text{ cfs}$$

   Orifice Flow

   $$\mathrm{Q} = 0.6 \times \mathrm{A} \times \sqrt{2\mathrm{gh}}$$

   $$Q = 0.6 \times 3.14 \times \sqrt{2 \times 32.2 \times 1.5} = 19 \text{ cfs}$$

   The lower flow rate (orifice) controls at 19 cfs. The design flow rate was 41 cfs; therefore, the proposed 24 inch riser is too small. Try a larger diameter.

   Try a 36 inch diameter riser. (d=3', A=7.1 ft^2)

   Weir Flow

   $$Q = 10.5 \times 3 \times 1.5^{\frac{3}{2}} = 58 \text{ cfs}$$

   Orifice Flow

   $$Q = 0.6 \times 7.1 \times \sqrt{2 \times 32.2 \times 1.5} = 42 \text{ cfs}$$

   The lower flow rate (orifice) controls at 42 cfs. This is greater than the design flow. Select a 36 inch diameter riser pipe for the principal spillway.

3. **Size the Dewatering Orifice:** To dewater 36,000 cubic feet, the average discharge, Q_a, is found as follows:

   $$Q_a = \frac{36{,}000}{6 \times 60 \times 60} = 1.7 \text{ cfs.}$$

   Next, determine the average head acting on the perforations during dewatering. Assume a rectangular orifice extends from the lowest set of perforations at the wet storage elevation (103.0), up to the upper row of perforations, 3 inches below the principal spillway (105.0-0.25 = 104.75). Based upon this, the maximum head, h_p, is 2 feet (105-103) and the distance to the centroid of the orifice is 0.875 feet [(104.75-103)/2].

   From Equation 7E-12.03, the average head acting on the openings is:

   $$h_a = \frac{(h_P - h_c)}{2} = \frac{(2 - 0.875)}{2} = 0.56 \text{ feet}$$

Once the average head and average discharge are known, the total orifice area can be calculated from Equation 7E-12.04:

$$A = \frac{Q_a}{0.6 \times \left(2g \times h_a\right)^{1/2}} = \frac{1.7}{0.6 \times \left(2 \times 32.2 \times 0.56\right)^{1/2}} = 0.47 \text{ sf}$$

Several perforation configurations could provide this area. One feasible selection would be to provide 18, 2 1/4 inch holes in three rows (6 holes per row).

4. **Size the Emergency Spillway:** Since this basin will have an emergency spillway, the principal spillway (riser) was only designed only to carry the 2 year storm. Larger storms, which exceed the capacity of the principal spillway, will be carried by the emergency spillway. The emergency spillway will be designed to carry the 25 year storm event.

From TR-55, using the methods described in Chapter 2 - Stormwater, assume the inflow from the 25 year storm is 99 cfs.

During high flow events, both the principal spillway and the emergency spillway will be bypassing flow from the basin. From step 2, the capacity of the principal spillway is 42 cfs. Therefore, from Equation 7E-12.05, the required capacity of the emergency spillway is as follows:

$$Q_e = Q_{25} - Q_p = 99 - 42 = 57 \text{ cfs}$$

The capacity of the emergency spillway must be at least 57 cfs. From the assumptions above, the difference in elevation between the invert of the emergency spillway, and the top of the embankment is 2 feet. Since a minimum of 1 foot must be provided between the design high-water elevation and the top of the embankment, 1 foot of head is available for discharge across the spillway.

From Table 7E-12.01, find the discharge (Q) that equals or exceeds the design value of 57 cfs. From the table, for 1-foot of head, move horizontally to the discharge value of 61 cfs. Moving vertically in the table, the corresponding width for a discharge of 61 cfs is 26 feet.

5. **Design Example Summary:**

Basin Volume: 72,000 cubic feet (split equally between wet and dry storage)

Principal Spillway Diameter: 36 inches

Dewatering Device: 18, 2 1/4 inch holes in 3 rows (6 holes per row)

Emergency Spillway Width: 26 feet

Sediment Traps

Source: Mississippi State University

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: A sediment trap is a temporary structure that is used to detain sediment-laden runoff from small drainage areas (less than 5 acres) long enough to allow sediment to settle out. These devices are constructed by excavating a temporary pond to a pre-determined shape and volume. A stone weir or spillway most commonly controls flow from the structure.

Typical Uses: Used to remove suspended soil particles prior to releasing runoff from a construction site. Normally located at the lowest point of a construction site.

Advantages:

- One of the most useful and cost-effective measures for treating sediment-laden runoff.
- Helps control overall stormwater runoff for small storms, thus protecting streams and rivers.
- Relatively easy and cost-effective to construct.

Limitations:

- May be large and require a substantial amount of site area.
- Sediment traps may need to be eliminated prior to final stabilization on high-density sites because the occupied area is planned for development. This may make it difficult to keep the sediment trap functioning during the entire construction phase.
- Sediment traps are fairly ineffective at removing fine silts or clay particles.
- Not designed to treat runoff during intense rainfall events, which can re-suspend sediment within the trap.

Longevity: 18 months

SUDAS Specifications: Refer to Section 9040, 2.12 and 3.17

A. Description/Uses

Sediment traps are temporary sediment control structures or ponds, having a simple outlet structure stabilized with engineering fabric and rip rap. They are typically installed in a drainage way or other point of discharge downstream from a disturbed area.

Sediment traps are one of the most reliable measures for treating sediment-laden runoff from small construction sites and may be considered the primary method of sediment removal for many sites.

Sediment traps are highly effective at treating runoff from disturbed sites up to 5 acres. For larger sites, multiple traps are recommended. For disturbed areas greater than 10 acres, a sediment basin may be required (see Section 7E-12).

B. Design Considerations

Sediment trap volumes and dimensions should be sized according to the criteria in Section 7D-1. A storage volume of 3,600 cf should be provided for every acre of disturbed ground. This storage volume should be divided equally between wet storage and dry storage.

Sediment traps should be constructed at a low point, or at the point where concentrated flows leave the site. The location should be reviewed to ensure that the trap can be easily accessed for cleanout and maintenance, and that a failure of the sediment trap will not cause a loss of life or property. Sediment traps are often constructed in ditches or swales by excavating a small area to create a depression.

Construction phasing must be considered when locating sediment traps. As construction progresses, the sediment trap may need to be removed in order to complete the proposed improvements. Select a location which will allow the sediment trap to remain in service as long as possible. If construction phasing does not allow a sediment trap to remain in service until final stabilization, the trap may need to be relocated.

The outlet for a sediment trap normally consists of a stone embankment, through which the runoff flows. The embankment slows the rate and velocity of the runoff, creating a temporary pond, which allows sediment to settle out. Equations for calculating the flow through a porous medium, which would allow for exact sizing of the outlet, are available. However, these equations require that the porosity of the stone be known. In addition, an adjustment would need to be made to account for clogging of the voids over time. These criteria are difficult to determine, therefore, it is recommended that the width of the embankment be based upon the drainage area as indicated in the following table:

Table 7E-13.01: Embankment Widths for Sediment Traps

Contributing Drainage Area (acre)	Embankment Width (feet)
1	4
2	6
3	8
4	10
5	12

Source: Roberts, 1995 (FHWA)

The stone embankment should be located at the low point of the basin. The bottom of the stone embankment should equal the elevation of the top of the wet storage portion of the trap. The stone embankment serves two purposes. The porous nature of the crushed stone allows water to seep through the embankment, providing a means to dewater the dry storage volume of the trap after each rainfall event. The top of the embankment serves as an overflow spillway to control the outlet of flows during large storm events.

Construction of the stone embankment should begin by placing a layer of engineering fabric down to protect the underlying soils and help prevent them from being washed away. Next, erosion stone, or a similarly-sized material, is placed over the filter fabric to create an embankment of the height and width required.

C. Application

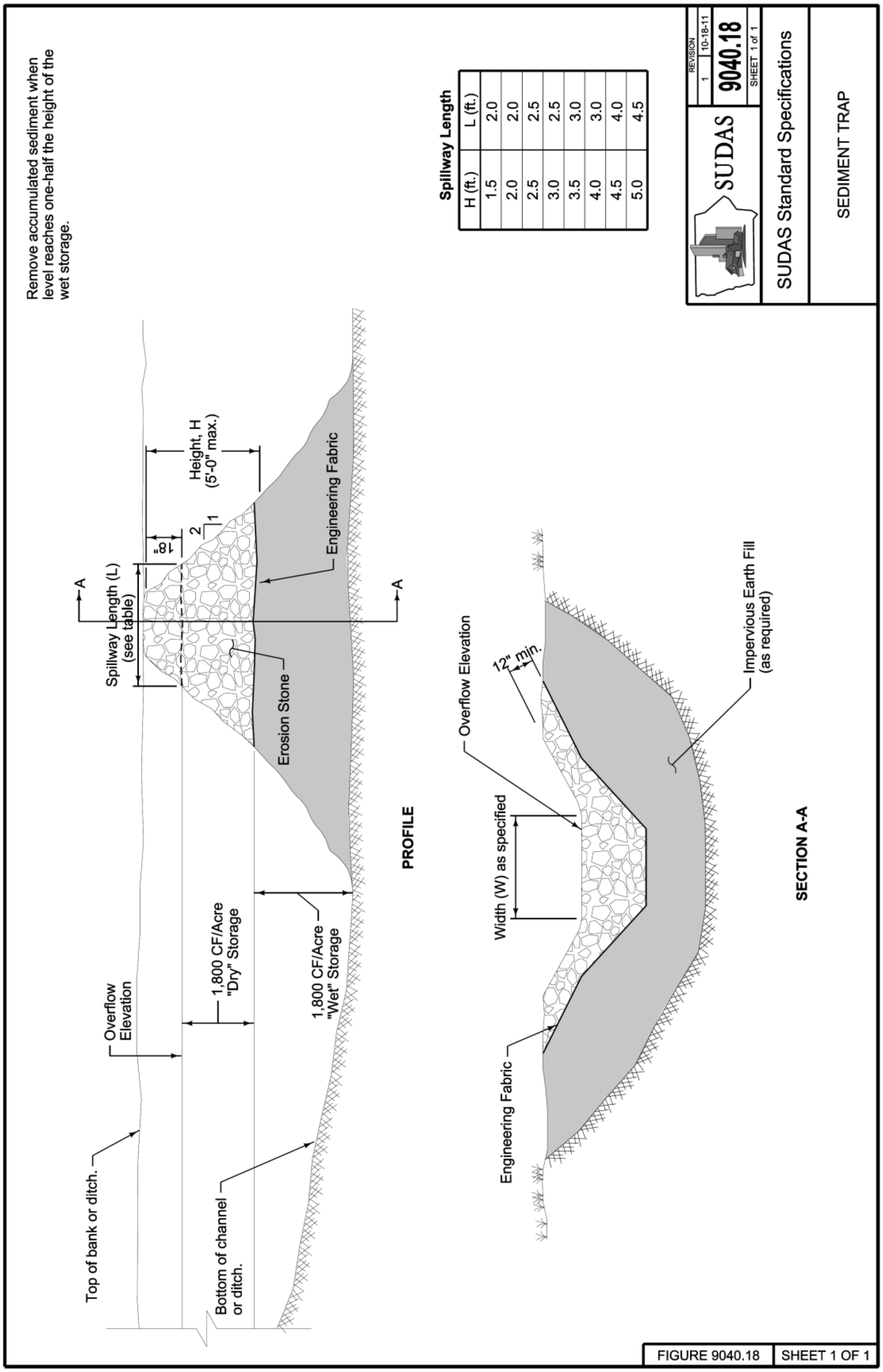
Sediment traps, in conjunction with other erosion control features, should be considered whenever more than 2 acres are disturbed. If more than 5 acres are disturbed, a sediment basin should be considered. If less than 2 acres are disturbed, sediment laden runoff may be controlled by other means such as silt fence or filtering products.

Sediment trap volumes and dimensions should be sized according to the criteria in Section 7D-1. 3,600 cf of storage should be provided for every acre of disturbed ground. This storage volume should be divided equally between wet storage and dry storage.

D. Maintenance

Sediment traps must be cleaned out as sediment accumulates within the trap. It is recommended to clean out the trap when it has lost one-half of the wet storage volume. Upon completion of the project, the trap area should be backfilled and stabilized. Alternatively, the trap may be converted to a permanent sediment basin or detention basin.

Figure 7E-13.01: Typical Sediment Trap with a Stone Outlet
(SUDAS Specifications Figure 9040.18)



Spillway Length

H (ft.)	L (ft.)
1.5	2.0
2.0	2.0
2.5	2.5
3.0	2.5
3.5	3.0
4.0	3.0
4.5	4.0
5.0	4.5

Silt Fences

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Silt fence is a temporary sediment barrier of geotextile fabric that is anchored into the ground and supported by posts on the downstream side of the fabric. Silt fences temporarily impound runoff and retain sediment onsite. They are most effective when designed to provide comprehensive water and sediment control throughout a construction site and if used in conjunction with erosion control practices.

Typical Uses: Used to control sheet flow runoff from disturbed land. May also be used to create a sediment trap for removal of suspended particles from low volume concentrated flows.

Advantages:
- Widely used BMP due to ease of installation and availability of materials.
- Relatively low cost.

Limitations:
- Ineffective against high flows.
- Must be removed after final stabilization.
- Could involve frequent maintenance related to removing accumulated silt behind the silt fence.

Longevity: Until sediment accumulates to one-half the height of the fence

SUDAS Specifications: Refer to Section 9040, 2.13 and 3.18

A. Description/Uses

Silt fence is a temporary barrier used to remove sediment from runoff. The fence works by intercepting sheet flow from slopes, causing the runoff to pond behind the fence, thereby promoting deposition of sediment on the uphill side of the fence.

Silt fence consists of a geotextile fabric that is trenched or sliced into the ground. The bottom of the fence is anchored into the ground by compacting the disturbed soil along both sides of the trench or slice. The top of the fence is attached to steel posts for support, creating a barrier to the flow of contaminated stormwater runoff.

Silt fence is one of the most commonly used sediment control practices. As such, it is often used improperly, or installed incorrectly. It should be placed at regular intervals on slopes to impound water. Silt fence can also be used in ditches and swales to create a small sediment containment system or ditch check. However, use as a ditch check should be limited to minor ditches and swales due to the potential for blow-out or undermining of the silt fence by high flows.

A common misconception among many designers is that the silt fence actually “filters” suspended particles from runoff. The effectiveness of silt fence is primarily derived from its ability to pond water behind the fence. This ponding action allows suspended particles to settle out on the uphill side of the fence. Particles are not removed by filtering the runoff through the fabric.

B. Design Considerations

1. **Overland Flow:**

   a. **General Guidelines:** Silt fence for sediment and slope control should be installed along the contour of the slope (i.e. the entire length should be at the same elevation). At each end of the silt fence a 20 foot segment should be turned uphill (“J”-hook) to prevent ponded water from flowing around the ends of the silt fence. Individual sections of silt fence should be limited to 200 foot lengths. This limits the impact if a failure occurs, and prevents large volumes of water from accumulating and flowing to one end of the installation, which may cause damage to the fence.

   b. **Sediment Control:** When used for sediment control, silt fence should be located to maximize the storage volume created behind the fence. Larger storage volumes increase the sediment removal efficiency of the silt fence, and decrease the required replacement/clean-out intervals.

      A common location to place silt fence for sediment control is at the toe of a slope. When used for this application, the silt fence should be located as far away from the toe of the slope as practical to ensure that a large storage volume is available for runoff and sediment.

   c. **Slope Control:** Silt fence can be installed on a slope to reduce the effective slope length and limit the velocity of runoff flowing down the slope. Silt fence also helps prevent concentrated flows from developing, which can cause rill and gully erosion. As a secondary benefit, silt fence installed on slopes can remove suspended sediment from runoff that results from any erosion that has occurred. For slopes that receive runoff from above, a silt fence should be placed at the top of the slope to control the velocity of the flow running onto the slope, and to spread the runoff out into sheet flow.

d. **Perimeter Control:** Silt fence is commonly used as a perimeter control along streets or adjacent to water bodies to prevent polluted water from leaving the site. When a diversion or perimeter control silt fence is installed in the direction of a slope, a 20 foot length of fence should be turned in, across the slope, at regular intervals (100 feet) to create a "J"-hook. These "J"-hooks act as check dams, controlling the velocity of the diverted runoff as it travels along the fence.

2. **Concentrated Flow:** For concentrated flows in swales or ditches, the silt fence is installed at right angles to the flow of water with the end posts turned uphill to prevent water from flowing around the edges. The 2 year discharge in the ditch should be checked to ensure that it does not exceed 1 cfs. For ditch or swale applications greater than 1 cfs, alternative methods of sediment removal and velocity control within the ditch, such as rock or manufactured ditch checks and sediment traps, are required.

3. **Diversion:** Silt fence can also be utilized as a synthetic diversion structure to redirect clean water around a site and intercept sediment-laden runoff and transport it to a sediment removal practice.

C. Application

For sediment control applications, the maximum contributing area should not exceed 1/4 acre per 100 feet of fence. If the contributing area exceeds this value, additional silt fence should be installed to break up the runoff into multiple storage areas.

When used as a velocity control measure for sheet flow on long slopes of disturbed ground, silt fence should be placed at the spacing interval stated in the table below:

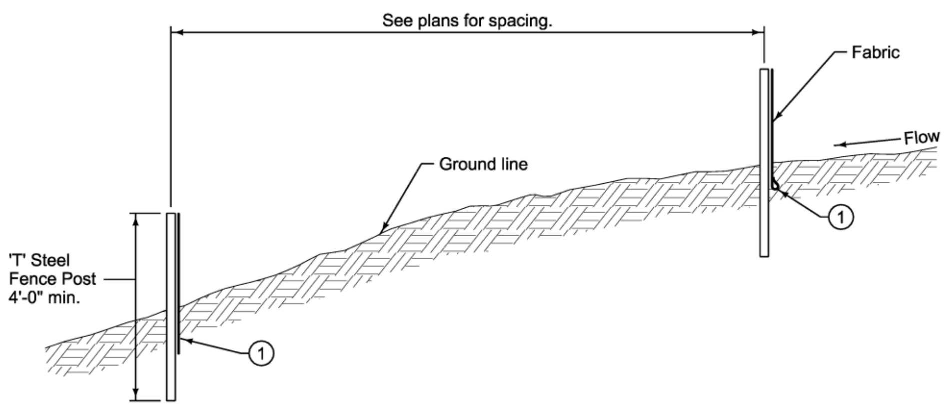
Table 7E-14.01: Silt Fence Spacing on Slopes

Slope	Placement Interval (feet)
≤ 10:1 (10%)	100
5:1 (20%)	60
4:1 (25%)	50
3:1 (33%)	40

When silt fence is used under concentrated flow, as a ditch check to intercept soil and debris from water flowing through ditches or swales, the following spacing guidelines should be used:

Figure 7E-14.01: Typical Ditch Check Spacing

Ditch Grade (%)	Spacing (feet)
1 to 2	150
2 to 4	75
4 to 6	40
> 6	25



D. Maintenance

When accumulated sediment reaches approximately one-half of the fence height, new silt fence should be installed, leaving the existing fence in place, and locating the new silt fence a sufficient distance away from it to provide area for sediment accumulation. When site conditions require that the silt fence be cleaned out, rather than replaced, extreme care must be taken to ensure that the silt fence is not damaged. Removed sediment should be spread out and stabilized. Any areas of damaged silt fence should be replaced immediately.

Upon project completion, fence fabric, posts, and accumulated sediment should be removed. Any areas disturbed by the removal of the silt fence or sediment should be stabilized.

E. Time of Year

Silt fences are effective on a year-round basis. Installation may not be possible when there is frost in the ground due to the requirement to trench or slice the fence below the ground surface.

Stabilized Construction Entrance

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control	■		
Runoff Reduction			
Flow Diversion			

Description: A stabilized construction entrance is a temporary, stabilized layer of large aggregate that is located at any point where traffic enters or leaves a construction site and enters a public road or other paved areas. Effectiveness depends on length, depth of rock, frequency of use, and maintenance of temporary rock entrance.

Typical Uses: Used where construction vehicles leave a construction site and enter onto a public street. The purpose of the rock entrance is to prevent mud from being tracked out onto the roadway, where it can cause plugging of storm sewers and fugitive dust problems.

Advantages:
- Low cost (based on stone availability) and easily installed.
- Helps prevent tracking of mud onto public streets, reducing fugitive dust and clogged storm sewers.
- Provides stable exit/entrance for construction traffic.

Limitations:
- Rock must be replaced once the voids become plugged with mud.
- May not remove all soil from vehicles, especially on muddy sites.
- Rock and sediment must be disposed of upon completion.

Longevity: Varies, based upon site conditions and volume of traffic

SUDAS Specifications: Refer to Section 9040, 2.14 and 3.19

A. Description/Uses

A stabilized construction entrance consists of a pad of large aggregate, often underlain with engineering fabric. Rock entrances should be located at any point where traffic will be leaving a construction site and entering a public roadway. The stabilized construction entrance reduces the amount of sediment (dust, mud, etc.) tracked offsite by construction equipment, especially if a wash-rack is incorporated for removing caked sediment.

B. Design Considerations

The entrance from a construction site is a significant source for offsite sediment deposition. Entrance and parking areas are continuously disturbed, leaving no opportunity for vegetation stabilization. During wet weather, these areas often become muddy, and construction vehicles track this mud off of the site and deposit it onto the public roadway where it clogs storm sewers and creates fugitive dust problems.

A stabilized construction entrance can reduce the amount of sediment that is tracked into the street by construction traffic. A rock entrance stabilizes the access to the site, and helps remove mud and clay from vehicle tires before they leave the site. A stabilized construction entrance should be constructed on every construction site, prior to the mobilization of construction equipment.

1. **Location:** A stabilized construction entrance should be located at every point where construction traffic leaves a construction site. Vehicles leaving the site should travel over the entire length of the rock entrance. When possible, the entrance should be located on level ground, at a location with appropriate sight distance. Construction vehicles should be prohibited from leaving the site at locations other than the stabilized construction entrance. Fence should be constructed if necessary. If additional access to the site is required, additional rock entrances should be constructed

2. **Site Preparation:** The area of the entrance should be excavated to the proposed thickness of the stone, stripping any topsoil, vegetation, and soft soils as necessary to provide a stable subgrade. When soft soil conditions exist, or when earthmoving or other heavy equipment will use the entrance, a subgrade stabilization fabric should be placed over the entire length and width of the entrance prior to placing the rock.

3. **Drainage:** Slopes should not exceed 15% and should be carefully graded to drain transversely to prevent runoff from the entrance from flowing into the street. All surface water flowing off of the construction entrance should be directed to a sediment removal device (sediment basin or trap, silt fence, filter sock, etc.).

4. **Tire Washing or "Wash-rack":** A properly constructed rock entrance should not be relied upon to remove all the mud from construction traffic. In some cases, the action of tires moving over a gravel pad may not adequately clean tires. If conditions on the site are such that the majority of the mud is not removed by the vehicles traveling over the rock, then the tires of the vehicles should be washed before entering the public road. Manual washing of the tires should be provided, or automated wash racks should be installed. Wash water must be carried away from the entrance to a sediment removal device (sediment basin or trap, silt fence, filter tube, etc.). All sediment shall be prevented from entering storm drains, ditches, or watercourses.

C. Application

1. **Length:** Minimum of 50 feet with an exception for single family residential lots which should be 30 feet. For sites that will be utilizing the entrance to haul a large volume of earth, the length of the entrance should be increased.

2. **Width:** Minimum of 20 feet wide. Busy entrances will need the capability of handling a lane of traffic each way, typically 30 feet wide. Flare the entrance where it meets the existing road to provide a turning radius.

3. **Geotextile:** If soft soil conditions exist, or when earthmoving or other heavy equipment will utilize the entrance, a layer of subgrade stabilization fabric should be placed over the prepared subgrade prior to placement of the rock to minimize migration of stone into the underlying soil by heavy vehicle loads. The barrier created by the fabric also aids in removal of the stone upon completion of the project, or as required for maintenance.

4. **Stone:** The rock for the entrance should consist of a nominal 2 to 3 inch clean crushed stone or recycled concrete. A 6 to 12-inch thick layer of stone, depending on anticipated traffic, should be placed over the entire length and width of the construction entrance. Rock with smaller aggregate does not adequately remove mud and clay from vehicles, and may be picked up by vehicle tires and carried out into the street.

D. Maintenance

Construction entrances should be inspected daily to ensure that mud and dirt are not being tracked onto roadways. All sediment deposited on paved roadways should be removed, not washed into the stormwater system or into waterways, at the end of each workday.

Rock entrances may require that additional stone be placed if the existing material becomes buried or if the subgrade is soft or becomes saturated.

Upon completion of the project the rock entrance, engineering fabric and any accumulated sediment should be removed and disposed.

Dust Control

Source: Jerico Services, Inc.

BENEFITS

	L	M	H
Flow Control			
Erosion Control	■	■	
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Dust control is the practice of controlling fugitive dust that results from grading, demolition, hauling, and traffic on construction sites. Fugitive dust may cause offsite damage, health hazards, and traffic problems if preventive measures are not taken.

Typical Uses: Used in open, windy areas such as the tops of hills and on construction sites with exposed soil in open areas. Also used in locations where construction traffic is high, such as the entrance to the site. Dust control may also be applied to soil stockpiles.

Advantages:
- Low visibility conditions caused by airborne dust are minimized.
- Dust control methods are widely applicable.
- Most dust control methods are inexpensive and promote the growth of stabilizing vegetation.
- Most dust control methods are easy to install/apply and maintain.

Limitations:
- Some temporary dust controls must be reapplied or replenished on a regular basis.
- Some controls are expensive (e.g., chemical treatment), may be ineffective under certain conditions, or have their own associated impacts.
- If chemical dust control treatment is over-applied, excess chemicals could potentially cause both surface and groundwater contamination.
- Petroleum products should not be used for dust control as there is potential for stormwater pollution and groundwater contamination.

Longevity: Usually short term; actual time varies by method and weather conditions

SUDAS Specifications: Refer to Section 9040, 2.15 and 3.20

A. Description/Uses

Earth-moving activities comprise the major source of construction dust emissions, but traffic and general disturbance of the soil also generate significant dust emissions. Therefore, dust control should be used when open dry areas of soil are anticipated on the site.

Dust control measures include minimization of soil disturbance, spray-on adhesives, tillage, chemical treatment and water spraying, and ensuring trucks are tarped upon leaving the construction site. In many cases, measures incorporated into the project to prevent soil erosion by water will indirectly prevent wind erosion.

While there are a number of temporary alternatives for dust control, one option is to permanently modify the site to eliminate dust generation. Modifications could include measures such as covering exposed areas with vegetation, mulch, stone or concrete. For the purpose of this standard, the focus is on temporary dust control measures.

B. Design Considerations

While several different products and practices are available for dust prevention, the most important tool is proper planning. During the design phase, the site should be analyzed for potential dust problems and the work coordinated to minimize dust problems.

The first step is to identify construction entrances and haul roads and provide a stable surface by paving, providing rock, or by chemical stabilization. Construction traffic on unstabilized haul roads should be limited as much as possible. When necessary, construction traffic on unstabilized ground should be limited to low speed operations (15 mph or less).

Existing vegetation or crop residue should be left in place as long as possible. When possible, existing tree lines should be left in place to act as a windbreak.

When dust problems are anticipated or are occurring during construction, there are a number of methods and products available to temporarily stabilize the surface and suppress the dust. Selection of these products or practices depends on several factors, including soil type, climate, and the necessary duration of treatment.

1. **Watering:** Spraying the surface of the ground with water is a readily available and highly effective method of suppressing dust, though very a short term one. Water trucks can provide onsite control of fugitive dust on haul roads and disturbed surfaces on an as-needed basis. The frequency of watering depends on several factors, including weather, soil type, and construction traffic. Water treatment is typically only effective for one-half hour to 12 hours. Water should be applied at a rate so that the soil surface is wet, but not saturated or muddy. If watering is to be employed at a construction site, it should be used in conjunction with a temporary gravel rock entrance, created to prevent mud from being spread on local streets.

2. **Tillage:** (See Section 7E-19 - Surface Roughening). Large, open, disturbed areas should be deep plowed to bring dirt clods to the surface. As the wind blows across smooth disturbed ground, the entire surface is exposed to the wind, creating a high potential for suspending dust particles. When the surface is roughened, only the peaks of the surface are exposed to the wind. In addition, the clods lying on top of the ground help stabilize the surface. This is a temporary emergency measure that can be used as soon as dust generation starts. Plowing should begin on the windward side of the site and leave 6-inch furrows, preferably perpendicular to the prevailing wind direction, to gain the greatest reduction in wind erosion. Tillage is only applicable to flat areas.

3. **Soil Stabilizers and Dust Suppressants:** These are chemicals applied on or mixed into the soil surface that maintain the moisture levels in exposed soils, or chemically bind the surface material to reduce fugitive dust emissions from the site. These products include:

    a. **Calcium Chloride:** Maintains water levels in the surface layer by absorbing humidity out of the air. May be applied by mechanical spreader as loose, dry granules or flakes, or as a liquid solution. Generally requires one or two treatments per season. Calcium chloride treated soils can inhibit the growth of vegetation and runoff from these areas can pollute water bodies. Therefore, calcium chloride should not be applied to large areas for site-wide dust control. When used, calcium chloride applications should be restricted to haul roads, and small areas.

    b. **Lignosulfonate:** Derived from wood pulp, lignosulfonate is a byproduct of the paper industry and is often referred to as “tree sap.” It is applied as a liquid to the ground surface, and binds the surface particles together. Generally requires one or two treatments per season.

    c. **Soybean Oil (Soapstock):** Acidulated soybean oil soapstock is a by-product of the refining process of soybean oil. It is applied as an undiluted liquid to the ground surface and binds the surface particles together. Proper storage and transportation of soybean oil require that the material be kept at a constant temperature of 155 degrees Fahrenheit and continuously agitated. Application of the material may require special pumping equipment. These restrictions may limit the use of soybean oil for dust control. Generally requires one treatment per season.

4. **Track-out Control:** (See Section 7E-15 - Stabilized Construction Entrance). Soil tracked out onto streets by construction vehicles eventually dries and creates a fugitive dust. A stabilized construction entrance should be provided to aid in removing soil from vehicles before they enter the roadway.

C. Application

Apply chemical controls at the manufacturer's specified rates and according to all federal, state, and local regulations governing their use. If a chemical dust control treatment is over-applied, excess chemicals could potentially cause both surface and groundwater contamination. Recommended application rates are listed in the table below. Chemical products must be stored, handled, and disposed of according to all applicable local, state, and federal regulations.

Table 7E-16.01: Recommended Application Rates for Dust Suppression Products

Product	Mixture	Application Rate[1]
Calcium chloride	Dry flake or liquid solution	1 lb/SY on anhydrous basis
Lignosulfanate	Diluted with water to 25% solids	1 gal/SY
Soybean oil (soapstock)	Undiluted	0.70 gal/SY

[1]Application rates are approximate and may need to be adjusted based upon site conditions

Source: Bolander, 1999 and Morgan, 2005

D. Maintenance

All dust control methods are temporary and require periodic maintenance. Wetting the ground surface with water may be necessary several times a day during hot and dry weather. Other methods provide longer effectiveness, and may only need to be applied once or twice per year.

Erosion Control Mulching

Source: Iowa DNR, 2005

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control	■	■	
Sediment Control	■		
Runoff Reduction	■		
Flow Diversion			

Description: Mulching is the application of organic material over soil that is bare or immediately over soil that has been seeded. Mulch prevents erosion by preventing the detachment of soil particles, slows runoff velocity, and retains moisture to improve germination and establishment of vegetative cover.

Typical Uses: This practice may be applied on exposed soils as a temporary control where soil grading or landscaping has taken place or in conjunction with temporary or permanent seeding. When time constraints prevent the establishment of vegetation (seeding), mulch such as wood chips, straw, or compost can be used independently as a temporary soil stabilization practice that protects the soil surface until vegetation establishment can be completed.

Advantages:
- Provides immediate surface protection.
- Suppresses weed growth.
- Conserves soil moisture.
- Acts as a thermal layer for seed.
- If used in conjunction with seed, allows seed growth through the mulch.
- Useful for dust control.

Limitations:
- If applied too thick, it may inhibit seed germination.
- Can blow or wash away if not anchored properly.

Longevity: Varies by material (three months to one year)

SUDAS Specifications: Refer to Section 9040, 2.16 and 3.21

A. Description/Uses

Used alone or applied over seed, mulch provides immediate erosion protection. Mulching without seeding may be considered for very short-term protection. Mulch protects the disturbed soil surface by absorbing the impact of raindrops, thereby preventing detachment of the soil particles. It also retains and absorbs water, slowing runoff. These properties allow for greater infiltration of water into soil; help to retain seeds, fertilizer and lime in place; and improve soil moisture and temperature conditions for seed germination. Mulch is essential in establishing good stands of grasses and legumes. In order to prevent movement by wind or water, it is important that the mulch be anchored to the soil.

B. Design Considerations

The plans and specifications should address the type of mulch used, application rate, timing of the application, method of anchoring, and schedule for installation, inspection, and maintenance.

1. **Site Preparation:** The soil surface shall be prepared prior to the application of mulch in order to achieve the desired purpose and to ensure optimum contact between soil and mulch.

2. **Material Considerations:**

   a. **General:**
      1) Mulching should not be performed during periods of excessively high winds that would preclude the proper placement of mulch.
      2) Concentrated flows should be diverted around areas where mulch is applied.
      3) If ground is seeded, mulching should be completed during or immediately after seeding.
      4) Depending on the seeding period, a heavier application of mulch may be needed to prevent seedlings from being damaged by frost.
      5) In areas where lawn-type turf will be established, the use of tackifiers is the preferred anchoring method. Crimping tends to leave an uneven surface and netting can become displaced and entangled in mowing equipment.
      6) The use of mulch behind curb and gutter may not be desirable unless anchored by netting, because air turbulence from nearby traffic can displace mulch. Consider the use of erosion mat or sod as an alternative.
      7) The product longevity should match the length of time the soil will remain bare or until vegetation occurs.

   b. **Straw:**
      1) Straw mulch should be applied in conjunction with temporary or permanent seeding, except when applied for short-term (less than three months) stabilization prior to the allowable seeding date.
      2) To prevent straw from being wind blown, it is anchored to the soil surface using tackifiers, nets, or a mulch-crimping machine. Mechanical anchoring or crimping is recommended only for slopes flatter than 2:1. Mulch on slopes steeper than 2:1 should be anchored to the soil with netting, or other alternatives, such as a rolled erosion control product considered.
      3) Only use straw free from all noxious weeds, seed bearing stalks, or roots
      4) Expected longevity is less than three months.

c. Wood Chips/Grindings:
1) Do not use wood chips/grindings over newly seeded areas.
2) Chips may be produced from vegetation removed from site.
3) Chips are effective on slopes up to 3:1.
4) Wood chips decompose over an extended period of time. This process may take nitrogen from the soil. To prevent nitrogen deficiency in the soil, the wood mulch should be treated with a nitrogen rich fertilizer.
5) Do not use in areas where fine turf will be established.
6) Expected longevity is less than 12 months.

d. Hydromulch: Hydromulching is normally conducted in conjunction with hydroseeding, but can also be applied as a stand-alone practice. Several different types of hydromulch are available, and each has different material properties and typical uses:

1) Wood Cellulose Fiber Hydromulch:
a) Produced from wood pulp and recycled paper
b) Most commonly used hydromulch
c) Use is limited to slopes 6:1 or flatter.
d) Typically require 24 hours to dry before rainfall occurs in order to be effective against erosion.
e) Expected longevity is 3 to 12 months.

2) Bonded Fiber Matrix (BFM) Hydromulch:
a) Produced from strands of elongated wood fibers and a binding agent
b) May be used on slopes up to and including 2:1.
c) Typically requires 24 hours to dry before rainfall occurs in order to be effective against erosion.
d) Expected longevity is 3 to 12 months.
e) Provides significantly superior erosion protection than straw mulch or wood cellulose hydromulch.

3) Mechanically Bonded Fiber Matrix (MBFM) Hydromulch:
a) Produced from strands of elongated wood fibers and crimped synthetic fibers to create an interlocking mechanism between the fibers. Material is combined with additional binding agents.
b) May be used on slopes up to and including 2:1.
c) Provides immediate protections against erosion. No cure time is required to develop surface protection.
d) Expected longevity is 12 months or greater.
e) Provides significantly superior erosion protection than straw mulch or wood cellulose hydromulch.

e. Compost:
1) Compost may be used as mulch, either with or without seeding for erosion protection. See Section 7E-2 - Compost Blanket.
2) Expected longevity is less than 12 months.

C. Application

1. **Mulching without Seeding:** Wood mulch and compost applied without seed, should be applied to a uniform depth of 1 to 3 inches depending on slope. Straw mulch should be applied at a rate of 2 tons per acre to achieve the specified coverage rate. Wood cellulose fiber hydromulch should be applied at a rate of 2,600 pounds per acre. BFM and MBFM hydromulch should be applied at a rate of 3,600 pounds per acre.

2. **Mulching for Seeding:** Straw mulch over newly seeded areas should be applied at a rate of 1 1/2 tons per acre. This application provides some protection of the surface, while allowing some sunlight to penetrate and air to circulate thereby promoting seed germination. When compost is used as mulch over newly seeded areas, a minimum thickness of 1 inch should be spread evenly over flat surfaces. For compost used as mulch on slopes, see compost blankets in Section 7E-2. Hydromulch products applied with seeding (hydroseeding) are applied at the same rate as without seeding (see paragraph above).

The NPDES General Permit No. 2 requires that all disturbed areas where no construction activities are scheduled for a period of 21 calendar days or more, be stabilized within 14 days of the final construction activity. Mulching is one way to meet this requirement.

D. Maintenance

Inspect mulched areas for signs of thin or bare spots. Add mulch as required to maintain the thickness of the cover. Areas that show signs of erosion should be repaired, and may require additional protection with an erosion control blanket or other method.

E. Time of Year

Mulch applications for establishing vegetation should be done when weather and soil conditions are favorable. Mulch can be applied over bare frozen ground that has not been seeded to help prevent erosion until such time as vegetation can be established.

Turf Reinforcement Mats (TRM)

Source: SI Geosolutions

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control	■	■	■
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Turf reinforcement mats (TRMs) are composed of non-degradable synthetic fibers, filaments, nets, wire meshes, and/or other elements, processed into a permanent, three dimensional matrix. TRMs are designed to impart immediate erosion protection, enhance vegetation establishment, and permanently reinforce vegetation during and after maturation.

Typical Uses: TRMs are typically used on steep slopes and in hydraulic applications such as high flow ditches and channels, stream banks, shorelines, and inlet/outlet structures. TRMs are used where erosive forces may exceed the limits of natural, unreinforced, vegetation, or in areas where limited vegetation establishment is anticipated.

Advantages:
- Can withstand high hydraulic shear stresses and velocities.
- Provides permanent, long-term reinforcement of vegetation. Does not degrade over time like RECPs.
- Ability to be vegetated creates a more aesthetically pleasing appearance than rip rap, concrete, or other “hard armor” techniques.
- Can stabilize ground where vegetation is difficult to establish.
- Normally a less expensive alternative to “hard armor” techniques.

Limitations:
- Performance is dependent upon proper product selection and installation
- Can only withstand a limited amount of flow before hard armoring is required.

Longevity: Permanent

SUDAS Specifications: Refer to Section 9040, 2.17 and 3.22

A. Description/Uses

Turf reinforcement mats (TRMs) are a three dimensional product, constructed of synthetic, non-degradable (though some products are a composite of degradable and non-degradable materials) materials. The non-degradable matting creates a permanent reinforcing system for vegetation. The resulting reinforced vegetation is able to withstand significantly greater erosive forces than normal vegetation.

Traditionally, hard-armor erosion control techniques, such as rip rap and reinforced paving systems, have been employed to prevent soil erosion in highly erosive areas. Although these permanent measures can withstand substantial hydraulic forces, they are costly, and they do not provide the pollutant removal capabilities of vegetated systems.

TRMs enhance the natural ability of vegetation to permanently protect soil from erosion. In addition to providing scour protection, TRMs are designed to encourage vegetative root and stem development. By protecting the soil from scouring forces and enhancing vegetative growth, TRMs can raise the threshold of natural vegetation to withstand higher hydraulic forces on slopes, in streambanks and channels, and at inlet/outlet structures.

TRMs, unlike temporary erosion control products, are designed to remain in place permanently to protect seeds and soils, improve germination, and reinforce established vegetation. Some TRMs incorporate natural, degradable fiber material to assist in the initial establishment of vegetation; however, the permanent reinforcement structure of TRMs is composed entirely of non-degradable materials.

In addition to providing permanent reinforcement of vegetation, TRMs also protect disturbed surfaces immediately after installation (prior to establishment of vegetation). This benefit is important for preventing soil loss and protecting newly seeded areas.

B. Design Considerations

TRMs are produced by a number of manufacturers, and are available in a wide variety of configurations. The following steps should be considered when designing and specifying an appropriate TRM.

1. **Hydraulic Stresses:** TRMs in channels should be designed based upon the calculated shear stress. The shear stress imposed on the TRM in the channel should be evaluated under two conditions: temporary (unvegetated) and permanent (vegetated). The temporary condition represents the unvegetated conditions immediately after installation of the TRM. The permanent condition represents the long-term protection provided by the TRM in its fully vegetated state.

   A TRM in a permanent vegetated state should be designed to withstand a 10 year storm event. In a fully vegetated channel, the TRM is located well below the top of the exposed vegetation. As a result, it has little impact on the level of shear created by the flow and its presence can be ignored. In doing this, the shear stress in the fully vegetated channel is determined in the same manner as described in Section 7E-23 - Grass Channel.

   The TRM should also be analyzed for an unvegetated state. Since this condition is temporary, the unvegetated TRM can be evaluated for a 2 year storm (rather than the 10 year). This analysis also follows the method described in Section 7E-23 - Grass Channel, but since there is no vegetation, the Manning coefficient is constant. The Manning coefficient of a TRM is normally provided in the manufacturer's literature.

Many TRM manufacturers have software available to aid in the calculation of shear stress. This software may be available through the manufacturer's website or local product representative.

Once the anticipated shear stresses are known, a TRM can be selected. Most TRM manufacturers report the permissible shear stresses that their products can withstand in both the vegetated and unvegetated conditions. These values are typically determined from full-scale, third party hydraulic flume testing. Commonly accepted facilities for conducting these tests include the Texas Transportation Institute (TTI), Colorado State University, and Utah State University. The designer should select a product with a greater permissible shear stress than the actual calculated hydraulic shear stress of the system. Note: for TRMs containing degradable components, the reported permissible values must represent only the permanent, synthetic portions of the TRM to satisfy the long-term design and performance requirements.

2. **Non-hydraulic Stresses:** In addition to the hydraulic stress (shear), consideration must also be given to non-hydraulic stresses. Examples of non-hydraulic stresses include heavy mowing equipment, occasional vehicular traffic, and heavy debris in the channel, or on a slope.

   The materials that most TRMs are constructed from are not intended to withstand these non-hydraulic stresses. This type of loading can cause the material to tear, creating the potential for failure of the entire system.

   For installations that will be exposed to these types of stresses, a high tensile strength material should be specified. These high tensile strength materials are commonly called high survivability or high performance TRMs. These high strength TRMs will provide long-term structural integrity, even when exposed to potentially damaging non-hydraulic stresses.

Table 7E-18.01: TRM Material Requirements and Acceptable Applications

	Property[1]	**Test Method**	**Type 1**	**Type 2**	**Type 3**	**Type 4**
Material	Thickness	ASTM D 6525	0.25 in	0.25 in	0.25 in	0.25 in
	Tensile Strength[2]	ASTM D 6818	125 lb/ft	240 lb/ft	750 lb/ft	3,000
	UV Resistance[3]	ASTM D 4355	80% @ 500 hrs	80% @ 1,000 hrs	80% @ 1,000 hrs	90% @ 3,000 hrs
Performance	Maximum Shear Stress[4] (Channel Applications)	ASTM D 6460	7 lb/ft^2	10 lb/ft^2	12 lb/ft^2	15 lb/ft^2
	Maximum Slope Gradient (Slope Applications)	N/A	1:1 (H:V) or flatter	1:1 (H:V) or flatter	1:1 (H:V) or greater	1:1 (H:V) or greater

1 For TRMs containing degradable components, all values must be obtained on the non-degradable portion of the matting.
2 Minimum Average Roll Values, machine direction only.
3 Tensile strength of structural components retained after UV exposure.
4 Minimum shear stress that fully-vegetated TRM can sustain without physical damage or excess erosion (0.5 in soil loss) during a 30 minute flow event in large scale testing. Acceptable large scale testing protocol includes ASTM D 6460 or independent testing conducted by the Texas Transportation Institute, Colorado State University, Utah State University, or other approved testing facility. Bench scale testing is not acceptable.

C. Application

Turf reinforcement mats should be selected and used in locations where vegetation alone cannot withstand the anticipated flow velocities and shear stresses, and where hard armor (concrete and rip rap) is not necessary or is visually unappealing, or where stormwater quality and sediment/pollutant removal is desirable.

D. Maintenance

Once installed, there is little maintenance that needs to be done to TRMs. If the TRM is to be vegetated, the vegetation should be watered as needed (refer to Section 7E-24 - Permanent Seeding). Until the vegetation is fully established, the ground surface should be inspected for signs of rill or gully erosion below the matting. Any signs of erosion, tearing of the matting, or areas where the matting is no longer anchored firmly to the ground should be repaired.

E. Design Example

Assume a channel with a 4 foot bottom, 3:1 side slopes, and a slope of 3% is designed to carry 265 cfs. Lining the channel with a TRM is being proposed. Determine if a selected vegetated TRM, with Class C vegetation is adequate. Also analyze the TRM for the unvegetated condition to ensure that it will provide sufficient protection until vegetation is established. The manufacturer of the TRMs provided the following information on the TRM's properties:

Permissible shear stress, vegetated - 8 lbs/ft^2
Permissible shear stress, unvegetated - 4.55 lbs/ft^2
Manning's n Coefficient - 0.026

Solution:

First determine the shear stress for the vegetated condition. Using Manning's equation, find the depth of flow. This can be done through a trial and error process, or by using various tables and charts.

Trial 1 - Assume a depth of 2 feet.

Area of Flow, $A = (b + Z \times d) \times d = (4 + 3 \times 2) \times 2 = 20 \text{ ft}^2$

Wetted Perimeter, $P = b + 2 \times \sqrt{d^2 + (Zd)^2} = 4 + 2 \times \sqrt{2^2 + (2 \times 3)^2} = 16.6 \text{ ft}$

Hydraulic Radius, $R = A/P = 20/16.6 = 1.2$

Manning coefficient (from Equation 7E-23.01, Section 7E-23 - Grass Channel):

$$n = \frac{0.1.2^{1/6}}{19.97\left(\log\left(44.8 \times 0.66^{0.6} \times 1.2^{-0.4}\right) + \log\left(0.1.2^{1.4} \times .03^{0.4}\right)\right)} = 0.051$$

Solving Manning's yields: $Q = {}^{1.49}\!/_{n}\, AR^{2/3} S^{1/2} = \left({}^{1.49}\!/_{0.051}\right)(20)(1.2)^{2/3}(0.03)^{1/2} = 114\text{cfs}$

Since 114 cfs is less than the design value of 265 cfs, a larger depth should be assumed.

Trial 2 - Assume a depth of 3 feet.

Following the procedure for Trial 1 – A = 39; P=23; R=1.7; n=0.045; Q= 318 cfs. Q is too large.

Trial 3 - Assume a depth of 2.8 feet

A=34.7; P=21.7; R=1.6; n=.046; Q=266 – Say 265 cfs OK.

Now that the depth is known for the vegetated condition, the shear stress can be determined by Equation 7E-23.02 from Section 7E-23 - Grass Channel.

$$\tau_{max} = \gamma \times d \times S = 62.4 \times 2.8 \times 0.03 = 5.2 \text{ lbs/ft}^2$$

Since the maximum shear stress of 5.2 lbs/ft^2 is less than the capacity of the vegetated TRM (8.0 lbs/ft^2), the design is acceptable.

Now analyze for the unvegetated condition to ensure that an adequate level of protection will be provided until the vegetation is established.

Following the same procedure as the previous example, the channel properties are calculated. Note that for the unvegetated condition, a constant Manning coefficient (in this case 0.026) can be assumed.

Assuming a depth of 2.1
A=21.6; P=17.3; R=1.25; Q=249. Q is too low. Select a larger depth.

Assuming a depth of 2.16
A=22.6; P=17.7; R=1.28; Q=265. Assume 266 cfs. OK

Calculate Shear stress on the unlined channel.

$$\tau_{max} = \gamma \times d \times S = 62.4 \times 2.16 \times 0.03 = 4.04 \text{ lbs/ft}^2$$

Since the maximum shear stress of 4.04 lbs/ft^2 is less than the capacity of the unvegetated TRM (4.55 lbs/ft^2), the design is acceptable.

Surface Roughening

Source: Clackamas County, 2000

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control	■		
Sediment Control			
Runoff Reduction	■		
Flow Diversion			

Description: Surface roughening is a temporary practice incorporated during grading, that reduces soil loss by reducing the flow velocity of runoff. Surface roughening may also be used as a method of reducing dust (See Section 7E-16 - Dust Control).

Typical Uses: For slopes where additional grading is anticipated prior to permanent/temporary stabilization. To reduce runoff velocity, trap sediment, increase infiltration, and aid in the establishment of vegetative cover. Typically performed as an end-of-day practice.

Advantages:
- Simple and cost-effective.
- Immediate, short-term control.
- Reduces both wind and water erosion.

Limitations:
- Could increase soil compaction, requiring additional seedbed preparation.
- Not a stand-alone practice - it must be used in conjunction with other erosion and sediment control measures.

Longevity: Short-term, depends on precipitation

SUDAS Specifications: Refer to Section 9040, 3.23

A. Description/Uses

Disturbed, non-vegetated areas that are graded smooth and have compacted soil cause increased runoff and reduce the ability of vegetation to be re-established, resulting in erosion. Surface roughening abrades the soil surface with horizontal ridges and depressions across the disturbed area. The use of this practice helps lessen erosion and sediment transport during grading operations.

B. Design Considerations

Surface roughening is not a stand-alone measure, and should always be used in conjunction with other erosion and sediment control practices. Surface roughening may be applied after grading activities cease (temporarily), but will be resumed again within 21 days. Surface roughening might also be employed on an actively graded slope, prior to an impending storm, to provide some level of erosion protection.

Roughening methods include creating furrows across the slopes and tracking up and down the slope. The type of roughening depends on the steepness of the slope and the soil type.

1. **Directional Tracking:** Tracking uses the depressions formed by the tracks from bulldozers and other construction vehicles. The vehicle is driven up and down the slope, leaving behind horizontal depressions in the soil. These depressions interrupt the runoff's flow, reducing its velocity and erosive capacity.

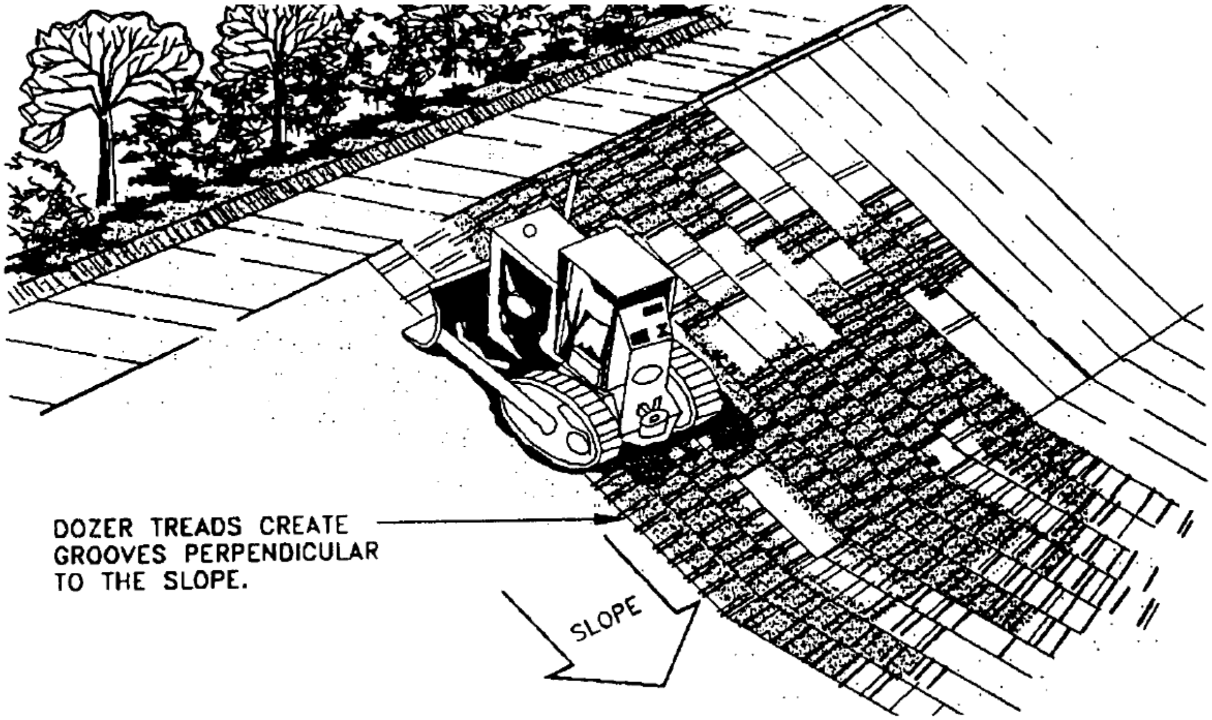
   Directional tracking is the least effective, but likely most convenient, method of surface roughening. Directional tracking should only be performed on slopes that are 3:1 or flatter, as its use on steeper slopes may not prevent concentrated flow from developing. For slopes steeper than 3:1, grooving/furrowing should be used (see information below).

   Directional tracking is ideally suited for sandy soils, as they do not compact as severely. Its use on clay-based soils should be limited, unless no other alternatives are available. As few passes of the machinery should be made as possible in order to limit compaction.

   It is imperative that the equipment track perpendicular to the contour, creating groves that are parallel to the contour. Tracking along the contour will create vertical grooves and ridges for the runoff to follow, actually increasing the erosion potential.

2. **Grooving:** Grooving is a method of surface roughening that creates a series of ridges and depressions along the contour of the slope. Grooving may be accomplished with rippers, disks, spring harrows, chisel plows, or any equipment capable of operating safely on the slope. The grooves created should be no more than 15 inches apart and should not measure less than three inches in depth. Grooving is more effective erosion control practice than vehicle tracking and may be used with all soils types and all slopes.

Figure 7E-19.01: Typical Directional Tracking on a Bare Slope



Source: US Army Corps of Engineers, 1997

Regardless of the method used, after the disturbed area has been roughened it should be protected from vehicular traffic as it may greatly reduce the efficiency of the roughening and require the practice to be repeated. At no time should slopes be bladed or scraped to produce a smooth, hard surface.

C. Application

Surface roughening is a simple method of providing at least a minimal level of short-term erosion protection for slopes which are still under construction, but on which work is being halted for a short period of time. Surface roughening should be provided on all slopes at the end of the workday. It can also be done, in conjunction with mulching, after the fall seeding period has passed to stabilize a site and carry it through the winter months.

D. Maintenance

Surface roughening is a short-term practice that needs to be reapplied whenever the roughened surface is removed by re-grading or weather conditions. Typically, surface roughening on a slope will need to be reestablished after each rain event, regardless of intensity.

Inlet Protection

Source: Soil Tek

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control	■		
Runoff Reduction			
Flow Diversion			

Description: Inlet protection devices consist of a variety of manufactured sediment barriers and products, which are used to filter runoff before it enters the storm sewer system.

Typical Uses: Inlet protection is considered the last line of protection against releasing sediment into the stormwater system or a water body. Inlet protection should be considered around all stormwater intakes and culverts that accept runoff from disturbed areas.

Advantages:
- Provide one last opportunity to remove suspended particles from stormwater runoff.
- Areas requiring protection are easy to identify during both planning and construction.

Limitations:
- Available practices are not effective at removing fine particles.
- May be used improperly as the sole method of erosion and sediment control.
- Require high level of maintenance.
- Limited to treating runoff from areas of 1 acre or less.

Longevity: Varies by product; until sediment accumulates and clean out is required

SUDAS Specifications: Refer to Section 9040, 2.18 and 3.24

A. Description/Uses

Inlet protection can be provided by a variety of methods. A number of new manufactured products are currently available which claim to adequately filter runoff before it enters the storm sewer intake. The effectiveness of these products has yet to be determined.

The traditional method of providing inlet protection is to construct a filter at the opening. The filter is constructed from wire mesh or a steel plate, filter fabric, and crushed stone.

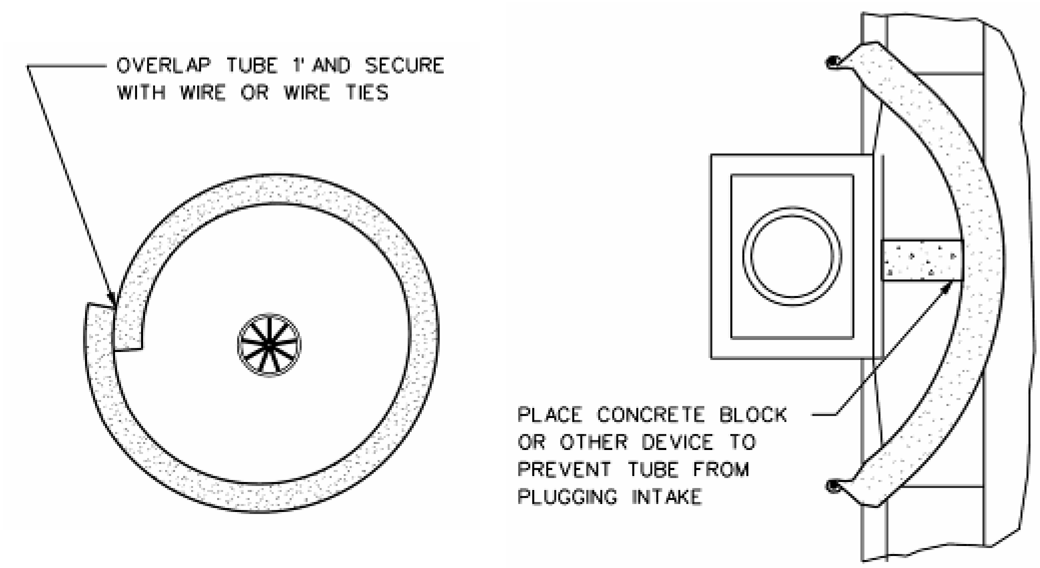
B. Design Considerations

Most inlet protection devices rely on filtering techniques or on ponding small volumes of water to remove suspended particles. In general, the only way to remove fine particles from suspension is to detain the runoff for an extended period of time. Because inlet protection devices do not have the ability to pond and store large volumes of water, they are generally considered ineffective at removing fine particles from suspension in runoff. However, they are the last line of protection against releasing sediment-laden runoff into a stormwater system or water body. In addition, they may provide some benefit by trapping a portion of the larger suspended particles.

Because of their relative inefficiency compared to other techniques, inlet protection devices should not be used on a project as the sole method of sediment removal.

The traditional method for providing inlet protection was to construct a filter at the opening. The filter was constructed from wire mesh, filter fabric, and crushed stone. Runoff flowing to the intake would percolate through the stone and filter fabric before entering the intake. This stone medium slowed the flow of water and filtered larger sediment particles from the water. Today, these methods have been replaced with alternative techniques and materials.

Figure 7E-20.01: Filter Tubes Used for Inlet Protection



Silt fence, placed around the perimeter of an area intake, can also serve as an inlet protection device. Silt fence used around an intake should be reinforced with 6 by 6 inch welded wire fabric, placed on the inside of the silt fence and securely attached to the posts. Silt fence should not be placed where concentrated flows are expected.

Filter socks may be used around the perimeter of an area intake, or in locations where silt fence cannot be installed, such as paved areas. Refer to Section 7E-4 - Filter Socks for additional information on using filter socks around intakes.

A variety of manufactured products are available including storm intake filter socks, synthetic filter tubes for open throat curb intakes, intake inserts, pop-up filters for area intakes, and many others. These products should be used and installed according to the manufacturer's recommendations.

Using any inlet protection device that restricts the flow into the intake should be avoided for intakes that are on-grade. Because of the flow restriction, a majority of the flow to an on-grade intake will be bypassed to the downstream intake. This creates the potential for flooding problems downstream. To limit the potential for flooding, the drainage area to a protected inlet should be limited to 1 acre. For drainage areas larger than 1 acre, temporary sediment traps, flow diversion, or other methods should be considered.

C. Application

Inlet protection devices should be considered for inlets that are to receive runoff from small disturbed areas (less than 1 acre). These devices are used as a last line of defense against releasing sediment into the storm sewer or a water body.

D. Maintenance

Inlet protection devices are easily plugged, and may require a high level of maintenance. The devices should be cleaned out or replaced when standing water is still evident 48 hours after a rain event.

Flow Transition Mats

Source: ScourStop, 2006

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control	■	■	■
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Transition mats are a synthetic alternative to using rip rap.

Typical Uses: Used to dissipate energy and prevent scour at the transition from highly concentrate flow outlets to channel flow.

Advantages:
- Vegetated condition is more aesthetically pleasing than rip rap.
- Installation can be mowed with conventional equipment.
- May be utilized as a temporary measure.
- May be more economical than other “hard-armor” methods.
- Installation does not require any heavy equipment.

Limitations:
- Continuous flow channels may not support vegetation.
- Not appropriate for high velocity discharges (>16 fps).

Longevity: Permanent

SUDAS Specifications: Refer to Section 9040, 2.19 and 3.25

A. Description/Uses

A transition mat is a HDPE, UV stabilized, plastic sheet approximately 4 feet by 4 feet by 1/2 inch thick, comprised of multiple voids which allow vegetation to grow through, or small gravel and pebbles to accumulate and stabilize the area. The mat protects the area at pipe outlets from scour until the water spreading out in the channel diminishes the turbulent forces. The channel downstream of the outlet, where flow becomes uniform, must still be evaluated to ensure that the channel lining can withstand the anticipated shear stress.

B. Design Considerations

Generally, vegetation alone and a vegetated turf reinforcement mat (TRM) (Section 7E-18) can carry significant storm water shear, but cannot withstand the turbulence and concentrated flow generated by a hard surface such as storm sewers, culverts, or parking lots. At these locations, additional measures are usually required to prevent scour. Transition mats are one option for protecting that critical area.

Transition mats can be installed in several different configurations to meet the particular site requirements.

Transition mats installed over sod are good applications for parking lot outlets or pipe outlets conveying storm water through residential developments. The installation can be mowed with standard equipment and unsightly rock rip rap is avoided.

Installing sod and a Type 1 TRM under the transition mat adds a strong supporting element to the system. Vegetated TRMs already have proven shear force resistance of 12 pounds per feet. The sod eliminates the germination issue of a plain TRM installation, even though it adds a slight cost of material and labor, as well as potential short-term irrigation needs. Appropriate uses for a transition mat over a Type 1 TRM and sod would be 24 to 48 inch storm water pipes.

Transition mats may also be used without sod. A transition mat with a Type 1 TRMs over bare soil might be used in situations where turfgrass is not desired, such as a rural area, or as a temporary installation. When used without sod, the flows should be slow and the area fairly flat to encourage sediment accumulation in the voids, where vegetation could also start. Pipe sizes should be limited to 24 inches.

Higher flow installations without sod can be accommodated using a higher class, Type 3 TRM over the bare soil. This type of installation may be applicable for temporary, pre-vegetation erosion control use (temporary meaning remove and reinstall when vegetation can be established), or as a permanent installation requiring substantial soil protection and vegetation growth over time. This installation could also be used in a streambed, where the mats would collect small gravel and sediment in the voids and appear naturally stabilized.

Installations with continuous low flows, such as irrigation over charge, should utilize a sub-surface drainage system directly downstream of the outlet to drain that low flow from the surface, thus allowing vegetation to properly establish. Of course, adequate slope is required for a subdrain system. In some instances, marsh plants could be planted into a transition mat and TRM combination as another solution.

For installations where the slope of the discharge area or channel is greater than the outlet, but not a waterfall situation, transition mats should perform as specified. When the slope of the discharge area or channel is flatter than the outlet, and the grade break between the two exceeds 8%, the flow velocities and vector forces directed into the transition mat should be considered to determine if a flow transition mat is appropriate for the situation.

A temporary installation, for example the outlet of a temporary slope drain, can be readily achieved with a transition mat and TRM combination. Vegetation would generally not be necessary or desired, but scour protection would be quickly achieved, and the materials could be easily picked up and moved to another area on demand.

Transition mats do not dissipate energy by impact like rip rap, but generally rely on the expansion area downstream to dissipate scour forces. The expansion area should be as wide and flat as possible. Channel side slopes that restrict expansion require protection with either a TRM or other means.

In addition to the potential scour area at the outlet, the channel downstream of the transition mat should be evaluated to ensure that it can carry the anticipated flows without eroding the streambank. Additional information on evaluating channel linings can be found in Section 7E-23 - Grass Channel and 7E-18 - Turf Reinforcement Mats.

C. Application

Outlet protection should be designed to withstand the 10 year storm event. The following table lists the recommended dimensions for transition mat and TRM (if used) installations based upon pipe diameter.

Table 7E-21.01: Flow Transition Mat Application

Pipe Diameter (inches)	Discharge[1] (cfs)	Transition Mats		TRM
		Width (ft) x Length (ft)	*Quantity*	*Width (ft) x Length (ft)*
12	8	4 x 4	1	6 x 8
24	30	4 x 8	2	11 x 12
36	75	8 x 12	6	17 x 16
48	100	12 x 16	12	23 x 20

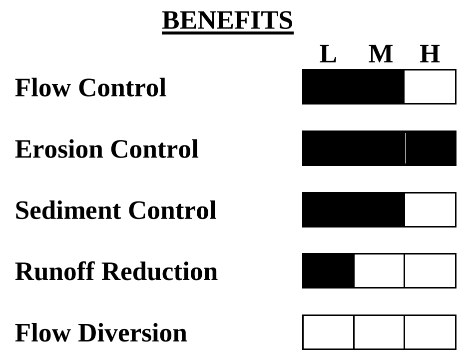
[1] If the design discharge exceeds that for the diameter shown, alternative methods of outlet protection should be provided.

D. Maintenance

Transition mats are generally permanent installations, and maintenance should not be necessary. Utilized in a temporary installation, the transition mats and TRMs can be picked up and moved when appropriate.

Temporary Erosion Control Seeding

Source: Mississippi State University



Description: Temporary seeding is a means of growing a short-term (less than one-year) vegetative cover on disturbed areas that may be in danger of erosion. The purpose is to stabilize disturbed areas with existing or expected high rates of soil erosion by water or wind, reduce damage from sediment and runoff to downstream areas, and improve water quality.

Typical Uses: Temporary seeding may be used to stabilize rough-graded disturbed areas that will not have permanent stabilization or further work performed on them for a period of 21 days or more, and which require temporary stabilization for a period of less than one year.

Advantages:
- Relatively low cost.
- Competes with undesirable vegetation such as noxious weeds.
- Reduces flow velocity, thus reducing erosion potential.
- Traps suspended sediment.
- Improves construction site appearance.
- Reduces maintenance and clean out requirements associated with other erosion control structures (i.e. sediment basin cleanout frequency will be reduced if site is stable).
- Effective measure for dust control.

Limitations:
- Requires sufficient time and moisture to establish.
- Planted areas are susceptible to wind and water erosion until vegetation is established.
- Seasonal limitations on planting may not coincide with construction schedule.
- Method is only effective for one growing season.

Longevity: One growing season

SUDAS Specifications: Refer to Section 9010 (Seeding)

A. Description/Uses

Temporary seeding for construction site erosion control consists of planting appropriate rapidly growing vegetation on disturbed/denuded soil areas to reduce soil loss (erosion and sedimentation), decrease stormwater runoff volume, and lessen problems associated with mud and dust production from bare, unprotected soil surfaces. Through seeding, a fibrous root system is established. This holds the soil in place and provides a canopy over the soil, protecting it from raindrop impact. Typical applications for temporary seeding include stabilizing the denuded surface of excavations, slopes, diversions, dams, sediment basins, road embankments, and stockpiles.

The NPDES General Permit No. 2 requires that all disturbed areas where no construction activities are scheduled for a period of 21 calendar days or more, be stabilized within 14 days of the final construction activity. Temporary seeding is one way to meet this requirement.

B. Design Considerations

The following should be considered for all sites that are to be stabilized with either temporary or permanent seeding.

1. **Site Stabilization:** Minimize steep slopes, which increase the erosion hazard, and make seedbed preparation difficult. Concentrated flows should be diverted away from the seeding area.

2. **Sediment and Water Control Devices:** Prior to seeding, necessary control practices such as dikes, swales/waterways, or sediment basins or diversions should be installed.

3. **Seeding Methods:** There are four seeding methods to consider:

    a. Broadcast seed spreader/cyclone seeder

    b. Mechanical drill or cultipacker

    c. Hydroseeder in which the seed is intermixed with mulch and water to creates a slurry.

    d. Pneumatic seeder in which the seed is intermixed with compost or a compost/soil blend

    When hydroseeding and pneumatic seeding are utilized, the surface may be left with a more irregular surface, since these practices will fill small depressions and cover small bumps. These two types of seeding methods can be used in situations where slope and accessibility is a limiting factor and seedbed preparation is not possible, or where the application of seed, mulch and fertilizer (if necessary) in one operation is desirable.

    Hand broadcasting seed may be utilized for small or inaccessible areas; however, it is not recommended for larger areas because of the difficulty in achieving a uniform distribution.

4. **Seedbed Preparation:** Seedbed preparation is essential for the vegetation’s ability to germinate and grow. Seedbed preparations considerations include:

    a. Topsoiling is not necessary for temporary seeding, though it may improve the chances of vegetation establishment. When the area is crusted or hardened, the soil surface should be loosened by disking, raking, harrowing, or other acceptable means to a depth of approximately 2 inches. If the area has been recently loosened, no further roughening is required.

b. Soil compaction severely hinders seeding success rate and increases runoff rates. If the area has been compacted by heavy equipment, the surface to be seeded should be chisel plowed to a depth of 6 to 12 inches once heavy equipment has been removed from the area.

c. The soil pH should have a range of 5.5 to 7.5. Where soils are known to be highly acidic (pH 6.0 and lower), lime should be applied at the rate recommended by the soil-testing laboratory.

d. Fertilizer application is required for temporary seeding.

5. **Seed Mixture:** Unless a specific seed mixture is required, the seed mixtures described in SUDAS Specifications Section 9010 may be utilized. These are annual seed mixtures and are only intended to provide protection for one growing season (6 to 8 months). For applications requiring protection longer than one growing season, reseeding in the spring or dormant seeding in the fall may be required. Alternatively, areas which will not be disturbed for a period greater than can be protected by a temporary seed mixture, should have permanent seeding applied (refer to Section 7E-24)

Table 7E-22.01: Temporary Erosion Control Mixes

Seed Mixture	Allowable Seeding Dates
Spring Mix	March 1 - May 20
Summer Mix	May 21 - August 14
Fall Mix	August 15 - September 30

Source: SUDAS Specifications Section 9010

6. **Weather:** When seeding, be aware of the weather. Do not seed when heavy rainfall is predicted, during windy weather or on wet/frozen ground (hydroseeding and pneumatic seeding may be an exception to seeding on wet/frozen ground).

7. **Matting:** A rolled erosion control product is recommended for slopes steeper than 3:1. RECPs may also be required for flatter slopes greater than 100 feet in length, to hold the seed in place and protect new vegetation from runoff until it becomes established. Refer to Section 7E-5 - Temporary Rolled Erosion Control Products.

8. **Mulching:** For temporary seeding, mulching is advised when seeding in the summer or during excessively hot or dry weather to maintain moisture levels; in the fall for winter cover; on slopes steeper than 3:1; and on adverse soils (shallow, rocky, or high in clay or sand). Mulching is not advised in concentrated flow situations. Refer to Section 7E-17 - Erosion Control Mulching.

9. **Moisture:** If normal rainfall is insufficient to ensure vegetation establishment, mulching, matting, or controlled watering should be completed to keep seeded areas adequately moist.

C. Application

In order to achieve the appropriate vegetation density, temporary seed mixtures and fertilizer should be applied at the rates specified in the SUDAS Specifications.

D. Maintenance

Once the area is seeded, it should not be disturbed and should be protected from traffic. Newly seeded areas should be inspected weekly as part of the overall erosion control inspection, to ensure that grass is growing satisfactorily. Areas that have bare spots or where erosion has occurred, should be re-seeded. Temporary seeding should be maintained until the area is again disturbed by construction, or permanent stabilization is achieved.

E. Time of Year

The temporary seeding mixture used should be based upon the time of year as indicated in Table 7E-22.01. The dates given are approximate and may be adjusted to account for annual weather patterns.

Grass Channel

Source: NRCS photo gallery

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Grass channels consist of ditches, swales, or waterways that are lined with vegetation to stabilize the surface from erosion.

Typical Uses: Used to carry intermittent, low to moderate concentrated flows of surface runoff.

Advantages:

- Low cost method of conveying surface runoff.
- Highly effective for controlling channel erosion for low to moderate flows.
- Aesthetically pleasing.
- Reduces flow velocity and removes sediment.

Limitations:

- Cannot withstand forces from high flows.
- There may be some difficulty establishing vegetation.
- Not suitable for channels that carry constant flows, or that remain submerged for extended periods of time.

Longevity: Permanent

SUDAS Specifications: Refer to Section 9010 (Seeding) or 9020 (Sodding)

A. Description/Uses

Grass channels consist of swales, ditches, and waterways that are lined with permanent vegetation. The purpose of the vegetation is to stabilize the surface of the channel and prevent erosion from concentrated stormwater flow.

Because these structures are lined with vegetation, they cannot be used for channels which have constant flow, or which will be submerged for extended periods of time.

Grass channels are the least costly and most aesthetically pleasing option for lining channels.

B. Design Considerations

As water flows through any conduit or channel, the surface of the conduit or channel imparts drag on the flowing water. The amount of drag a particular surface will create is related to the commonly known Manning’s “n” coefficient. This drag force not only slows the flow of the water, but also imparts a corresponding force onto the lining of the channel. This force is known as shear stress.

The ability of a channel to withstand shear stress is dependent on the properties of the lining. If the shear stress imposed on the bottom and sides of a channel by the flowing stormwater exceeds the ability of the channel lining to withstand it, the lining will be moved or damaged. Various types of vegetation provide different levels of resistance to shear. Table 7E-23.01 lists the various classifications of vegetation that have been established and analyzed.

Prior to movement of the lining, the underlying soil is protected from the erosive forces of the flowing water. Therefore, the erodibility of the underlying soil has little effect on the permissible shear stress of the lining. However, if the grass lining is moved or damaged, the underlying soil properties become a significant factor in determining the degree of erosion that will occur.

Calculating shear stress in a channel is a two step process. First, the depth of flow in the channel is determined with Manning’s equation. For temporary stabilization, the channel liner should be designed to carry a 2 year storm event. For permanent stabilization, the liner should be designed for a 10 year event.

For most channel lining materials, Manning’s n value does not vary significantly as the depth of flow varies, and is normally assumed to be constant. For grass channels however, the n value varies greatly with the depth of flow. This variation is caused by the reaction of the grass to the flow. As flow depth increases, the grass is bent over, thereby reducing its height and changing the resistance it imparts on the flow.

The following equations, along with the vegetation data listed in Table 7E-23.02, can be used to calculate the Manning value for a given depth of flow and vegetation type. For vegetated conditions, NRCS has determined that actual Manning’s n values range only from 0.02 to 0.5. When calculated values fall outside of this acceptable range, the designer should use the upper or lower limit of the range. If the denominator of Equation 7E-23.01 is zero or less than zero, a Manning’s n value of 0.5 should be used.

$$n = \frac{R^{1/6}}{19.97\left(\log\left(44.8 \times h^{0.6} \times MEI^{-0.4}\right) + \log\left(R^{1.4} \times S^{0.4}\right)\right)}$$ **Equation 7E-23.01**

Where:

n	=	Manning's coefficient (dimensionless)
R	=	Hydraulic radius (ft.)
h	=	Average height of vegetation; from Table 7E-23.01 (ft)
MEI	=	Stiffness factor; from Table 7E-23.02 (lb.· ft^2)
S	=	Channel slope (ft/ft)

Source: Chen & Cotton, 1988 (HEC-15)

Because the Manning coefficient changes with depth, calculating the depth of flow is an iterative process. Once the flow depth is determined, the shear stress on the channel liner is determined by the following equation:

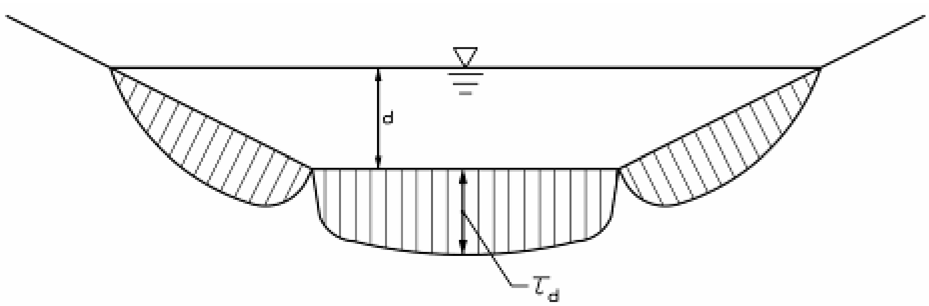
$$\tau_d = \gamma \times d \times S$$ **Equation 7E-23.02**

Where:

τ_d	=	Shear stress in channel at maximum depth (lbs/ft^2)
γ	=	Unit weight of water (62.4 lbs/ft^3)
d	=	Depth of flow (ft)
S	=	Channel slope (ft/ft)

The shear stress distribution along the wetted perimeter of a channel is not uniform, as indicated in Figure 7E-23.01. In a trapezoidal channel, the peak shear stress in a straight channel occurs at the center of the bottom of the channel. The stress in the corners of the channel approaches zero. The peak shear stress along the sides of a straight channel occur near the bottom third of the channel.

Figure 7E-23.01: Stress Distribution in a Trapezoidal Channel

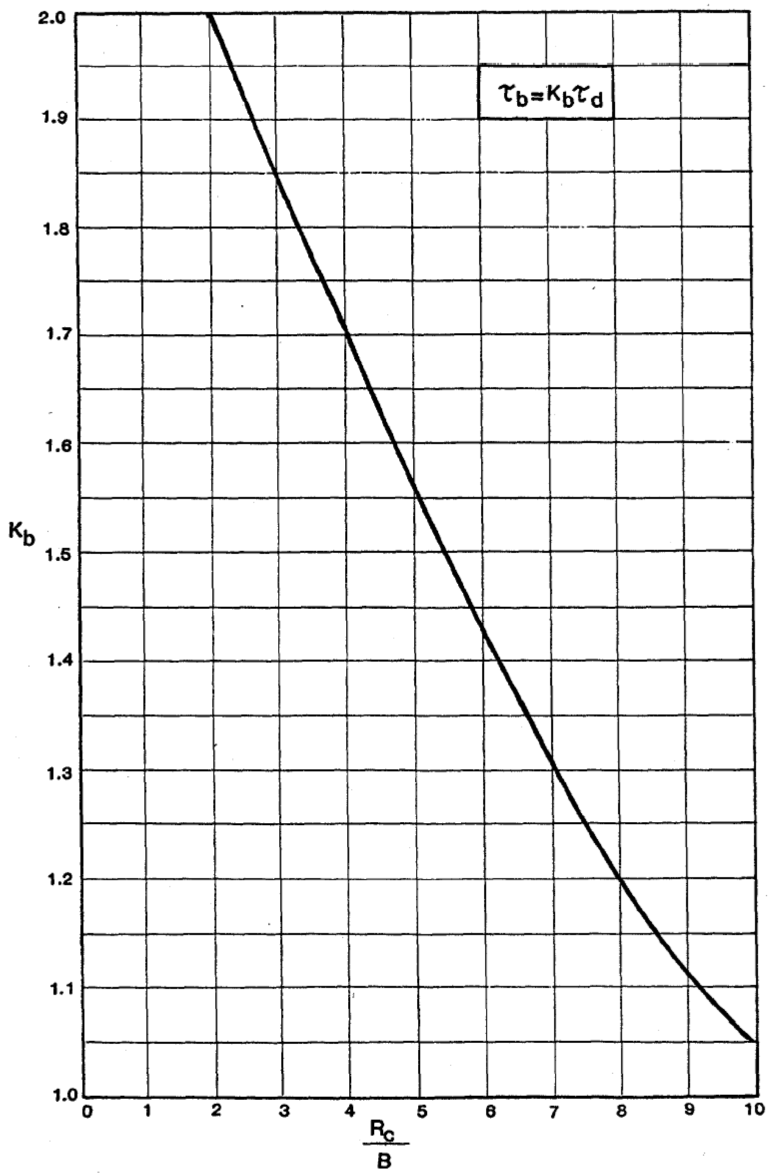


Source: Adapted from Chen & Cotton, 1988 (HEC-15)

If the flow travels around a bend, the current imposes additional forces on the channel as the flow is redirected. These forces result in increased shear stress on the bottom and sides of the channel. The additional shear stress imposed on the channel is related to the ratio of the radius of the bend, R_c, and the bottom width of the channel, b. As the bend becomes sharper, the shear stress increases. The maximum shear stress in the bend is determined by multiplying the calculated shear stress in a straight section of channel by the bend coefficient, K_b (Equation 7E-23.03). K_b is determined from Figure 7E-23.02.

$$\tau_b = K_b \tau_d$$ **Equation 7E-23.03**

Figure 7E-23.02: Bend Coefficient for Maximum Shear Stress in Channel Bends



Source: Chen & Cotton, 1988 (HEC-15)

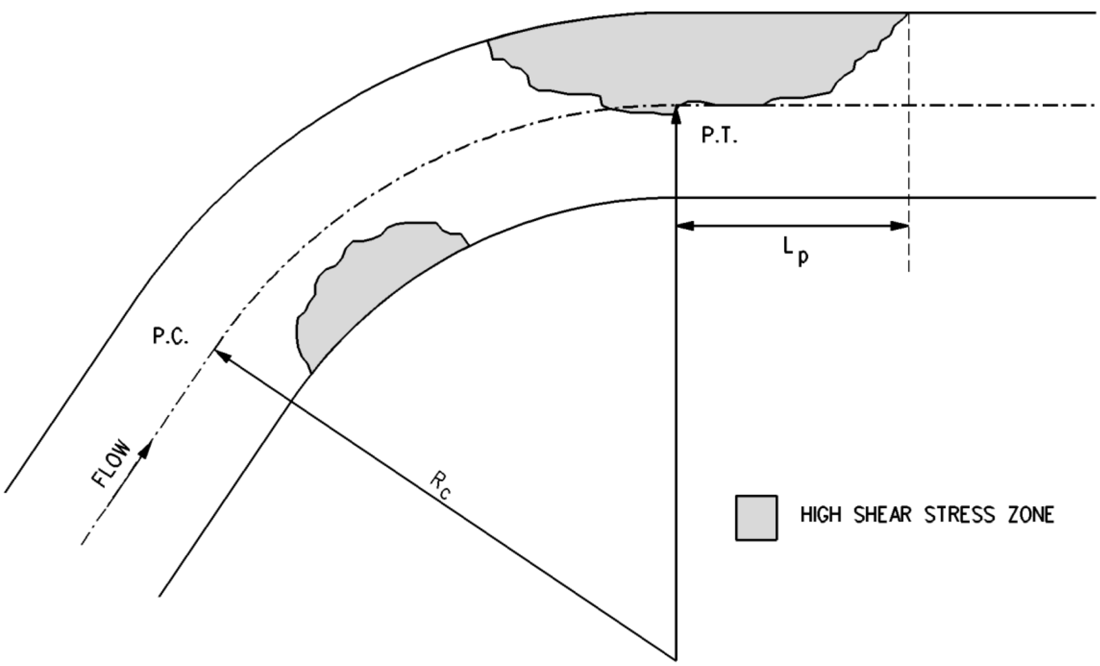
As flow travels around a bend, the increased shear stresses begin along the inside radius and move toward the outside. These increased stresses are also transmitted down the channel for a distance L_p, due to the turbulence created in the flow as it traveled around the bend (see Figure 7E-23.03). This distance can be determined by Equation 7E-23.04. When additional channel protection is provided in the bend, it should also be extended through this length.

$$L_p = 0.604 \frac{R^{7/6}}{n}$$ **Equation 7E-23.04**

Where:

L_p = Length of protection required downstream of bend, ft
R = Hydraulic radius
n = Manning's coefficient

Figure 7E-23.03: Shear Stress Distribution in a Channel Bend



Source: Adapted from Chen & Cotton, 1988 (HEC-15)

Once the anticipated shear stress on the channel liner is determined, it is compared to the allowable shear stress values of the proposed vegetation. If the calculated shear stress value exceeds the allowable shear stress of the liner, additional protection may be required. Depending on the level of shear stress anticipated, additional protection may be provided by an alternate type of vegetation, by reinforcing the vegetation with a turf reinforcement mat, lining the channel with rip rap, or modifying the geometrics of the channel.

For channels where establishment of vegetation may be difficult, a rolled erosion control product may be considered. A complete discussion on RECPs can be found in Section 7E-5.

A more complete discussion on channel stabilization is provided in Chapter 2 - Stormwater.

Table 7E-23.01: Classification of Vegetation

Vegetation Class	Cover	Condition
A	**Weeping lovegrass**	Excellent stand, tall (average 30”)
	Yellow bluestem ischaemum	Excellent stand, tall (average 36”)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12”)
	Native grass mixture (**little bluestem**, bluestem, **blue grama,** other long and short Midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24”)
	Lespedeza serices	Good stand, not woody, tall (average 19”)
	Alfalfa	Good stand, uncut (average 11”)
	Weeping lovegrass	Good stand, unmowed (average 13”)
	Kudzu	Dense growth, uncut
	Brome, smooth	Good stand, mowed (average 12” to 15”)
	Tall fescue	Good stand, uncut (average 18”)
	Tall fescue with birdsfoot trefoil	Good stand, uncut (average 18”)
	Grass - Legume mixture - Timothy, Smooth bromegrass, or Orchardgrass	Good stand, uncut (average 20”)
	Blue grama	Good stand, uncut (average 13”)
C	Crabgrass	Fair stand, uncut (10” to 48”)
	Bermuda grass	Good stand, mowed (average 6”)
	Red top	Good stand, headed (15” to 20”)
	Common lespedeza	Good stand, uncut (average 11”)
	Grass-legume mixture - Summer (**orchard grass**, **redtop,** Italian ryegrass, and common lespedeza)	Good stand, uncut (6” to 8”)
	Centipedegrass	Very dense cover (average 6”)
	Kentucky bluegrass	Good stand, headed (6” to 12”)
D	Bermuda grass	Good stand, cut to 2.5” height
	Common lespedeza	Excellent stand, uncut (average 4.5”)
	Buffalo grass	Good stand, uncut (3” to 6”)
	Grass-legume mixture fall, spring (**orchard grass, redtop,** Italian ryegrass, and common lespedeza)	Good stand, uncut (4” to 5”)
	Kentucky bluegrass or Lespedeza sericea	Good stand, cut to 2” height. Very good stand before cutting.
	Red fescue	Good stand (headed (12” to 18”)
E	Bermuda grass	Good stand, cut to 1.5” height
	Bermuda grass	Burned stubble

Note: covers classified have been tested in experimental channels. Covers were green and generally uniform
Items shown in **Bold** are seed varieties included in the SUDAS Specifications.

Source: Chen & Cotton, 1988 (HEC-15) and USDA NRCS, 1986

Table 7E-23.02: Vegetation Properties

Vegetation Class	Permissible Shear Stress (lb/ft^2)	Average Height, h (feet)	Stiffness, MEI (lb/ft^2)
A	3.7	3.0	725
B	2.1	2.0	50
C	1.0	0.66	1.2
D	0.6	0.33	0.12
E	0.35	0.13	0.012

Source: Chen & Cotton, 1988 (HEC-15)

C. Application

Grassed channels are an excellent low-cost stabilizing method for swales and ditches that carry intermittent low to moderate concentrated flows.

D. Maintenance

Proper maintenance of the channel is critical. For designs where vegetation is assumed to be unmowed or at a minimum height, it is important to ensure that the vegetation in the channel is maintained in the manner intended. Mowing a channel, which was not designed to be kept at a short height, could result in failure of the grass channel. If there is a possibility that the channel could be mowed, it should be designed as such.

Newly seeded or sodded areas should be maintained and watered as required to ensure establishment of the grass. See Sections 7E-22 - Temporary Erosion Control Seeding and 7E-25 - Sodding.

E. Time of Year

Grass channel liners require the vegetation to be well-established in order to provide maximum protection from erosion. Seeding a channel near the end of the annual seeding window may not allow enough time for the vegetation to develop sufficiently to resist flows from winter snowmelt or spring rains.

F. Design Example

Assume a grass channel with a 3 foot bottom, 4:1 side slopes, and a slope of 1% is designed to carry 24 cfs. Determine if the proposed Class C vegetation is adequate.

Solution:

First, use Manning's equation to find the depth of flow. This can be done through a trial and error process, or by using various tables and charts. For grass channels, Manning's n value varies, and must be calculated based upon the depth of flow. From Table 7E-23.02, the average height, h, for Class C vegetation is 0.66 ft, the stiffness, MEI, is 1.2 $lb \cdot ft^2$, and the permissible shear stress is 1.0 lbs/ft^2.

Trial 1 - Assume a trial depth of 1.2 feet.

Area of Flow, $A = (b + Z \times d) \times d = (3 + 4 \times 1.2) \times 1.2 = 9.4\,\text{ft}^2$

Wetted Perimeter, $P = b + 2 \times \sqrt{d^2 + (Zd)^2} = 3 + 2 \times \sqrt{1.2^2 + (4 \times 1.2)^2} = 12.9\,\text{ft}$

Hydraulic Radius, $R = A/P = 9.4/12.9 = 0.73\,\text{ft}$

Manning coefficient (from Equation 7E-23.01):

$$n = \frac{0.73^{1/6}}{19.97\left(\log\left(44.8 \times 0.66^{0.6} \times 1.2^{-0.4}\right) + \log\left(0.73^{1.4} \times .01^{0.4}\right)\right)} = 0.092$$

Solving Manning's yields: $Q = {}^{1.49}\!/_{n}\, AR^{2/3} S^{1/2} = \left({}^{1.49}\!/_{0.092}\right)(9.4)(0.73)^{2/3}(0.01)^{1/2} = 12.2\,\text{cfs}$

Since 12.2 cfs is lower than the design value of 24 cfs, a larger depth should be assumed.

Trial 2 - Assume a depth of 1.5 feet.

Following the procedure for Trial 1: A = 13.5; P=15.4; R=.88; n= 0.077; Q= 24 cfs
Now that the depth of flow is known, the shear stress on the channel bottom can be determined by Equation 7E-23.02.

$$\tau_{max} = \gamma \times d \times S = 62.4 \times 1.5 \times 0.01 = 0.94\ \text{lbs/ft}^2$$

Since the maximum shear stress of 0.94 lbs/ft^2 is less than the capacity of the grass channel liner (1.0 lbs/ft^2), the design should be adequate to protect the channel from erosion.

Permanent Seeding

Source: Iowa NRCS, 2004

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Permanent seeding is a means of establishing permanent, perennial vegetative cover on disturbed areas. The purpose of permanent seeding is to prevent erosion, remove sediment from runoff, reduce the volume of runoff, and improve water quality.

Typical Uses: Permanent seeding is used to stabilize the ground after grading and land-disturbing activities have been completed, or whenever construction activities will be halted for a time period longer than temporary seeding can provide protection (i.e. one growing season).

Advantages:
- Relatively low cost.
- Most common method of providing permanent stabilization of disturbed ground.
- Highly effective as a stand-alone measure in all but the most extreme situations (i.e. continuously flowing channels, steep slopes, high flows, etc.).
- Competes with undesirable vegetation and noxious weeds.
- Vegetation absorbs water, reducing the volume of stormwater runoff.
- Vegetation filters out sediment and other pollutants, improving water quality.
- Provides an aesthetically pleasing, finished look to the site.

Limitations:
- Does not provide instant protection; requires sufficient time and moisture to establish.
- Difficult to establish in area subjected to concentrated flows.
- Seasonal limitations on planting may not coincide with construction schedule.

Longevity: Permanent

SUDAS Specifications: Refer to Section 9010 (Seeding)

A. Description/Uses

Permanent seeding consists of planting perennial vegetation on disturbed/denuded soil areas. Through seeding, a fibrous root system is established. This holds the soil in place and provides a canopy over the soil, protecting it from raindrop impact. The vegetation slows the velocity of the runoff, protecting the surface from sheet and rill erosion, while allowing suspended sediment to be removed. Vegetation also absorbs water from the soil, reducing the total volume of runoff.

B. Design Considerations

Permanent seeding is the most commonly used method of providing permanent surface stabilization. It is an economical, long-term method of providing highly effective stabilization, and is aesthetically pleasing. However, in order to be effective, the designer must select the proper vegetation and recognize the practical limits of vegetation.

The following should be considered for all sites prior to permanent seeding:

1. Site Stabilization: Steep slopes, which increase the erosion hazard, should be minimized. Vegetation alone is normally an effective method of stabilizing slopes that are 3:1 or flatter. For slopes steeper than 3:1, or for flatter slopes carrying runoff from upland areas, a rolled erosion control product may be required to provide slope stabilization until the vegetation is established.

In addition, slopes that are very steep (2:1 or greater) and areas that receive intermittent concentrated flows may require application of a turf reinforcement mat to provide permanent reinforcement to the vegetation.

2. Sediment and Water Control Devices: Measures should be taken to divert sheet and concentrated flows away from areas that are to be seeded until the vegetation is established.

3. Seeding Methods: There are four seeding methods to consider:

a. Broadcast seed spreader/cyclone seeder

b. Mechanical drill or cultipacker

c. Hydroseeder in which the seed is intermixed with mulch and water to create a slurry

d. Pneumatic seeder in which the seed is intermixed with compost or a compost/soil blend

When hydroseeding and pneumatic seeding are utilized, the surface may be left with a more irregular surface, since these practices will fill small depressions and cover small bumps. These two types of seeding methods can be used in situations where slope and accessibility is a limiting factor and seedbed preparation is not possible, or where the application of seed, mulch and fertilizer (if necessary) in one operation is desirable.

Hand broadcasting seed may be utilized for small or inaccessible areas; however it is not recommended for larger areas because of the difficulty in achieving a uniform distribution.

4. Seedbed Preparation: Proper seedbed preparation is essential for the seed to germinate and develop into a dense, healthy stand of vegetation.

a. Subsoil Preparation: Newly graded areas may be severely compacted by the weight of heavy earth-moving and construction equipment. Disking or tilling reduces compaction in the uppermost layer of the soil, providing an adequate growing bed for the seed; however, the soil below this level may remain severely compacted. This compacted layer acts as an impermeable barrier, slowing or preventing the infiltration of water into the ground. Infiltration of precipitation reduces runoff, and recharges groundwater supplies. Techniques for reducing ground compaction, such as deep tillage, should be investigated.

b. **Topsoil:** In order to provide an adequate growing medium, a minimum of 6 inches of topsoil should be placed over the disturbed area prior to seeding. Deeper topsoil depths (8-12 inches or greater) are desirable as they increase the organic matter available for use by the plants, allow for deeper root penetration and increase the moisture holding ability of the soil. These benefits will increase the drought tolerance and long-term health of the vegetation. Where sufficient topsoil is not available, composted material may be incorporated at the rate of 1 inch of compost for every 3 inches of deficient topsoil. This will increase the organic matter content of the soil, and provide an adequate growing medium for vegetation.

c. **Soil pH:** The soil pH should have a range of 5.5 to 7.5. Where soils are known to be highly acidic (pH 6.0 and lower), lime should be applied at the rate recommended by the soil-testing laboratory.

d. **Soil Fertilization:** Soil fertilization is required for permanent seeding. Fertilizer rates specified in the SUDAS Specifications are recommended for most applications. Sites without sufficient topsoil or low organic matter may require higher fertilizer rates, or fertilizer with a higher nitrogen concentration.

5. **Seeding Properties:**

a. **General Mixtures:** The SUDAS Specifications provide a number of seed mixes that are acceptable for most general applications. These mixes and a description of their intended usage are shown in Table 7E-24.01.

Table 7E-24.01: SUDAS Seeding Mixtures

Description	Typical Uses	Allowable Seeding Dates
Type 1 - Permanent Lawn Mixture	Used for residential and commercial turf sites. Fertilized; typically mowed.	March 1 - May 31 August 10 - September 30
Type 2 - Permanent Cool - Season Mixture for Slopes and Ditches	Not typically mowed. Reaches maximum heights of 2 to 3 feet; low fertility requirements; grows in spring and fall; can go dormant in summer.	March 1 - May 31 August 10 - September 30
Type 3 - Permanent Warm-Season Slope and Ditch Mixture	Not typically mowed. Reaches heights of 5 to 6 feet; stays green throughout summer; responds well to being burned in spring; do not apply fertilizer.	March 1 - June 30
Type 4 - Temporary Erosion Control Mixture	Short-lived (6 to 8 months) mix for erosion control.	March 1 - September 30 (seeding dates vary by seasonal mix)
Wetland Seeding	Used in areas designated for wetland grass seeding.	April 1 - June 30 August 1 - August 31
Native Grass and Wildflower Seeding	Used in areas designated for native grass and wildflower seeding.	April 1 - June 30

b. **Special Mixtures:** Some sites require specifically designed or selected mixtures to address individual site characteristics. Site characteristics that require special consideration include very shady areas, detention ponds, wet areas, streambanks, severe slopes, and areas with poor soils.

6. **Weather:** When seeding, be aware of the weather. Do not seed when heavy rainfall is predicted, during windy weather or on wet/frozen ground (hydroseeding and pneumatic seeding may be an exception to seeding on wet/frozen ground).

7. **Matting:** A rolled erosion control product is recommended for slopes steeper than 3:1. RECPs may also be required for flatter slopes greater than 100 feet in length, to hold the seed in place and protect new vegetation from runoff until it becomes established. Refer to Section 7E-5 - Temporary Rolled Erosion Control Products.

8. **Mulching:** Mulching is recommended for most permanent seeding applications. Mulch aids in stabilizing the surface until vegetation is established. Mulch also helps retain soil moisture and maintains temperature conditions favorable to germination. Refer to Section 7E-17 - Erosion Control Mulching.

9. **Moisture:** If normal rainfall is insufficient to ensure vegetation establishment, mulching, matting, or controlled watering should be completed to keep seeded areas adequately moist.

C. Application

In order to achieve a dense, healthy stand of vegetation that will provide long-term surface stabilization, seed mixtures and fertilizer should be applied at the rates specified in the SUDAS Specifications.

D. Maintenance

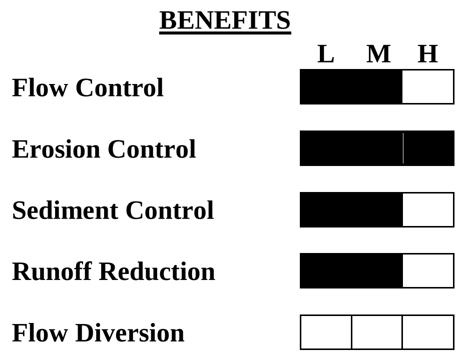
Once the area is seeded, it should not be disturbed and should be protected from traffic. Newly seeded areas should be inspected weekly as part of the overall erosion control inspection, to ensure that grass is growing satisfactorily. Areas that have bare spots, or where erosion has occurred should be re-seeded.

E. Time of Year

The seed mixtures within the SUDAS Specifications should be placed within the dates specified, or as weather conditions allow and if approved by the Jurisdictional Engineer.

Sodding

Source: Welch, J.



Description: A section of grass-covered surface soil held together by matted roots that is cut in pre-determined sections, transported, and delivered directly to the job site ready to install.

Typical Uses: Sod is placed to prevent erosion and damage from sediment and water by stabilizing the soil surface and to improve the visual quality and utility of the area quickly. Sod is typically used in residential or commercial areas where prompt use or aesthetics are important such as building entrance zones or high activity areas. Sod is also used in areas of intermittent concentrated flow such as waterways and channels. Sod may also be utilized in critical areas such as storm drain inlets, steep slopes, and any area where conditions make seeding impractical or impossible.

Advantages:
- Provides immediate erosion and dust control.
- Provides finished landscape appearance at time of installation.
- Reduces likelihood of weed growth.
- Placement can occur any time soil moisture is adequate and ground is not frozen.
- Rapid stabilization of surfaces for traffic areas, channel linings, or critical areas.

Limitations:
- More costly when compared to seeding and mulching.
- Vegetation selection is limited (typically a cool-season bluegrass based mix).
- Time is necessary for root establishment.
- Watering is required to ensure establishment.

Longevity: Permanent

SUDAS Specifications: Refer to Section 9020 (Sodding)

A. Description/Uses

Sodding consists of transplanting turf-type vegetation to promptly stabilize areas that are subject to erosion. Sod may be field sod or commercial sod, a cultured product utilizing specific grass species. A sodded area provides one of the best methods for preventing soil particles from leaving the site, providing immediate protection against soil erosion from water and wind.

B. Design Considerations

The following should be considered for all sites stabilized with sod.

1. **Fertilization:** Test soil to determine the exact requirements for lime and fertilizer. Soil tests should be conducted by the state soil testing lab or a reputable commercial laboratory. Information on soil testing and testing services is available from the Iowa State University Extension (http://www.extension.iastate.edu/).

2. **Site Preparation:** In areas where topsoil has been stripped, a sodbed should be constructed by spreading a minimum of 6 inches of topsoil prior to sodding. Deeper topsoil depths (8 to12 inches or greater) are desirable as they increase the organic matter available for use by the plants, allow for deeper root penetration and increase the moisture holding ability of the soil. These benefits will increase the drought tolerance and long-term health of the vegetation. Where sufficient topsoil is not available, composted material may be incorporated at the rate of 1 inch of compost for every 3 inches of deficient topsoil.

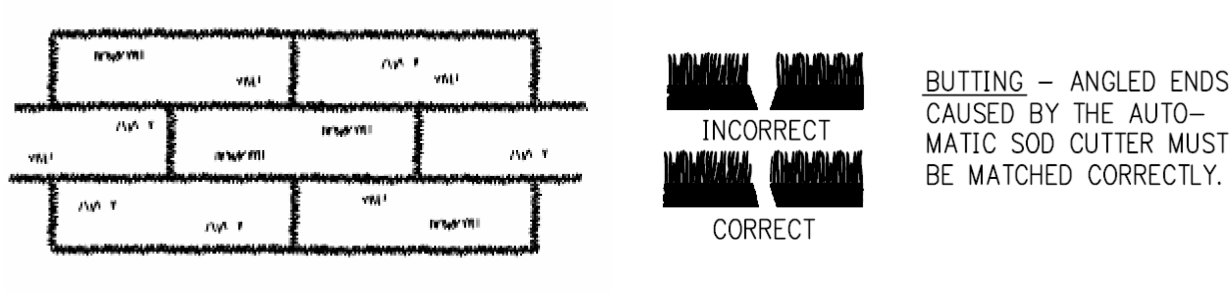
   The top of the completed sodbed should contain a minimum soil organic matter content of 5%. In areas where topsoil has not been stripped, and the soil organic content is low, compost may be placed, as required, to increase the soil organic matter content.

   The top 3 inches of the sodbed should be prepared by tilling, and the surface cleared of any trash, debris, roots, branches, and stones or clods larger than 3/4 inch in diameter. Any low spots should be filled or leveled to avoid standing water. The fertilizer and any other soil amendments should be uniformly applied and incorporated into the top 1 1/2 inches of the soil by tilling or disking. Complete soil preparation by rolling or cultipacking to firm the soil. Avoid using heavy equipment on the area, particularly when the soil is wet, as this may cause excessive compaction and make it difficult for the sod to take root.

   Newly graded areas may be severely compacted by the weight of heavy earth moving and construction equipment. Disking or tilling reduces compaction in the uppermost layer of the soil, providing an adequate growing bed for the sod; however the soil below this level may remain severely compacted. This compacted layer acts as an impermeable barrier, slowing or preventing the infiltration of water into the ground. Infiltration of precipitation reduces runoff and recharges groundwater supplies. Techniques for reducing ground compaction, such as deep tillage, should be investigated.

3. **Installation Techniques:** Sod should be placed as soon as possible after the ground surface has been graded, to take advantage of the ground moisture, and installed within 36 hours of cutting. The soil should be slightly moist, but firm enough not to leave depressions if walked on. Install sod in a straight line at right angles to the direction of the slope, starting at the base of the area to be sodded and working uphill. Sodding operations should be planned so that sloped areas can be completely protected, from bottom to top, prior to halting operations for the day, or before significant precipitation is expected. The angled ends caused by the automatic sod-cutting machine must be matched correctly.

Figure 7E-25.01: Proper Sod Installation



Source: Kansas City APWA, 2003

Place the strips together tightly so that no open joints are left between strips or between the ends of strips. Lateral joints shall be staggered in a brickwork-type pattern to promote uniform growth and strength. Sod should not be overlapped or stretched, and all joints should be butted tightly to prevent voids. Sod should be laid perpendicular to the flow of water on slopes and in waterways. The edges of the sod at the top of the slopes should be slightly tucked under. A layer of soil should be compacted over the edge to conduct surface water over and onto the top of the sod. Fill any spaces between the joints and all sod edges with at least 2 inches of topsoil.

Care shall be taken to prevent voids or over-exposure of the roots, which would cause drying. As sodding of defined areas is completed, sod shall be rolled or tamped to provide firm contact between roots and soil. Seam openings between the mats are a sign the turf is shrinking and that the sod requires more water. Gaps between edges or ends of sod mats should be filled with topsoil and rolled. If sod placement is delayed, it should be kept cool and moist. When placed on slopes steeper than 3:1, or in areas subject to concentrated flow, the sod should be anchored with pins, staples, or other approved methods at the ends and center, or every 3 to 4 feet for longer strips, to prevent movement. Sod should be kept moist until it is firmly rooted which typically takes a minimum of two weeks (see supplemental watering).

4. **Sod Properties:** Sod should be of high quality, which the genetic origin is known, free of noxious weeds, disease, and insect problems consisting of a 3/4 inch mat of vigorous turf. It should appear healthy and vigorous, and conform to the following specifications:

   a. Sod should be live grass, machine cut at a uniform depth of 1/2 to 2 inches (excluding shoot growth and thatch).

   b. Sod strips should be cut with smooth, clean edges and square ends to facilitate laying and fitting.

   c. Sod should not be cut in excessively wet or dry weather.

   d. Frozen sod should never be placed.

   e. Sod should not be permitted to dry out.

   f. Harvested sod pieces can vary from widths of 12 to 48 inches and lengths of 2 to 100 feet, but should be in sections strong enough to support their own weight and retain their size and shape when lifted by one end.

g. As noted in the installation considerations, harvest, delivery, and installation of sod should take place within a period of 36 hours.

h. Sod should be moistened after it is unrolled, which helps to maintain its viability, and stored in the shade if possible, during installation.

5. **Supplemental Water:** After placement is complete, the sod should be irrigated to a depth sufficient that the underside of the sod mat and 4 inches of soil below sod is thoroughly wet. Irrigate at a rate that does not result in runoff. The moisture level can be checked by lifting a corner of a sod roll, and verifying that water is penetrating well into the subsoil.

   As a rule of thumb, watering should be scheduled as follows:

   a. **First Week:** The sod soil should be kept moist at all times. During dry spells, the sod should be watered daily, or as often as necessary to maintain moist soil. The sod should be watered during the heat of the day to prevent wilting.

   b. **Second and Subsequent Weeks:** Water sod to maintain adequate moisture in the soil until the grass takes root. This can be determined by gently tugging on the sod. Resistance indicates that rooting has occurred.

   c. **Summer Installations (June through August):** Summer installations require high levels of attention to water application needs, as newly installed sod will dry out rapidly, suffering significant setback or total loss.

C. Application

The NPDES General Permit No. 2 requires that all disturbed areas, where no construction activities are scheduled for a period of 21 calendar days or more, be stabilized within 14 days of the final construction activity. Sodding is one way to meet this requirement.

D. Maintenance

The sodded area should be inspected daily for at least two weeks, or until the sod is established, to ensure that the moisture content is sufficient and that root establishment is proceeding. The sod should not be mowed regularly until it is well established, and the roots have knitted down. The turf should never be mowed shorter than 2 1/2 inches and no shorter than 3 inches during June, July, and August, in order to increase drought tolerance.

E. Time of Year

Sod availability is seasonal, although it can be laid in nearly all weather conditions. Sod laid during the middle of the summer will require significantly more maintenance and watering. If the ground is frozen, sod cannot be cut and should not be laid; however, if it is available, unfrozen, dormant sod can be laid on unfrozen ground, provided there is not a significant layer of snow.

Vegetative Filter Strip

BENEFITS

	L	M	H
Flow Control	■		
Erosion Control	■		
Sediment Control	■	■	
Runoff Reduction	■		
Flow Diversion			

Description: A filter strip is a natural system that uses plants to filter stormwater runoff. The vegetated filter strip (VFS) serves as a perimeter buffer zone and a last defense between an area needing protection, such as a street or water body (stream, wetland, lake, etc.), and the adjacent property. Its primary purpose is to remove sediment and other pollutants from runoff water by filtration, deposition, infiltration, absorption, and vegetative uptake.

A VFS relies on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from construction site runoff. There can be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils while contained within the filter strip. Sheet flow must be maintained across the entire filter strip for it to maintain its effectiveness.

Typical Uses: As a sediment control practice, a VFS installed specifically during construction is typically used to prevent sediment from moving into/onto adjacent property. It is intended to keep downstream areas free of sediment.

Advantages:
- Natural measure to remove coarse sediment.
- Reasonably low construction costs.
- Filter strips are a low-maintenance practice, but maintenance increases as sediment volumes increase.
- May be able to utilize existing vegetation as a treatment device.
- Provides an aesthetically pleasing appearance.

Limitations:
- Are not intended to treat concentrated flow.
- The disturbed area draining to the vegetated strip should have slopes of 6% or less.
- Requires more land area than other sediment control practices.

Longevity: Permanent

SUDAS Specifications: Refer to Section 9010 (Seeding) or 9020 (Sodding)

A. Description/Uses

Vegetative Filter Strips (VFS) are densely vegetated strips of land (typically installed with sod-forming grasses) with a uniform slope to maintain sheet flow, which are designed to treat runoff and remove pollutants through vegetative filtering and infiltration during the construction phase. A VFS is located along the length of the downslope edge of the entire disturbed area to treat the runoff.

A VFS relies on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from construction site runoff. There can be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils while contained within the filter strip. To be effective, sheet flow must be maintained across the entire filter strip.

B. Design Considerations

Because of the large area required to provide adequate treatment, filter strips are generally used to treat small drainage areas (less than 5 acres). However, larger sites that have significant undisturbed vegetation or sufficient area for construction of a VFS may be well suited for accommodating a vegetated filter strip.

The size of the filter strip depends on the drainage area and the filter strip slope. Flow must enter the filter strip as sheet flow and spread out over the width of the strip. It is desirable to keep flow depth across the strip below a 1/2 inch in order to maintain sheet flow.

A level spreader may be required at the upstream end of the strip to dissipate concentrated flows and ensure sheet flow across the strip. Level spreaders or filter berms should be constructed perpendicular to the slope every 100 feet for slopes less than 5% and every 50 feet for slopes greater than 5% to prevent concentrated flows from forming.

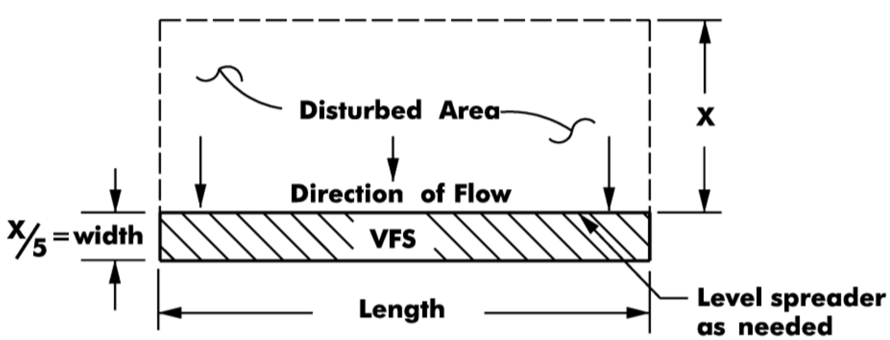
It should be noted that existing vegetation along a channel or drainage way can be used as a VFS. By simply protecting this vegetation during construction, an inexpensive VFS may be created.

C. Application

There are three different approaches to determining the size of a filter strip:

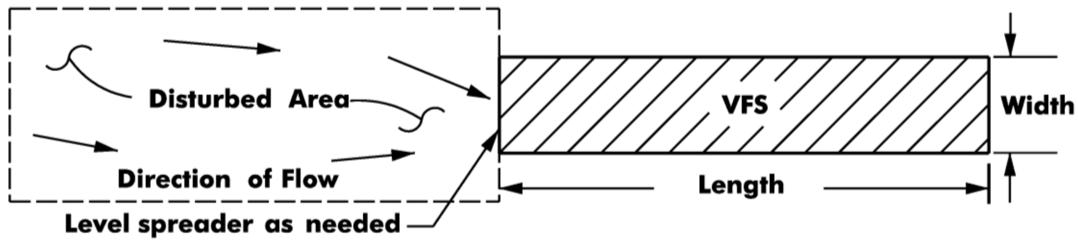
1. Adjacent to the disturbed ground, a VFS with a minimum width of 25 feet is adequate for treating runoff from disturbed areas up to 125 feet in width. For larger disturbed areas, the width of the VFS should be increased by 1 foot for every 5 feet beyond the 125 foot limit.

Figure 7E-26.01: Vegetated Filter Strip Adjacent to Disturbed Area



2. When the VFS is not completely adjacent to the site, it should be sized to be a minimum of one-half the area of the disturbed ground (i.e. Length X Width = 1/2 X Disturbed Area). The width of the VFS should be sized according to Manning's equation (refer to Chapter 2 - Stormwater, to limit the flow depth to a 1/2 inch or less (a minimum of width of 25 feet should be provided).

Figure 7E-26.02: Vegetated Filter Strip Downstream of Disturbed Ground



3. The VFS may also be designed to provide a 20 minute travel time (contact time) for runoff. This level of contact time with the vegetation is able to achieve 85% removal of Total Suspended Solids (TSS) (US-EPA, 1980).

D. Maintenance

Vegetated filter strips should be protected from vehicular traffic and construction equipment. The stand of vegetation should be maintained at a height of 3 to 6 inches. Unwanted weeds, brush, and trees should be controlled.

Vegetated filter strips require regular inspection to ensure proper distribution of flows, examine for signs of rill formation, and check for and remove accumulated sediment.

Rock Chutes and Flumes

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Rock chutes and flumes are devices used to convey concentrated flows down an embankment or slope to a lower level without causing erosion.

Typical Uses: Commonly used as a permanent feature at the release point where runoff enters a ditch, stream, or lake. They are also used as a temporary measure to stabilize the inlet slope to a sediment trap or basin.

Advantages:
- Stabilizes slopes and areas where high flow volumes occur.
- Prevents further erosion at entrance to sediment removal devices, reducing the required cleanout frequency.

Limitations:
- May not be considered aesthetically pleasing for permanent installations.
- May be a relatively expensive measure for temporary structures.
- Requires careful construction practices.
- Difficult to maintain level, especially through freeze-thaw cycles.

Longevity: Permanent

SUDAS Specifications: Refer to Section 9040, 2.09 and 3.13

A. Description/Uses

Rock chutes are devices used to stabilize the inlet slopes to sediment traps, sediment basins, rivers, ponds, lakes, and other drainage structures. The chutes consist of a rock-lined channel constructed on a steep slope.

Proper construction of the rock chute is imperative to its performance. The chute must be carefully notched into the ground to the thickness of the rock, to ensure positive drainage into the chute from the edges. If drainage into the chute from the edges is not provided, runoff will flow along the top of the chute, creating the potential for scouring under the chute.

After constructing the chute to the appropriate cross-section, a layer of engineering fabric is usually placed to protect the underlying soils. Crushed stone of the size or weight specified is then placed over the fabric, creating a stable surface to transport large flows down steep grades.

B. Design Considerations

The design of a rock chute is dependent on several factors including: the steepness of the slope; the shape of the channel; the volume and velocity of the water; the size of the rip rap material; and the downstream tailwater.

In order to simplify the process of designing and sizing a rock chute, a spreadsheet has been developed by the Iowa Division of the National Resource Conservation Service (NRCS). This spreadsheet is available on the internet and may be accessed from the following address: http://www.nrcs.usda.gov/wps/portal/nrcs/main/ia/technical/engineering/.

For permanent structures, an articulated or modular block system may also be considered. These products may be more aesthetically pleasing than a rock chute. Many can be vegetated to hide or mask the underlying armoring. Design information for these products is available from their respective manufacturers.

Installation of a turf reinforcement mat (TRM) might also be considered as an alternative to a rock chute (see Section 7E-18)

C. Application

Rock chutes should be considered at all locations where an elevation drop may create flow velocities that exceed the ability of the existing ground surface (bare or vegetated) to prevent erosion.

D. Maintenance

If designed and installed properly, maintenance of rock chutes is normally minimal. If the chute is left over a winter, it should be inspected in the spring to ensure that it is level. Any movement caused by freeze-thaw should be corrected.

Flocculents

Source: Applied Polymer Systems, 2006

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control			
Runoff Reduction			
Flow Diversion			

Description: Flocculents are natural materials or a class of chemicals that cause colloidal particles (clay) to coagulate. The coagulated particles group together to form flocs that will settle out of detained stormwater.

Typical Uses: Used in conjunction with sediment basins and sediment traps to remove suspended clay and fine silt particles from stormwater runoff prior to discharge.

Advantages:

- Ability to remove fine particles that would not settle out otherwise.
- Increases the percentage of fines removed during the detention period.
- May be used to remove suspended particles during dewatering operations.

Limitations:

- Requires specific dosing of the appropriate flocculent to achieve proper sedimentation.
- Flocculent must be thoroughly mixed with the stormwater.
- Flocculated particles must still be allowed to settle which takes time.
- Some flocculents are considered chemical pollutants. When these are used, the discharge must be carefully monitored to ensure that flocculent is adequately removed by settling.
- Flocculated material must be removed upon completion of the project for basins that are to be converted into permanent structures.

Longevity: Only effective on the runoff volume they are applied to; no long-term benefits

SUDAS Specifications: Typically, flocculents are only used in special circumstances and therefore have not been included in SUDAS Specifications

A. Description/Uses

Even with the proper sediment controls in place, suspended clay and loess particles are difficult to remove. Water ponded for days, or even weeks, can remain murky due to the suspension of these fine particles. Flocculents aid in removing these fine particles and may be a desirable treatment method for locations with clay or loess soils that are upstream of lakes, ponds, or other sensitive waterways.

B. Design Considerations

Fine soil particles, such as loess and clay particles, are difficult to remove with conventional settling techniques (basins, traps, etc.). The colloidal particles contain a negative electrostatic surface. Particles with like charges repel each other, preventing them from sticking together and settling out. This allows these small particles to remain in suspension indefinitely.

Coagulants are a class of chemicals that may be added to turbid stormwater to aid in the removal of suspended colloidal particles. Negatively-charged soil particles are attracted to the positively-charged coagulant particles. These particles stick together and form a larger, neutrally-charged particle called a floc. Since the colloidal particle forms a neutrally-charged floc, it no longer repels other particles, and can combine with other floc particles. The process of combining flocs into larger flocs that can be settled out of suspension is called flocculation. While the class of chemicals that ultimately cause the process of flocculation are technically called coagulants, the term flocculents has been adopted and is more widely used within the industry.

One flocculent that has been commonly used for stormwater applications is Polyacrylamide (PAM). Two versions of PAM are available, cationic and anionic. Only anionic PAM should be used, as cationic PAM is considered highly toxic. Anionic PAM has been used for many years in the water and wastewater industry and is considered safe for humans and aquatic life when used at the recommended rates.

Chitosan is another flocculent that is derived from the exoskeletons of crustaceans. It is generally considered safe for use in stormwater and water bodies.

A variety of other flocculent materials are available. Since trace amounts of flocculent will undoubtedly be discharged, the product used should be non-toxic and safe for both human and aquatic life and should not create Biochemical Oxygen Demand (BOD) problems in the downstream discharge waters.

Selection of an appropriate flocculent is highly dependent on the soil particle type and concentration. Analysis of a sample of the contaminated water is usually required to select the proper product and application rate. Manufacturers of these products will normally assist in this process.

C. Application

Several different methods of delivery are available for the application of flocculents to stormwater runoff. The most basic involves a solid form of the flocculent, either in block or pellet form, that is placed in a wire basket or mesh screen within the runoff as it flows into the sediment basin. The flowing water slowly dissolves the material, releasing flocculent into the basin.

More advanced methods involve equipment that will inject a liquid form of the flocculent into the runoff stream or storm sewer pipe at the desired rate.

Portable equipment that treats and filters the runoff is also available. Contaminated runoff is pumped from a sediment basin, treated with a flocculent, and then passed through sand filters to remove the suspended solids. The treated water may then be discharged.

Regardless of the flocculent material or method used, the material should never be added directly to the sediment basin or any standing water, unless adequate agitation is provided. The flocculent product should be introduced well in advance of the sediment control structure to allow for adequate mixing before the runoff arrives in the structure.

Adequate facilities need to be provided to allow for the settlement of the flocculated particles. Normally, a properly designed sediment basin is sufficient for this application. The accumulated material should be removed and disposed of properly.

D. Maintenance

Sediment should be removed on a routine basis to ensure a volume to receive the sediment. Timing will be based on having ample storage volume to accommodate anticipated runoff. Retention time and sediment storage volume are critical.

E. Time of Year

The effectiveness of the flocculent can be affected by temperature. The manufacturer should account for this when providing specific product and dosing rate recommendations.

Flotation Silt Curtain

BENEFITS

	L	M	H
Flow Control			
Erosion Control			
Sediment Control	■	■	
Runoff Reduction			
Flow Diversion			

Description: A flotation silt curtain (also called a turbidity curtain) consists of a geosynthetic fabric that is suspended vertically in a body of water. The top of the curtain is attached to floats, and the bottom is weighted.

Typical Uses: Flotation silt curtains are used when construction occurs in a water body or along a stream bank or shoreline. Flotation silt curtains prevent sediment, which is stirred up during construction, from migrating out of the work area and into the rest of the water body.

Advantages:

- Allows for containment of sediment-laden water within a water body.
- Protects contained water from turbulence, allowing particles to fall out of suspension.

Limitations:

- Limited to use only in areas where other erosion and sediment control practices cannot be used.
- Cannot stop the flow of a significant amount of water.
- Must not be used to filter entire stream flow.
- Difficult to remove fine silt and clay particles.

Longevity: One construction season (do not leave in place during winter)

SUDAS Specifications: Typically, flotation silt curtains are only used in special circumstances and therefore have not been included in SUDAS Specifications

A. Description/Uses

A flotation silt curtain, also called a turbidity curtain, consists of a heavy geosynthetic fabric that is suspended vertically in a water body, with floats at the top, and weights at the bottom. The purpose of the curtain is to act as a divider, preventing sediment laden-water from migrating to the rest of the water body.

Flotation silt curtains are commonly used when construction is required near or within a water body, where other erosion and sediment control practices cannot be used. This may include dredging operations, stream bank improvements, bridge pier construction, etc.

B. Design Considerations

For ponds or other relatively still water bodies, which do not have significant inflow into the containment area, the flotation silt curtain consists of a relatively impermeable membrane that provides a barrier between clean water and sediment-laden water. The barrier creates a containment basin, in which sediment is trapped and allowed to fall out of suspension. Runoff into this type of curtain should be minimized, as the available volume is limited.

For situations that have moving water, such as lakes or streams, a provision must be made to allow water to flow through the curtain. This is normally accomplished by constructing part of the curtain from heavy filter fabric. The filter fabric allows water to pass through the curtain, maintaining equilibrium, but retaining sediment particles. While these curtains are designed to allow for some water movement, they do not have high flow-through rates, and should not be installed across a channel. When used in a stream, channel, or other body of moving water, the flotation silt curtains must be placed parallel to the direction of flow.

Unless the water body is subject to wind or wave actions, the curtain should extend the entire depth of the water, and rest on the bottom. The weighted bottom of the curtain needs to maintain contact with the bottom of the water body in order to keep sediment from flowing under the curtain. In order to do this, enough slack must be provided to allow the curtain to rise and fall as the depth of the water varies, without breaking contact with the bottom of the water body.

In situations where there is significant wind or wave action, the weighted end of the curtain should not extend to the bottom of the water body. Wind/wave action on the flotation system can cause movement of the lower end of the curtain, causing it to rub against the bottom, stirring up additional sediment. In these situations, a minimum 1 foot gap should be provided between the lower end of the curtain and the bottom of the water body. In addition, it is not practical to extend the curtain deeper than 10 or 12 feet. Deeper installations can be affected by the moving water, stressing the material, and causing the bottom of the curtain to be pushed around, billowing up toward the surface.

When determining the required length of the flotation silt curtain, an additional 10 to 20% should be included over the straight-line measurements. This allows for easier installation and reduces stresses caused by high winds and wave action.

Once the curtain has been positioned within the water body, the top is held in place by connecting it to anchors that are installed at regular intervals. The ends of the curtain (both upper and lower) should be extended to the shoreline, and anchored to a stable object, such as a tree.

C. Application

Flotation silt curtains are divided into three types, Type I, Type II, and Type III, based upon the flow conditions within the water body. The information provided here applies to minimal and moderate flow conditions, where the velocity of flow is 5 feet per second or less. For situations where the flow is greater than this, additional investigation is required, and a qualified manufacturer should be consulted.

The three types of silt curtains are differentiated by the strength and flow through rate of the fabric, and the strength of the connecting materials used:

1. Type I curtains are considered light-duty and are intended for areas where there is no current, and where the area is protected from wind and wave action.

2. Type II curtains can be used in areas with moderate running current (up to 3.5 fps), or where wind and water currents can affect the curtain.

3. Type III curtains are used in areas with considerable current (up to 5 fps), or where the curtain is subject to more severe wind and wave action.

D. Maintenance

A decision must be made on how to handle the accumulated sediment. Unless the accumulation is significant, consideration should be given to leaving this sediment in place. The process of removing the sediment can re-suspend the particles. Regardless of whether or not the accumulated sediment is removed, suspended sediment should always be allowed to settle for a minimum of 24 hours prior to removal of the silt curtain.

Once they are suspended in the water, clay and silt particles are difficult to remove by settling methods alone. For waters contaminated with clay or fine silts, the addition of a flocculent to the containment area may be considered prior to removal of the silt curtain. Care must be taken when selecting a flocculent as some are detrimental to water bodies and should not be used. See Section 7E-6 - Wattles for additional information on flocculents.

E. Time of Year

Sediment curtains should not be left in place during winter months, as ice can cause the curtain to rip or be torn from its shoreline supports.

Appendix

Before construction can begin on a site the following steps must be taken to be in compliance with the Iowa DNR General Permit No.2:

- A Stormwater Pollution Prevention Plan must be created for the site
- A Notice of Intent (NOI) must be completed by the operator of the construction site and this document along with public notices must be submitted to the Iowa DNR.
- A signed affidavit must be filed with the local Soil and Water Conservation District stating that the project will not exceed the soil loss limits stated.
- A Letter of Authorization is provided to the Operator of the construction site, upon approval of the NOI by Iowa DNR.
- SWPPP review and approval is required by MS-4 cities prior to construction.
- Necessary best management practices should be in place prior to construction.
- Construction can then begin.

Copies of the NPDES stormwater permitting guidance and application forms can be found on Iowa DNR's website.

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CHAPTER 8
Parking Lots

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Chapter 8 - Parking Lots

8A General Information

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General Information

A. General

This chapter provides design criteria for off-street parking lots. These criteria include recommendations for the design of entrances and exits, vehicle circulation path, parking space dimensions, pavement thickness, etc. This chapter also includes site requirements for items such as number of parking spaces, landscaping, parking setback, etc.

While most jurisdictions have their own parking ordinance covering these items, they are included here as guidance for those communities that do not have a parking ordinance. This information may also be used as a supplement to existing parking ordinances.

B. References

The design for parking lot facilities should comply with the current edition of the following:

Urban Land Institute (ULI) & National Parking Association (NPA), *The Dimensions of Parking*

US Department of Justice. *2010 ADA Standards for Accessible Design.*

Layout and Design

A. Parking Lot Access

Properly designed parking lot access provides for safe and efficient movement of vehicles into and out of the parking lot. Refer to Chapter 5 - Roadway Design for additional information on access management and driveway design, spacing, and location selection.

The most efficient approach to designing parking lot access places a priority on moving inbound traffic from the public roadway into the facility. Entrances should be located on major streets, align with interior traffic lanes/aisles, and direct inbound traffic toward the destination. Traffic control within the lot should provide inbound traffic the right-of-way. Favoring inbound traffic expedites the rapid movement of vehicles from the street into the facility and prevents vehicles from lining up on public roadways. Where a high volume parking lot is adjacent to a high volume or high speed roadway, a dedicated deceleration/turning lane at the entrance helps eliminate rear-end accidents.

Exits should be located away from the destination point and discharge vehicles onto lower volume adjacent side streets if possible. Since exiting traffic tends to move more slowly, drivers can more comfortably navigate the turns required to reach the exit. Vehicles queued to exit the parking lot will stack up inside the lot and will not affect traffic on the public street.

Where separate entrances and exits cannot be provided, the driveway to the parking lot should be at least 24 feet wide to provide two 12 foot lanes.

Traffic studies may be required for entrances to large retail centers, event facilities, or businesses with large numbers of employees entering or exiting the lot at the beginning or end of a work day or shift.

B. Parking Lot Circulation

Off-street parking lots should be designed to accommodate traffic volumes and pedestrian circulation based on the land use served. The use of islands, medians, curbing, and landscaping is encouraged to separate parking spaces from traffic and pedestrian circulation areas.

Parking spaces at entrance and exit points should be terminated (except at one and two family dwelling units) to prevent conflict between vehicles attempting to enter or exit the parking space, and vehicles attempting to enter or exit the parking lot.

Access between adjacent commercial parking lots should be considered. This allows patrons to travel from one business to the adjacent business without entering the public street and then turning immediately into the next parking area. These types of movements can cause operational problems on the public street.

C. Parking Lot Dimensions

1. **Parking Spaces:** In order to determine parking space sizes, the design vehicle size must be defined. Since 1999, the size of the 85th percentile vehicle on the road has varied slightly, but has remained within an inch or two of 6 feet, 7 inches wide by 17 feet, 3 inches long.

   In addition to vehicle size, the designer must consider the intended function of the parking facility. For example, facilities with high turnover rates, such as convenience stores, should have greater clearances than those with low turnover rates. In addition, where a significant portion of users may be elderly, such as at hospitals, larger dimensions may be appropriate.

   Parking spaces that provide sufficient clearance for doors to be opened and occupants to enter and exit will also provide adequate width for maneuvering if the adjacent aisle is wide enough. Door opening clearances should range from 23 inches in low turnover facilities to 27 inches in high turnover facilities. Table 8B-1.01 lists recommended parking stall widths on the basis of turnover.

   Table 8B-1.01: Recommended Minimum Widths for Parking Stalls

Facility Type	Width
Low turnover (employees, students, etc)	8’-6”
Moderate to high turnover (retail, medical facilities, etc.)	9’-0”

   Source: Urban Land Institute, National Parking Association

   For stalls that are adjacent to walls, curbs, islands, or other obstructions, increase the stall width by at least 12 inches to allow for door opening and to reduce the risk of tripping.

   Unlike width, the length of a parking space is not affected by turnover rate or user type. The recommended length of a parking space is 18 feet. The length of the parking space may be modified up to 2 feet, if vehicle overhang is allowed. However, the designer should be aware that the aerodynamic design of many current vehicles often does not provide sufficient vertical clearance for vehicles to pull forward over the curb.

2. **Parking Module Design:** The drive aisle is the space between two parking stalls directly across from one another. The term “module” refers to the width of the drive aisle combined with the length of the parking stalls on one or both sides of the drive aisle. Table 8B-1.02 lists recommended minimum dimensions for parking facilities. Figure 8B-1.01 provides further definition of the terms used in Table 8B-1.02.

   The only dimension that varies by stall width is the interlock dimension. An interlock occurs with angled parking when two stalls in adjacent modules align. The overlap at the front of the stalls is the interlock dimension. When a parking facility is designed to take advantage of interlock, the effective width of the module may be reduced by the interlock dimension. For aisles with interlocking spaces on both sides, the effective width of the module may be reduced by two times the interlock distance. This approach can provide a more efficient parking lot facility and reduce the overall surface area required for the parking lot.

   Because snow can obscure pavement markings, vehicles will often pull too far into a parking space, which reduces the width of the aisle in the adjacent module. This has been taken into consideration in Table 8B-1.02. Therefore, when a curb, wall, or other physical restraint is provided for on at least 30% of the stalls, the aisle width (and therefore the overall module width) may be reduced by 1 foot.

Table 8B-1.02: Minimum Parking Dimensions

Parking Lot Dimension				Parking Angle (θ)				
				Two-way Aisle			**One-way Aisle**	
				90°	**60°**	**45°**	**60°**	**45°**
Stall Projection			SP	18’-0”	15’-7”	12’-9”	15’-7”	12’-9”
Aisle Width			A	24’-0”	25’-10”	29’-8”	20’-4”	21’-6”
Base Module			M_1	60’-0”	57’-0”	55’-2”	51’-6”	47’-0”
Single Loaded Module			M_2	42’-0”	39’-0”	37’-7”	32’-6”	29’-5”
Wall to Interlock			M_3	60’-0”	55’-10”	52’-2”	49’-4”	44’-0”
Interlock to Interlock			M_4	60’-0”	53’-8”	49’-2”	47’-2”	41’-0”
Overhang			o	2’-6”	2’-2”	1’-9”	2’-2”	1’-9”
Stall Width	*8’-6”*	Width Projection	WP	8’-6”	9’-10”	12’-0”	9’-10”	12’-0”
		Interlock	i	0’-0”	2’-2”	3’-0”	2’-2”	3’-0”
	9’-0”	Width Projection	WP	9’-0”	10’-5”	12’-9”	10’-5”	12’-9”
		Interlock	i	0’-0”	2’-3”	3’-2”	2’-3”	3’-2”

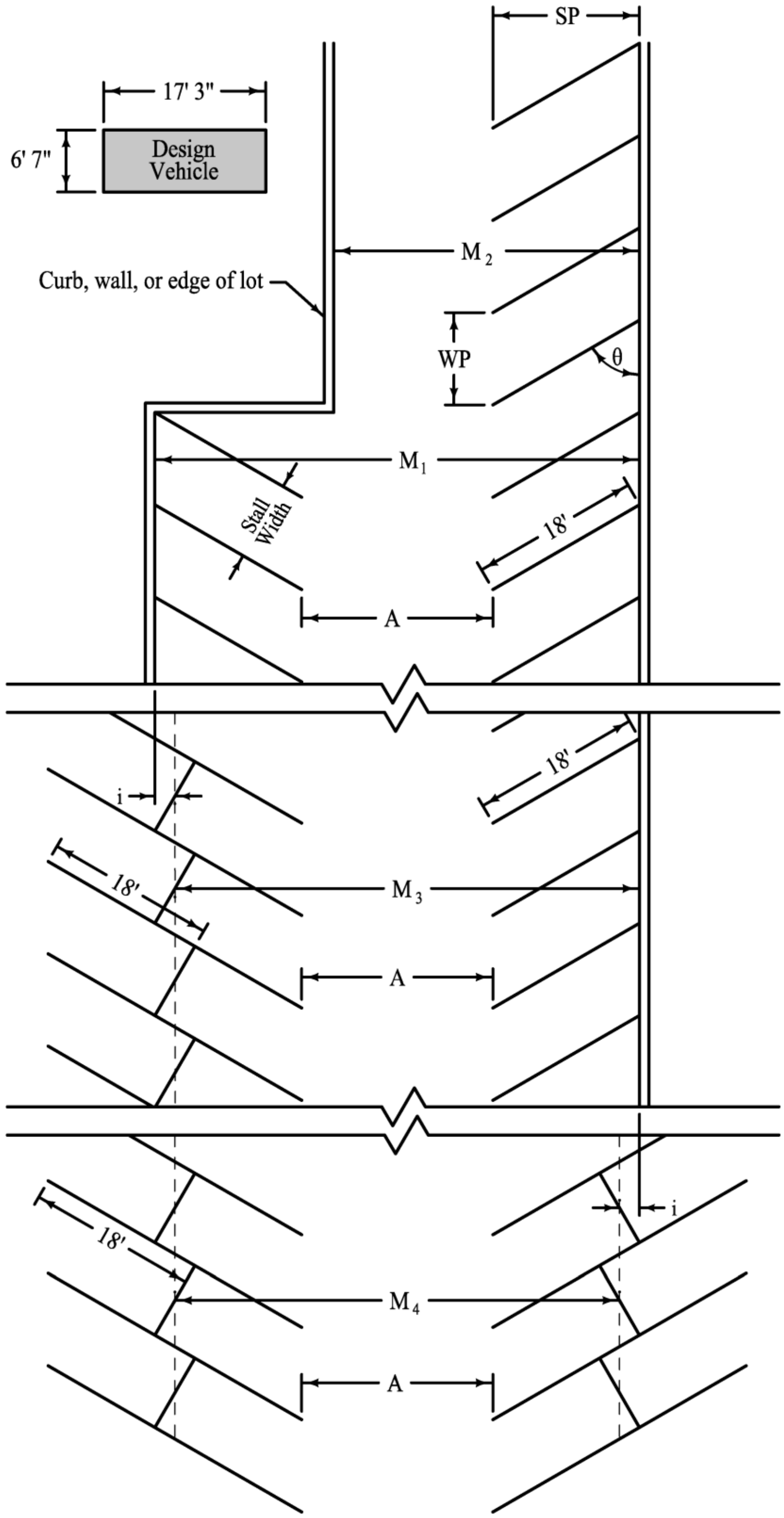
Notes:
1. Aisle width may be increased up to 3 feet to provide a higher level of comfort.
2. In lots where at least 30% of stalls have curbs, aisle width may be reduced by 1’-0”.
3. Light poles and columns may protrude a maximum of 2 feet into a parking module as long as they do not encroach on more than 30% of the stalls. When more than 30% of the stalls are encroached, interlock reductions cannot be taken.
4. For additional parking angles, refer to The Dimensions of Parking, ULI, NPA

Source: Adapted from Urban Land Institute, National Parking Association

Perpendicular parking provides the greatest number of parking spaces for a given a length of aisle. One-way angled parking provides fewer spaces than perpendicular for the same length of aisle, but has the advantage of a narrower drive aisle. Because of this, the surface area per parking space for perpendicular and angled one-way parking is approximately equal.

Two-way angled parking is also allowable and can be useful in certain situations; however, it is a less efficient design than two-way perpendicular or one-way angled parking. Two-way angled parking cannot take full advantage of the narrower drive aisle, requiring approximately 10% to 15% more area per parking space than perpendicular or one-way angled parking.

Figure 8B-1.01: Parking Dimensions



SP = Stall Projection		M_1 = Base Module	(2SP + A)
A = Aisle Width		M_2 = Single Loaded Module	(SP + A)
WP = Width Projection		M_3 = Wall to Interlock	(M_1 – i)
i = Interlock		M_4 = Interlock to Interlock	(M_1 – 2i)

3. **Compact Parking:** It is no longer recommended that compact car only spaces be provided. At the time when compact car parking spaces were introduced, the mix of automobiles consisted of clearly defined very large and very small vehicles. As a result, the use of compact parking only was largely self enforcing; however, the current mix of automobile sizes is much more diverse. There is no longer a clear definition among the public of what constitutes a compact vehicle. In addition, if a compact car space is available in a convenient location, many drivers of intermediate and large vehicles will attempt to utilize the space, encroaching into the adjacent space. This creates a domino effect down the row and eventually renders a parking space unusable. For these reasons, compact car only spaces are not recommended.

D. Accessibility Requirements

Accessible parking spaces must be provided according to the 2010 ADA Standards for Accessible Design (2010 Standards). In addition, certain facilities are required to provide accessible passenger loading zones. The 2010 Standards identify both the minimum dimensions and the minimum number of accessible parking spaces and loading zones required. Refer to Parts 502 and 503 of the 2010 ADA Standards for additional information.

1. **Accessible Parking Spaces:** The 2010 Standards identify two types of accessible parking spaces for vehicles - car and van-accessible parking spaces. The minimum dimensions and common requirements for each are provided below and in Figure 8B-1.02.

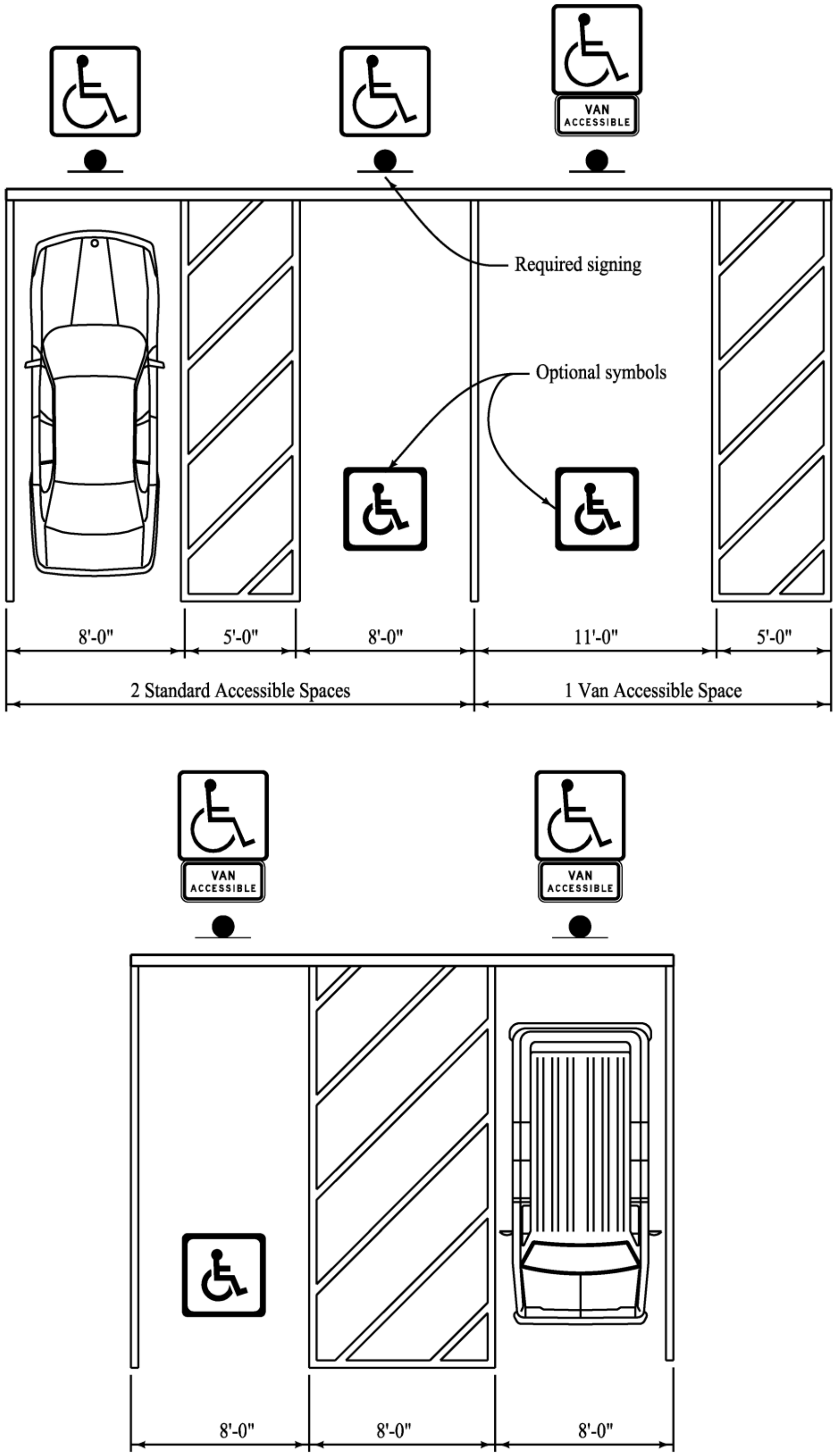
   a. **Car Accessible Spaces:** Minimum width of 96 inches (8 feet 0 inches)

   b. **Van-accessible Spaces:** Minimum width of 132 inches (11 feet 0 inches)

   c. **Access Aisle:** An adjacent access aisle is required for both car and van-accessible spaces. Two parking spaces may share an individual access aisle.
      1) **Width:** The minimum width of the access aisle is 60 inches (5 feet 0 inches). If the width of the access aisle is increased to 96 inches, the width of an adjacent van-accessible parking space may be reduced from 132 inches to 96 inches. With proper layout, this allows for a reduction in the total width consumed by two adjacent van-accessible spaces.
      2) **Length:** The access aisle must extend the full length of the parking spaces they serve.
      3) **Marking:** The access aisle must be marked; however, the 2010 Standards do not indicate the type of pavement marking required. Typically, the aisle is striped at an angle. While not required, the adjacent stalls may be painted with the international symbol of accessibility (wheelchair symbol) to aid motorist in identifying the space as being reserved.

   d. **Signing:** Accessible parking spaces must be designated with signs showing the international symbol of accessibility. Signs for van accessible spaces should also contain the designation "van accessible." Signs must be installed a minimum of 60 inches from the bottom of the sign to the ground surface. Additional signage related to enforcement or parking fines is not required by ADA.

Figure 8B-1.02: Accessible Space Dimensions

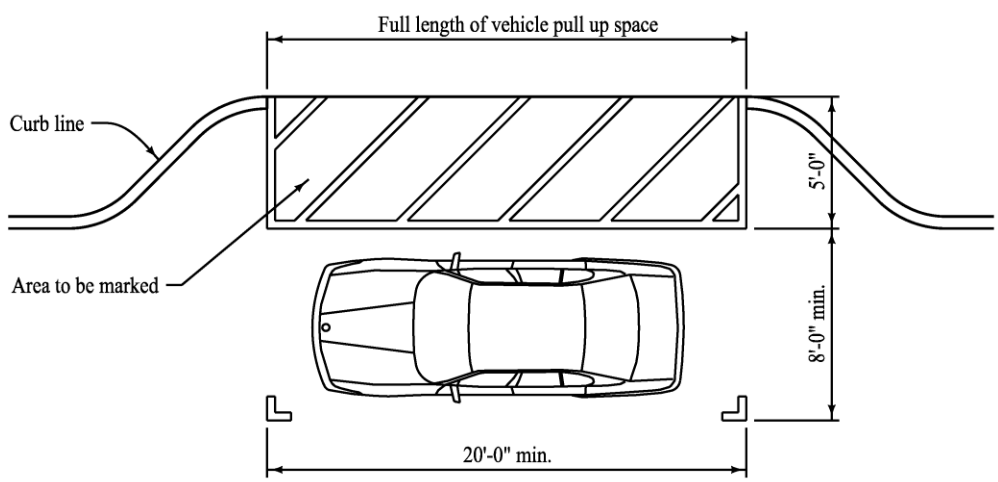


Alternate Van Accessible Parking Dimensions

2. **Passenger Loading Zone:** The 2010 ADA Standards require passenger loading zones only at licensed medical care and long-term care facilities (where the period of stay exceeds 24 hours). At other locations, the provision of passenger loading zones is optional; however, when they are provided, a portion of the loading zone must be accessible. At least one accessible passenger loading zone must be provided for every 100 continuous linear feet of loading zone space.

   Passenger loading zones must have a minimum pull-up length of 20 feet and a width of 96 inches. An access aisle adjacent to the loading zone must extend the full width of the vehicle pull up space they serve and have a minimum width of 60 inches. The access aisle must be at the same elevation as the vehicle pull-up spaces that serve them. The loading zone cannot discharge to a sidewalk on top of a curb. In addition, the access aisle must be marked to discourage parking. This is typically accomplished by striping at an angle.

Figure 8B-1.03: Passenger Loading Zone Dimensions



3. **Access Routes:** At least one accessible route must connect the building or destination with each accessible parking space or loading zone. To the maximum extent possible, the accessible route should coincide with the route for the general public. Like accessible off-street parking spaces and loading zones, accessible routes are covered by the 2010 ADA Standards. The basic requirements that apply to new construction for accessibility from a parking lot to a building or other destination are summarized in Chapter 4 of the 2010 Standards.

4. **On-Street Parking:** For requirements on accessibility for on-street marked or metered parking spaces, see Section 12A-2.

E. Drainage

Internal parking lot drainage should be designed according to Chapter 2 - Stormwater.

Stormwater runoff from parking lots serving other than single and two family dwellings should not be discharged directly into the street; such runoff should be collected internally or discharged to an adjacent drainage way. After providing detention, when required, the collected stormwater may be discharged to the public storm sewer, ditch, or other conveyance. Stormwater runoff discharged to the street over the back of the curb or through a parking lot entrance, should be minimized. Check with the local jurisdiction for their stormwater requirements.

Where narrow (less than 10 feet wide) raised islands are provided, their presence should generally be disregarded when determining the runoff coefficient or curve number for the parking lot as they provide little benefit in reducing runoff. Wider islands, or islands that are depressed to collect stormwater runoff, are encouraged and may be taken into consideration when determining the runoff potential.

Pavement slopes of 1.5% should be provided to ensure proper drainage and eliminate standing water and icy conditions. Minimum pavement slopes of 0.6% may be used, however since the potential for flat areas is greater, additional measures to address drainage, such as slotted drains or pervious pavement, may be necessary. Slopes greater than 2% in areas between the parking lot destination and the accessible parking stalls should be avoided as they create a situation where constructing an accessible route is difficult. Slopes greater than 5% are discouraged.

F. Pavement Design

Any off-street parking area should be surfaced with a flexible or rigid pavement. Check with the local jurisdiction to determine the requirements for paving parking lots. If no local requirements are stipulated, the pavement thickness for parking areas occupied by cars and small trucks for rigid and flexible pavements (see Chapter 5 - Roadway Design for mix designs) should be designed according to the following tables. It should be noted that the layer of aggregate used as the subbase needs to be drainable.

Parking lots should be designed for a minimum 20 year design life. If a design life of greater or less than 20 years is desired, see Chapter 5 - Roadway Design for pavement thickness determination. In addition, for pavements less than the recommended thickness, a pavement thickness determination should be completed to match the pavement structure with the needs of the project.

The subgrade should be designed according to Section 6E-1. If soils tests are not available to determine the CBR value and uniformity of the soil (before and after construction), a CBR value of 3 and a non-uniform subgrade should be assumed.

Table 8B-1.03: Pavement Thickness for Light Loads
(Parking lots with 200 or less cars/day and/or 2 or less trucks/day or equivalent axle loads)

Subgrade CBR	**Surface Material**	**On 12”of Prepared Subgrade**		**On 12” of Prepared Subgrade with 4” Granular Subbase**	
		Minimum	***Desirable***	***Minimum***	***Desirable***
9	Rigid	5”	6”	4”	5”
	Flexible	5”	6”	4”	5”
6	Rigid	5”	6”	4”	5”
	Flexible	5”	6”	4”	5”
3	Rigid	5”	6”	4”	5”
	Flexible	6”	6”	5”	5”

Table 8B-1.04: Pavement Thickness for Moderate Loads
(Parking areas, entrances, perimeter travel lanes, and frontage roads subject to 201 to 700 cars/day and/or 3 to 50 trucks/day or equivalent axle loads)

Subgrade CBR	**Surface Material**	**On 12"of Prepared Subgrade**		**On 12" of Prepared Subgrade with Granular Subbase**		
		Minimum	***Desirable***	***Thickness of Granular Subbase***	***Minimum***	***Desirable***
9	Rigid	5"	6"	4"	4"	5"
	Flexible	5"	6"	6"	4"	5"
6	Rigid	5"	6"	6"	4.5"	5"
	Flexible	6"	6"	8"	5"	5"
3	Rigid	5.5"	6"	6"	5"	5"
	Flexible	6"	7"	8"	6"	6"

The portions of the parking facility serving truck traffic such as entrances, perimeter travel lanes, trash dumpster sites, and delivery truck routes must be designed to accommodate heavier loads. The number, type, and weight of delivery vehicles can usually be predicted with a fair level of accuracy. With this information, ESAL values and pavement thicknesses can be determined using the methodology described in Chapter 5 - Roadway Design.

If the parking lot is to service an industrial area, such as a truck stop or manufacturing facility, the volume of truck traffic and the associated ESALs should be determined and an independent pavement thickness determination completed to ensure meeting the 20 year design life needs of the project.

Site Provisions

A. General

This section provides design criteria for site requirements such as number of parking spaces, landscaping, parking setback, etc. While most jurisdictions have their own parking and zoning ordinances covering these items, they are included here as guidance for those communities that do not have such ordinances. This information may also be used as a supplement to existing ordinances.

B. Number of Parking Spaces Required

1. **General Parking Ratios:** Adequate off-street parking should be provided for all residential, commercial, industrial, and public use properties. Table 8C-1.01 below provides minimum parking space standards for common land uses. For large traffic generators, a specific parking study should be completed.

Table 8C-1.01: Parking Ratios

Land Use	Spaces per Unit
Residential and Lodging	
Single and two family dwellings	2.0 / unit (tandem parking allowed)
Row dwellings	2.0 / unit + 1 visitor space per 4 units
Multiple family (apartment and condo)	1 to 2 bedroom units: 2.0 spaces / unit 3+ bedroom units: 2.5 spaces per unit plus 3.0 / 1,000 sf of GFA for lease management
Mobile home park	2.0 / unit (tandem parking allowed) plus 1 visitor space per 10 units plus 3.0 / 1,000 sf of GFA for lease management
Housing for seniors	1.0 / unit plus 10 / 1,000 sf of GFA for multipurpose buildings
Hotel/motel	1.25 / room + 10 / 1,000 sf of GFA of lounge or restaurant + 20 / 1,000 sf of GFA of conference or banquet facilities
Retail Sales and Services	
General and convenience retail	2.75 / 1,000 sf of GFA
Grocery stores	6.75 / 1,000 sf of GFA
Heavy/hard goods	2.5 / 1,000 sf of GFA, including outdoor sales area
Discount superstores	5.5 / 1,000 sf of GFA, including outdoor sales area
Specialty superstores	4.5 / 1,000 sf of GFA, including outdoor sales area
Shopping center	Special parking study required
Food and Beverage Services	
Restaurant, sit down	10 / 1,000 sf of GFA
Restaurant, fast food	1.5 / 1,000 sf of GFA
Restaurant, take out only	2 / 1,000 sf of GFA
Bar/nightclub	10 / 1,000 sf of GFA
Office and Business Services	
< 25,000 sf of GFA	3.8 / 1,000 sf of GFA
25,000 to 100,000 sf of GFA	Scale between 3.8 and 3.4 / 1,000 sf of GFA
100,000 sf of GFA	3.4 / 1,000 sf of GFA
Consumer services	4.6 / 1,000 sf of GFA
Data processing and telemarketing	6.0 / 1,000 sf of GFA
Medical office (not part of hospital)	4.5 / 1,000 sf of GFA
Industrial, Storage, and Wholesale	
Manufacturing or industrial	1.85 / 1,000 sf of GFA, plus parking for office, sales, or similar use where those uses exceed 10% of sf of GFA
Storage or wholesale	0.67 / 1,000 sf of GFA
Mini-warehouse	1.75 / 1,000
Educational or Institutional Uses	
Elementary or middle schools	1.0 / employee + 10 spaces for visitors
High school	1.0 / employee + 0.3 / student
Church or theatre	0.4 / seat

GFA: Gross Floor Area means the area in square feet within the exterior walls of a building, exclusive of any area used for off-street parking, courtyards, or mechanical equipment.

Source: Adapted from ULI/NPA

2. **Accessible Parking Ratios:** When parking spaces are provided, a portion of the parking spaces must be made accessible according to the 2010 ADA Standards for Accessible Design (2010 Standards). These standards specify the number of parking spaces within a parking facility that must be accessible. Table 8C-1.02 summarizes the minimum accessible parking ratios. For additional information, refer to Part 208 of the 2010 Standards.

Table 8C-1.02: Minimum Accessible Parking Ratios

Total Number of Spaces Provided	Minimum Number of Accessible Spaces
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 to 300	7
301 to 400	8
401 to 500	9
501 to 1,000	2% of total
1,001 and over	20, plus 1 for each 100, or fraction thereof, over 1,000

a. **Residential Facilities:** Accessible parking requirements for residential facilities differ from the table above and are based, in part, on the number of accessible dwelling units provided. Refer to Part 208.2.3 of the 2010 Standards for specific requirements.

b. **Hospital Outpatient Facilities:** Ten percent of the patient and visitor parking spaces provided to serve hospital outpatient facilities shall be accessible (2010 Standards, 208.2.1).

c. **Rehabilitation Facilities and Outpatient Physical Therapy Facilities:** Twenty percent of patient and visitor parking spaces shall be accessible. Rehabilitation and outpatient physical therapy facilities serve patients with conditions affecting mobility such as braces, canes crutches, prosthetic devices, wheelchairs or powered mobility aids, arthritis, neurological or orthopedic conditions affecting one’s ability to walk, and respiratory or cardiac conditions that impose significant functional limitations (2010 Standards, 208.2.2).

d. **Van-accessible Spaces:** For every six accessible parking spaces, or fraction thereof, one van accessible parking space must be provided. If only one accessible parking space is required, it must be van-accessible. This requirement applies to all facility types.

C. Parking Lot Setback Requirements

Tables 8C-1.03 and 8C-1.04 present recommended parking lot setback distances.

Table 8C-1.03: Residential Parking Lot Setbacks

Residential Parking Lot Location	Setback (feet)
In all residential districts (from street right-of-way)	10 [2, 3, 4]
Along alley line across from a residential district	5 [1, 2, 3]
Along adjacent residential district property lines	10 [2, 3]
Along adjoining residential district parking lots	5 [2, 3]
Along adjacent commercial or industrial district property lines	0 [2, 3]

[1] No setback required when use as single family, duplex, or when the use is across from a parking lot
[2] Setback area should consist of a permeable material and should be landscaped
[3] No vehicle should encroach into a required setback
[4] Parking on driveways parallel to a public sidewalk for single family residences should maintain a minimum setback of 10 feet from the public street right-of-way

Table 8C-1.04: Commercial / Industrial Parking Lot Setbacks

Commercial/Industrial Parking Lot Location	Setback (feet)
Along alley lines bordering a residential district	5 [1, 2, 3]
Commercial or industrial districts abutting a residential district	10 [2, 3]
Commercial or industrial districts abutting a residential district parking lot	5 [2, 3]
Adjacent to a commercial or industrial district property line	0 [2, 3]
Office and commercial districts	15 [2, 3, 4]
Light industrial and general industrial districts	10 [2, 3, 4]
Business park and professional commerce park districts	20 [2, 3, 4]

[1] No setback required along that portion of an alley across from a residential parking lot
[2] Setback area should consist of a permeable material and should be landscaped
[3] No vehicle should encroach into a required setback
[4] Setback from public street right-of-way

All parking lots should provide a curb or wheel barrier around the entire perimeter, unless a walkway or border is provided. When adjacent to required setback and adjoining property lines, wheel barriers or curbs should be located 2 feet from the edge of property lines, public sidewalks, and adjacent parking lots to prevent vehicle encroachment into the setback area.

D. Landscaping and Screening

1. General Requirements:

a. Landscaping: The Designer should refer to the individual Jurisdiction zoning ordinance for parking lot landscaping requirements. If no such ordinance exists then the requirements set forth in this section should be used.

It is desired that all parking areas be aesthetically improved to reduce obtrusive characteristics that are inherent to their use. Therefore, wherever practical, such parking areas should be effectively screened from general public view by incorporating the natural landscape and topography. All parking areas should include landscape areas, islands, screens, etc., equal to not less than 10% of the total paved area. Landscaped islands within the parking area should be ground cover of grass (i.e. sod), shrubs, or other acceptable living plant life, unless an alternate ground cover is specifically approved as part of the site plan review by the Jurisdiction.

Landscape islands should not be less than a minimum of 8 feet in width from back of curb to back of curb, landscape planters a minimum 6 feet in diameter, and no parking space should be greater than 75 feet from a landscaped open space. Parking spaces should be separated from any adjoining roadway, by a landscaped island or elevated separation (i.e. sidewalk) of a minimum of 9 feet in width except along the roadway or parking bay aisle that provides the direct access.

Earthen berms should be a minimum of 3 feet above the top of curb of the adjoining parking lot, if applicable, or public thoroughfare; should be designed to not affect the drainage and sight distance of the surrounding area; and should be aesthetically pleasing to the general public. Berms may be required to be higher if the minimum height is identified during the development review process as being inadequate to provide effective screening and buffering.

b. Screening: Screening may consist of one or any combination of the following.

- Wood or masonry walls or fences
- Landscaped earthen berms
- Plant materials of such size, branching density, spacing, and quantity to provide a minimum of 60% opacity while dormant. Such materials should provide screening function within three growing seasons after the initial planting. Failure to accomplish such function, whether due to slow growth, death, or other reason, may be grounds for requiring the addition of wood or masonry walls or fences. In some Jurisdictions, a published list of approved materials may be available. Any changes to this list must be made by a certified landscape architect.

2. Additional Requirements for Parking Lots in Residential Districts:

a. A 6 foot high opaque screen should be installed and maintained when a residential parking area abuts another lower density residential district except in any required front yard setback area. No screening is required when said residential parking area abuts another parking area or a non-residential district.

b. A three foot high opaque screen may be installed and maintained along each alley and street line when the premises is located across the street or alley from any lower density residential district. No screening is required when said residential parking area is located across the street or alley from another parking area or from a non-residential district.

3. Additional Requirements for Parking Lots in Commercial and Industrial Districts:

a. A 6 foot high opaque screen should be installed and maintained when a commercial and industrial parking lot abuts a residential district. No screening is required when said commercial and industrial parking area abuts another parking area or a non-residential use.

b. A 3 foot high opaque screen should be installed and maintained along each alley and street line when the premises is located across the street or alley from any residential district. No screening is required when said commercial and industrial parking area is located across the street or alley from another parking area or from a non-residential district.

c. A 3 foot high opaque screen should be installed and maintained along adjoining residential district. No screening is required when adjoining use is non-residential or parking.

E. Lighting

When lighting is required, cutoff style luminaries should be utilized to minimize glare and prevent light trespass onto adjacent properties. Design lighting according to Chapter 11 - Street Lighting. Check with the local jurisdiction for lighting requirements. If none exist, refer to Table 8C-1.05 for the recommended illuminance values and uniformity ratios for parking lots.

Table 8C-1.05: Recommended Maintained Illuminance Values and Uniformity Ratios

	Basic	**Enhanced Security**
Minimum horizontal illuminance on surface	0.2 footcandles	0.5 footcandles
Minimum vertical illuminance at 5 feet above surface	0.1 footcandles	0.25 footcandles
Uniformity ratio (max. to min.)	20:1	15:1

Source: ULI / NPA

F. Pavement Markings

The location of each parking space and direction of traffic flow should be identified by surface markings and should be maintained so as to be readily visible at all times. In general, yellow markings tend to stand out better than white from the background parking surface. White paint on concrete also tends to fade with time, making it difficult to distinguish the striping. All pavement striping should be 4 inches in width. Markings may either be painted or cold applied marking tape.

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CHAPTER 9
Utilities

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General Information

A. General

Proper regulation of the location design and methods for installation, maintenance, and adjustment of private and public utilities in the roadway right-of-way is necessary for safety, public service, and orderly development. Utility lines should be located to minimize need for later adjustment, to accommodate future roadway improvements, and to allow servicing such lines with minimum interference to traffic and interruption of other utility services.

Longitudinal installations should be located on uniform alignment to provide a safe environment for traffic operation and preserve space for future roadway improvements or other utility installations. Whenever feasible and practical, utility line crossings of the roadway should intersect on a line perpendicular to the roadway alignment. Consideration should be given to encasing or installing utility line crossings in tunnels or conduits to allow servicing without disrupting the traffic flow.

When street grades, alignments, or widths are changed, utilities are usually required to relocate. Often, standard locations are inapplicable and unobtainable in street areas where existing utilities are seriously crowded and where it would not be feasible to expect significant reorientation. The location criteria must be practical and applicable in new developments, in urban relocation work, and in cases where overhead facilities are being converted into underground structures and plans. Utilities are not expected to change existing facilities as to location or depth simply for the purpose of creating uniformity. However, when new or relocation work is undertaken, uniformity should be sought wherever possible.

The horizontal and vertical location of utility lines within the roadway right-of-way limits should conform to the clear zone policies applicable for the system, type of roadway, and specific conditions for the particular section involved.

B. Definitions

Right-of-way: The land area of which the right to possession is secured or reserved by the Jurisdiction for the project, including permanent roadway easements.

Roadway: The portion of the right-of-way designated or ordinarily used for vehicular travel.

Sidewalk: That portion of the street primarily constructed for the use of pedestrians.

Street (Road): A general term denoting public way for vehicular travel, including the entire area within the right-of-way.

Utility: Includes all privately, publicly, municipally, or co-operatively owned structures and systems for supplying water, sewer, electric lights, street lights and traffic lights, gas, power, telegraph, telephone, communications, transit, pipelines, and the like.

C. Design

1. Limited right-of-way widths:

    a. Because of lack of space for utilities in most metropolitan areas, special consideration should be given in the initial roadway design to the potential for joint usage of the right-of-way that would be consistent with the primary function of the roadway.

    b. When the sanitary sewer is located outside of the paved surface, the gas, electric, telephone, and/or cable TV may need to be located in special utility easements in the front and/or rear yard.

    c. Existing development and limited right-of-way widths may preclude location of the sanitary sewer outside the paved surface of the roadway. Some cities may allow sanitary sewer within the roadway. Location under the paved surface requires special consideration and treatment. Accommodation of these facilities under the paved surface should be accomplished in a manner that will ensure a minimum adverse effect on traffic as a result of future utility service and maintenance activities.

2. Utility poles, vent standpipes, and other above-ground utility appurtenances that would constitute hazards to errant vehicles should not be allowed within the roadway clear zone. The only exceptions allowed would be where the appurtenance is breakaway or could be installed behind a traffic barrier erected to protect errant vehicles from some other hazard. The clear zone dimension to be maintained for a specific roadway use will be found in Chapter 5, Roadway Design.

3. Attachments of utility lines to bridge structures should be avoided where possible. Where there are no feasible alternate locations, such installations on bridge structures should be concealed from view. When attachments to bridges or structures are approved, the Engineer should refer to specific Jurisdiction standards for price of attachment, method of attachment, and other requirements.

4. On new installations or adjustment to existing utility lines, provision should be made for known or planned expansion of the utility facilities, particularly those located underground or attached to bridges. It is important that the placement of the utility considers the future widening of the roadway.

5. All utilities located within the public right-of-way for new roadway construction should comply with the drawing based on the width of right-of-way and pavement width.

6. The order of elevation priority for underground installation should be as follows:

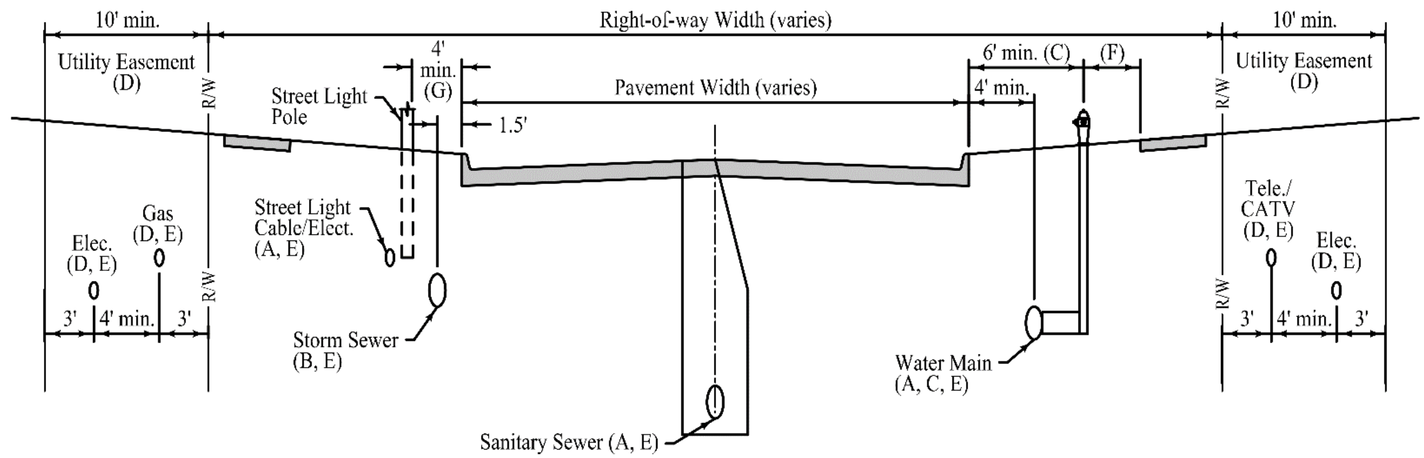
    a. Sanitary sewer

    b. Storm sewer

    c. Water main

    d. Other utilities

Figure 9A-1.01: Typical Urban Utility Locations



SANITARY SEWER UNDER THE STREET

GENERAL NOTES

A. When utilities are in the right-of-way, the following should be adhered to unless otherwise approved by the Jurisdiction:
   1. All utilities should be buried. When overhead utilities are allowed to cross the roadway, the minimum vertical clearances are 20 feet for the main cable and 18 feet for services.
   2. Telephone, Cable TV, and Water: Install on east and south side of road.
   3. Gas and Electric: Install on west and north side of road.
   4. Sanitary sewer: If under pavement, install in the center of the street. If outside the pavement, install on the west or north side of the street.

B. Storm Sewers: The normal location is 1.5 feet from back of curb. When combination manholes and intakes are used, the location increases to 5 feet.

C. Water Mains, Valves, and Hydrants:
   1. The normal location is 4 feet behind the back of curb. In areas of combination manholes and intakes, this distance is increased to a minimum of 6 feet.
   2. For local streets and minor collectors with limited right-of-way, use a 90° anchoring elbow between the hydrant tee and the valve. For maintenance purposes, the minimum distance between the centerline of the valve box and the face of the hydrant is 18 inches.

D. Utility Easements: Telephone, fiber optic, cable TV, electric, and gas lines should be located in front or rear yard easements. Upon approval of the Jurisdiction, these utilities may be placed in the right-of-way.

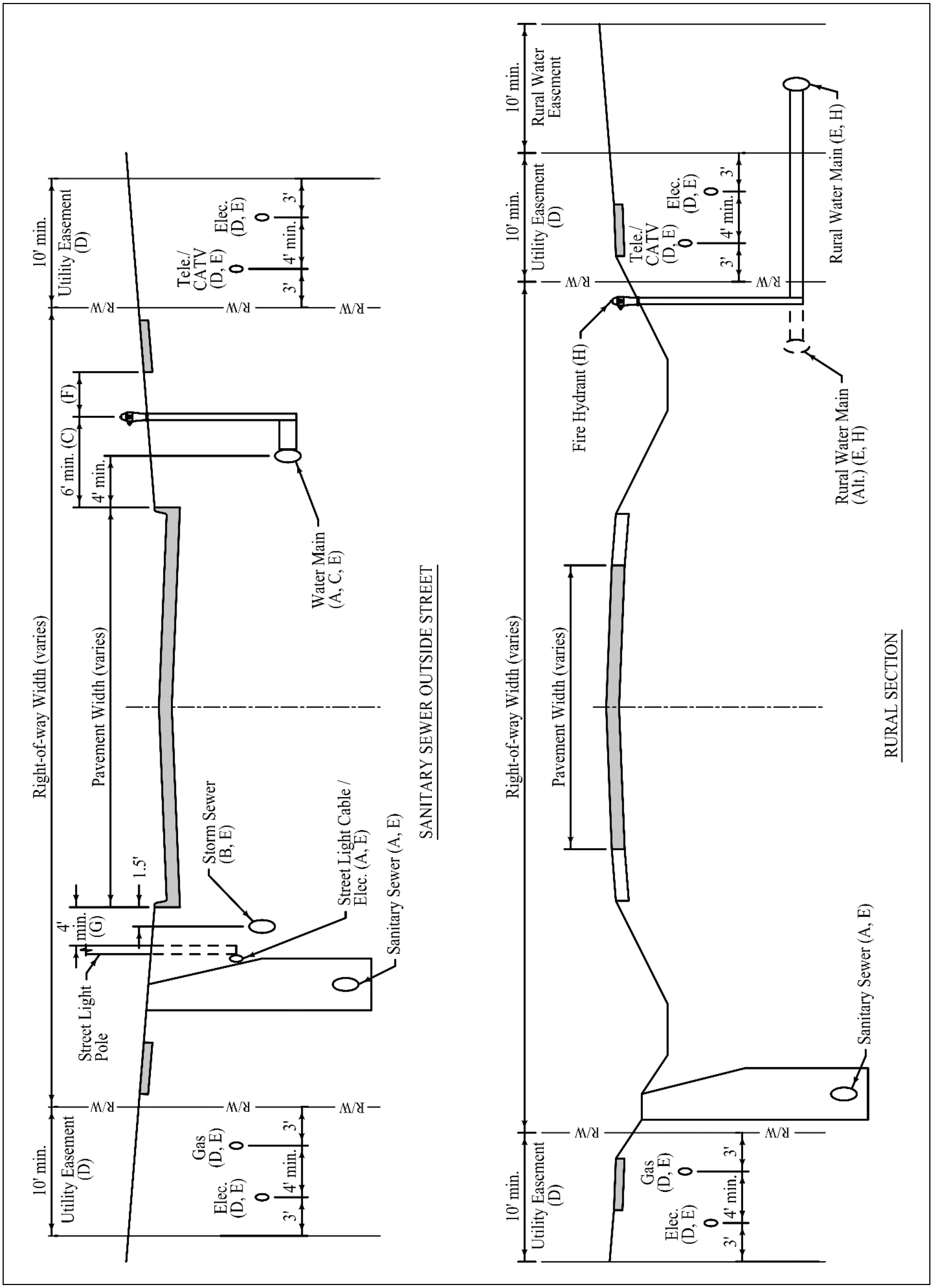
E. Depth of Bury:
   1. Public Utilities:
      Water: Varies from 4 feet to 5.5 feet. Refer to Chapter 4.
      Sanitary Sewer: Varies.
      Storm Sewer: Varies. Approximately 4 feet.
   2. Locate private utilities according to industry regulations. Typical depths are as follows:
      Cable TV: 3 feet minimum.
      Electric: 4 feet recommended. The minimum depth of bury for electric lines as per the National Electric Safety Code is:
         8kV to 50kV cable - 30 inches minimum.
         Up to 600-volt cable - 24 inches minimum.
         Street light cable - 18 inches minimum.
      Gas: 3 feet minimum.
      Telephone / Fiber Optic: 3 feet minimum.

F. Maintain a minimum separation of 2 feet from the edge of the sidewalk or shared use path to obstructions such as fire hydrants, utility poles, signs, etc.

G. Meet object setback, clear zone, and curb offset requirements of Section 5C-1.

H. Rural Water: Rural water mains are normally located in water main easements. If the size and pressure of the rural water main allows the placement of hydrants, the hydrant will be placed between the water main and the roadway at a location directed by the Jurisdiction. If the water main must be placed in the right-of-way, it should be placed at the toe of the back slope (or in the back slope if necessary to maintain a 3 foot minimum separation from other utilities).

Figure 9A-1.01 (Continued): Typical Urban Utility Locations



General Information for Trench Design

A. Trench Theory

When designing a pipe, the first step is to determine the flow capacity required, which will then determine the pipe type, diameter, and grade. The capacity of the pipe must be sufficient to carry external loads once it is buried.

A buried pipe must resist the dead load of the soil above it and any live loads applied at the surface, or the pipe will fail. Because buried pipes interact with the surrounding soil based, in part, on the stiffness of the pipe, different design methodologies have been developed for determining loads on rigid and flexible pipes.

A rigid pipe has significant strength but will crack if it is deformed. Because of this, a rigid pipe relies on its strength to carry external loads when buried. However, because of its stiffness, a rigid pipe must carry the entire load of the soil above it, and even some of the load from the soil adjacent to it. Rigid pipe design methodology is described in more detail in Section 9B-2 - Rigid Pipes.

A flexible pipe does not have the strength of a rigid pipe, but it will not crack when deformed. Because of this, a flexible pipe relies on its ability to deform to reduce the load on the pipe by transferring most of the load to the surrounding soil. Flexible pipe design methodology is described in more detail in Section 9B-3 - Flexible Plastic Pipes.

When analyzing a proposed pipe installation, both the magnitude of the load imposed on the pipe and the capacity of the pipe to carry the load must be determined. Both of these values are influenced by a number of factors, including the pipe's flexibility or rigidity, the pipe bedding, soil properties, and installation practices. Because these factors all interact with each other, it is important to understand how each one affects the ultimate performance of the pipe. These properties are described in more detail below and in the following sections.

B. Bedding and Foundation Materials

1. **Granular Bedding Material:** Bedding is the material installed in the bottom of the trench on which the pipe is laid. Proper pipe bedding is critical to the load carrying capacity of both rigid and flexible pipes. Clean, crushed Class I granular bedding material should be used for all gravity sewer installations. Granular bedding material is commonly used as backfill in the haunch zone (below the springline) as well as for primary and secondary backfill zones, depending on the pipe material.

   Although the interaction between the soil and pipe is different for rigid and flexible pipes, both types require proper placement and compaction of the bedding material under the pipe and in the haunch zone for proper support. By supporting rigid pipes along the bottom of the pipe, the pipe load is distributed over a larger area, thereby reducing the concentrated stresses at the invert of the pipe. Flexible pipes require proper bedding and haunch backfill to provide sidewall support for the pipe. As a flexible pipe deflects vertically, the sides of the pipe move outward. Without proper sidewall support and resistance to these lateral deflections, the vertical pipe deflections can exceed allowable levels.

Because bedding and haunch support is critical to the performance of both rigid and flexible pipes, proper installation of materials in these areas is critical. Most pipe installation guides recommend hand placement and slicing of granular bedding material with a shovel to ensure there are no voids in the haunch zone. Soil backfill should be compacted with hand compaction equipment in the haunch zone after placement of the pipe. Due to a variety of factors, including time constraints, lack of inspection, and concerns over trench safety, hand working and compacting backfill around a pipe are almost non-existent in today's construction industry.

In order to address these issues, the use of clean, self-compacting granular bedding and backfill material is recommended. Properly graded, clean, crushed stone that is dumped into a trench and shaped requires little or no additional compaction effort to provide a moderate degree of compaction. In addition, this material will bridge over some soft or yielding soils, reducing the need for over-excavation and foundation material. SUDAS Class I Material is a 1 inch, clean, crushed stone that should be used for most pipe bedding applications.

Where Class I or similar material is not available, gravel or crushed concrete may be used. However, these materials do not possess the self-compacting quality that Class I material has. Additional time and effort will be required to place and compact these materials. Increased construction observation may be required to ensure that the materials are properly placed and compacted.

2. **Stabilization (Foundation) Material:** The bottom of the trench should be firm, stable, and uniform to support the pipe and prevent movement during backfill and compaction. When Class I bedding material is inadequate for bridging trench bottoms with soft or yielding soils, overexcavation of the trench bottom and installation of stabilization material should be considered.

   Stabilization material consists of 2 1/2 inch clean crushed stone. It is installed in the bottom of the trench after overexcavation to remove any soft or yielding soils. The required depth of overexcavation and stabilization material varies as required to provide a firm base. Class I granular bedding and normal backfill are placed on top of the stabilization material as in a normal installation.

   Stabilization material can also be substituted for Class I bedding material when installing heavy pipe, such as concrete pipe 48 inches and greater. For heavy pipe, Class I bedding may be susceptible to movement under the weight of the pipe. Since stabilization material is significantly larger, it is better able to resist movement under heavy loads.

C. Backfill Materials

1. **Haunch Support:** The haunch support zone extends from the top of the bedding material to the springline, or mid-point, of the pipe. Like pipe bedding, this zone is critical to the support and performance of the pipe. For flexible pipes, this zone should be backfilled with Class I granular bedding material in order to provide adequate sidewall support to the pipe. Rigid pipes may be backfilled with Class I bedding material or suitable native soil, depending on the depth of the installation. Because a portion of the haunch zone is located underneath the pipe, this area is difficult to compact. If Class I granular bedding material is not used to backfill the haunch support zone, careful attention must be paid to ensure that proper compaction is achieved in this area and that the pipe is not damaged by compaction equipment.

2. **Primary and Secondary Backfill:** The primary backfill zone extends from the springline of the pipe to the top of the pipe. The secondary backfill zone extends from the top of the pipe to 1 foot over the top of the pipe. For most plastic pipes, the Class I granular bedding material should be extended from the haunch through the primary and secondary backfill zones. While these areas can be more easily compacted than the haunch area, plastic pipes are susceptible to damage by compaction equipment. Therefore, this zone is typically backfilled with Class I bedding material in order to protect the pipe from damage during compaction.

   Though care must still be taken during compaction, rigid and ductile iron pipes may be backfilled with suitable native materials.

3. **Final Backfill:** The final backfill zone extends from the top of the secondary backfill (1-foot above the top of the pipe) to the top of the trench. The materials placed in the final backfill zone have little impact on the load carrying capacity of the pipe. However, it is important that this area be backfilled with suitable soils and properly compacted to avoid future settlement. This is particularly important when the trench is located under future paving.

D. Dewatering

All pipes should be installed in a trench with a dry bottom. Ideally, the water table should be at least 2 feet below the bottom of the excavation. For installations below the water table in soils with a high coefficient of permeability, dewatering may be required. Due to the slow flow of water, installations in clay soils do not generally require dewatering, and water in the trench can be controlled with sump pumps. A number of methods are available when dewatering is required.

Well points (often called sand points) are one of the most common methods of dewatering. The well point system consists of a number of small diameter wells installed at regular intervals (typically 3 to 8 feet) adjacent to the proposed trench. Each well contains a pipe that extends down to the bottom of the well. At the surface, the end of each pipe is connected to a header pipe, which is connected to a vacuum pump. The use of well points results in a localized drawdown of the water table. Because groundwater is extracted by vacuum, the maximum depth of dewatering by well points is limited to 15 to 20 feet.

For installations where the trench bottom exceeds the limits of the well point system, or where the well points interfere with the construction, a deep well system may be required. A deep well system consists of a single well or a much smaller number of wells than a well point system. Wells may be spaced at 50 foot intervals or larger. In a deep well system, each well has a pump located at the bottom of the casing. This eliminates the depth limitations of the well point system. Deep wells can be installed to depths of up to 100 feet with a single-stage pump. Deep wells can draw down the water table over a significantly larger area than a well point. For this reason, they should be used with caution in areas where existing structures are present, as the reduction in water table level can cause settlement.

Rigid Pipes

A. Introduction

Rigid pipes are generally considered pipes that cannot deflect 2% of their diameter before failing. Common rigid pipes include concrete and clay as well as other specialized pipe materials. Because rigid pipes do not deflect significantly when loaded, the pipe itself must be capable of supporting the backfill placed over it and any additional loads that are applied to it.

A number of factors, including trench width, pipe strength, and bedding type, affect the magnitude of the load transmitted to the pipe and the ability of the pipe to carry the load. In an urban setting, most public utilities, including storm sewer, sanitary sewer, culverts, or water mains, are installed in an open trench; therefore, it is important to understand each of these characteristics and how they affect the structural capacity of a rigid pipe.

B. Trench Width

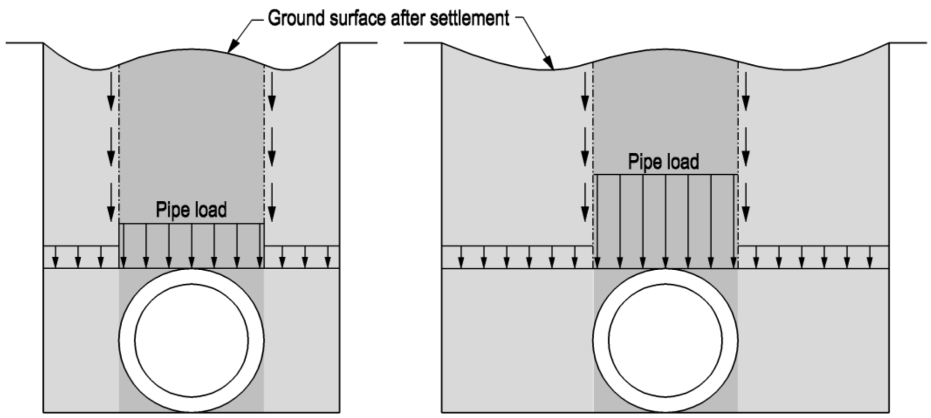
After trench excavation and pipe installation, bedding and backfill materials are placed in the trench. Even with proper compaction, the bedding and backfill materials in the trench will continue to undergo settlement. Because the height of the soil columns between the pipe and the trench walls are greater than the height of the soil column immediately above the pipe, and because a rigid pipe does not deflect, the areas adjacent to the pipe will undergo greater settlement than the soil column directly over the pipe.

As differential settlement occurs between the soil columns adjacent to the pipe and the columns directly above it, frictional forces are generated along the trench walls and along the central soil column over the pipe. As the adjacent soil columns settle, they drag the central soil column down, transferring a portion of their load to the pipe. Consequently, a rigid pipe tends to carry the entire load of the soil column directly above it and a portion of the load from the soil columns adjacent to it. The magnitude of the load from the adjacent soil columns that is transferred to the pipe varies, depending on trench width (see Figure 9B-2.01).

As the trench width increases, the load on the pipe will continue to increase until reaching a limiting value called the transition width. At this point, any additional increase in trench width does not affect the load on the pipe, and the pipe behaves as if it were buried in an embankment.

It should be noted that it is the width of the trench at the top of the pipe that affects the load on the pipe. The width or shape of the trench walls above this level can be sloped or stepped without increasing the load on the pipe.

Figure 9B-2.01: Effect of Trench Width on Rigid Pipe Load



Narrow Trench Wide Trench

In 1913, Anson Marston published a report on the interaction between the pipe and the surrounding soil after studying the problem at Iowa State College. The resulting "Marston Equation" has been used extensively to determine the earth load on pipes. For additional information on the Marston Equation, transition width, and the method of determining actual backfill loads, refer to the "Marston/Spangler Design Procedure" in the American Concrete Pipe Association's (ACPA) *Concrete Pipe Design Manual*.

C. Pipe Strength

A three-edged bearing test is used to determine the strength of a rigid pipe. The pipe is supported at two locations along the bottom, and a vertical load is applied at the top until the pipe fails. For concrete pipe, two failure methods are defined. The first, $D_{0.01}$, is the load at which a 0.01 inch crack develops in the pipe. The second, D_{ult}, is the ultimate load that the pipe can carry. D-load strengths are measured in pounds per liner foot per foot of pipe diameter. For design purposes, the $D_{0.01}$ value is used to provide a factor of safety. The following table correlates the pipe class with the two D-load values.

Table 9B-2.01: RCP Load Equivalents

Pipe Class[1] (ASTM C76)	**$D_{0.01}$ (lb/ft/ft diameter)**	**D_{ult}[2] (lb/ft/ft diameter)**
Class II	1,000	1,500
Class III	1,350	2,000
Class IV	2,000	3,000
Class V	3,000	3,750

[1] SUDAS specifies concrete pipe according to pipe class (minimum Class III)
[2] Iowa DOT specifies pipe by the ultimate load (e.g. Class III RCP is specified as 2000D)

Clay pipes are also tested, using a three-edged bearing test. The specified strength of clay pipe does not follow the D-load concept and varies depending on pipe diameter. Clay pipe strengths are the ultimate strength of the pipe and vary from 2,000 to 8,000 pounds per foot.

D. Bedding Factors

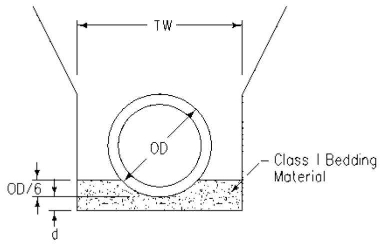
The forces imparted on a pipe in an installed condition are considerably different than the concentrated forces generated in the lab during a three-edged bearing test. In the installed condition, the pipe is supported and the load is distributed over the entire width of the pipe. Depending on the type of bedding provided, this can significantly increase the load carrying capacity of the pipe.

In order to relate the strength of the pipe from the three-edged bearing test to the actual capacity of the pipe in an installed condition, bedding factors have been developed to address a number of standard bedding types. The SUDAS Specifications closely follow the standard bedding types and the bedding factor values described in the ACPA *Concrete Pipe Design Manual*. For a given pipe strength, the $D_{0.01}$ strength is multiplied by the bedding factor to represent the strength of the pipe in the installed condition (see Figure 9B-2.02).

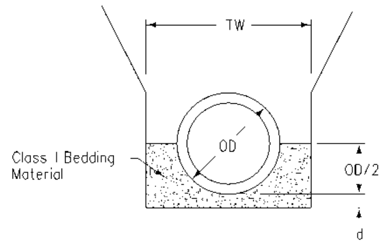
E. Live Load

In addition to the backfill load, a buried pipe must also support any live loads applied at the surface. For pipes installed under asphalt or concrete pavements, the live load from vehicular traffic is distributed sufficiently by the pavement that the live load transmitted to the pipe is negligible. Likewise, for pipes that are buried deeper than 6 feet, live load can usually be neglected. However, for shallow pipes in unpaved areas, or for pipes that cross under railroads, live loads on the pipe should be considered.

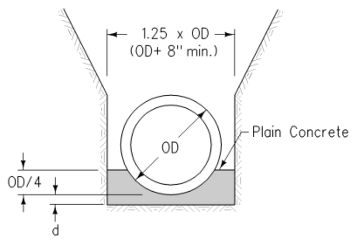
Figure 9B-2.02: Rigid Pipe Bedding Types and Bedding Factors



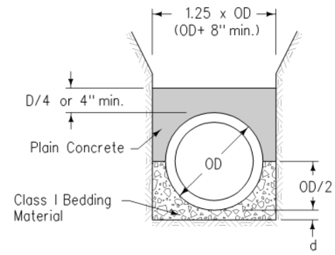
Class R-1
Bedding Factor = 1.5



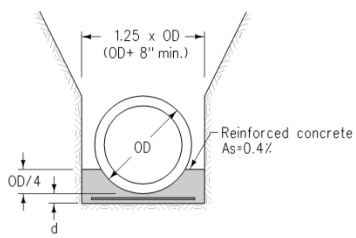
Class R-2
Bedding Factor = 1.9



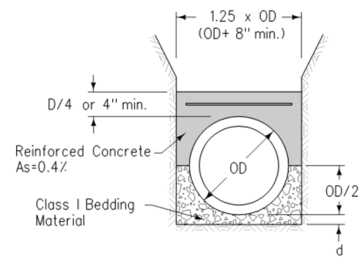
Class R-3 (Unreinforced)
Bedding Factor = 2.8



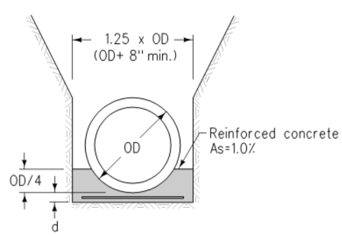
Class R-4 (Unreinforced)
Bedding Factor = 2.8



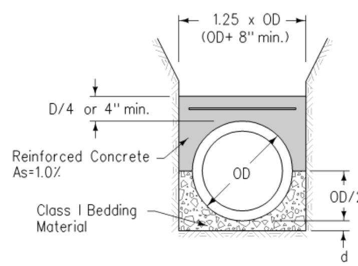
Class R-3 (0.4% Reinforcing)
Bedding Factor = 3.4



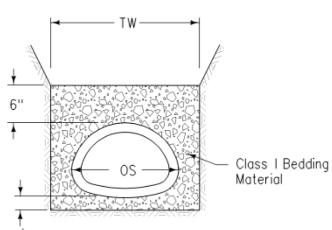
Class R-4 (0.4% Reinforcing)
Bedding Factor = 3.4



Class R-3 (1.0% Reinforcing)
Bedding Factor = 4.5



Class R-4 (1.0% Reinforcing)
Bedding Factor = 4.5



SUDAS Class R-5
Bedding Factor = 1.9

<u>Figure Legend</u>
d = Bedding depth below pipe; OD/8 or 4" min.
D = Pipe diameter
OD = Outside pipe diameter
TW = Trench width

Flexible Pipes

A. Introduction

Flexible pipes are generally considered pipes that will deflect at least 2% of their diameter without any damage. However, most flexible pipes used for utility applications are required to undergo deflections of 20 to 30% during testing and certification without failing. The key to the performance of a flexible pipe is its ability to deflect without buckling or cracking.

The most common flexible pipes currently in use are polyvinyl chloride (PVC), high density polyethylene (HDPE), and polypropylene. Numerous varieties of each are produced. PVC pipe is used extensively for sanitary sewers and water mains. HDPE pipe is commonly used for subdrains and is also available for storm sewer applications. Polypropylene pipe is used for both sanitary and storm sewer applications.

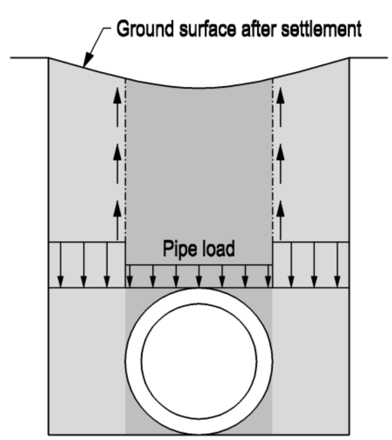
In order to take advantage of the benefits and avoid the limitations of flexible pipes, it is necessary to understand how they perform and how their properties are defined.

B. Soil-pipe Interaction

A flexible pipe obtains its load-carrying ability from its flexibility. Just like a rigid pipe, after the installation of a flexible pipe, the trench bedding and backfill materials will settle. However, because the flexible pipe deflects when loaded, the central soil column, directly over the pipe, will settle more than the adjacent soil columns.

As differential settlement occurs between the soil column over the pipe and the soil columns adjacent to the pipe, frictional forces between the soil columns transfer some of the load from the central soil column to the adjacent soil columns (see Figure 9B-3.01). This reduces the load on the flexible pipe. As the pipe is loaded, the pipe deflects vertically, pushing the sides of the pipe outward toward the sides of the trench. This results in the development of sidewall support from the pipe bedding.

Figure 9B-3.01: Soil-pipe Interaction for Flexible Pipes



Flexible pipe performance is highly dependent on proper bedding to provide the required sidewall support. A pipe with lateral sidewall support is capable of carrying a significantly larger load than an unsupported pipe. Without sidewall support, some flexible pipes would be crushed by the weight of the backfill above. For this reason, it is imperative that flexible pipes be properly backfilled with high-quality materials.

Given the significance of sidewall support, consideration must be given to the locations where flexible pipes are installed. Flexible pipes should not be used in areas where future adjacent excavations are likely. These excavations could expose or weaken the bedding envelope supporting the pipe.

For additional information on the soil-pipe interaction of flexible pipes, and the method to determine pipe load and predicted deflection, refer to the Uni-Bell PVC Pipe Association's: *Handbook of PVC Pipe: Design and Construction* or the Plastic Pipe Institute's publication: *The Complete Corrugated Polyethylene Pipe Design Manual and Installation Guide*.

C. Pipe Design

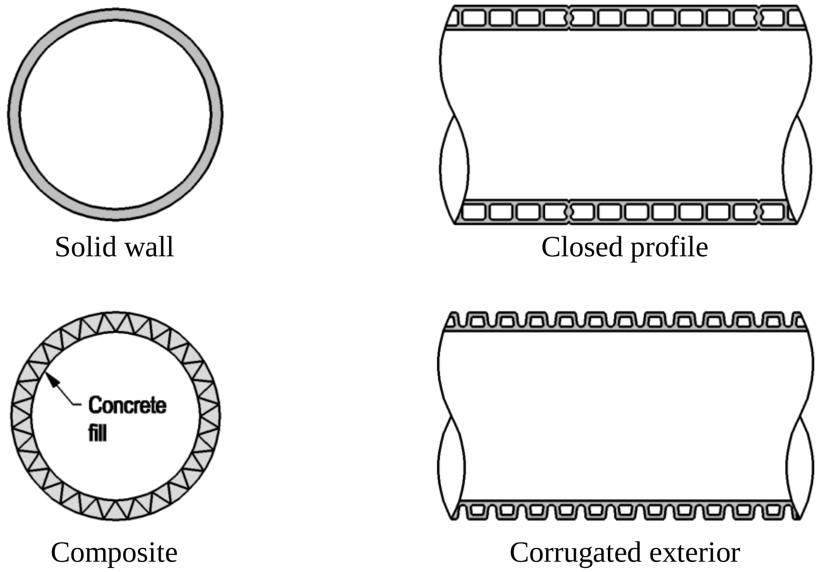
The design of most flexible pipes is based upon pipe stiffness. Pipe stiffness is a term used to describe the resistance of a flexible pipe to deflection when subjected to a load. Pipe stiffness is measured by placing a section of flexible pipe between two flat plates. A load is applied until the pipe is deflected 5% of its diameter. The load at which this occurs is the pipe stiffness. Pipe stiffness is specified in lb/in^2.

In general, pipe stiffness is related to the material properties and wall thickness of the pipe (see Figure 9B-3.02). For solid walled pipe with a given modulus of elasticity, the ratio of the pipe diameter to its wall thickness (diameter ratio or DR) determines the stiffness of the pipe. Solid walled pipes of different diameters, but with the same DR, all have the same pipe stiffness. To provide a stiffer pipe, the wall thickness is increased (i.e., DR is reduced).

Another way to increase pipe stiffness is to change the shape of the pipe wall. Pipe manufacturers have developed a number of different wall cross-sections that increase or maintain pipe stiffness while using less material per foot of pipe. Closed profile pipe uses an "I"-beam-type cross-section. Composite pipe is a dual-walled pipe with a truss-type structure in the middle and the area filled with lightweight concrete. Flexible pipe is also commonly produced with a corrugated exterior and smooth interior.

Regardless of the wall shape, the generally accepted standard for minimum pipe stiffness is 46 lb/in^2. This corresponds to a PVC pipe with a DR of 35. Pipes with a lower stiffness are also available; however, they should be used with caution in the right-of-way or other areas subject to disturbance for the reasons described in the Section 9B-2 - Rigid Pipes. In addition, installing pipes with stiffness lower than 46 lb/in^2 should be done under careful supervision to ensure that the pipe has proper bedding. After installation, these pipes should be tested with a mandrel to ensure deflections do not exceed 5%.

Figure 9B-3.02: Types of Commonly Used Flexible Pipe



1. **PVC:** PVC pipe is used primarily for sanitary sewer mains and service lines. Sanitary sewer installations are generally deep enough that it is unlikely an adjacent excavation will encroach on the pipe bedding envelope. Therefore, the use of PVC for sanitary sewers in the right-of-way is acceptable. However, for shallow installations, consideration should be given to the possibility of adjacent excavations causing damage. If this is likely, an alternate pipe material or a PVC pipe with higher pipe stiffness should be considered. Use of PVC pipe for storm sewer applications requires specific approval by the Engineer due to the increased potential for impact to the pipe envelope and subsequent damage to the pipe from adjacent or crossing excavation activity.

   For a given class of PVC pipe, pipe stiffness is generally consistent regardless of the diameter. The minimum PVC pipe stiffness allowed in the SUDAS Specifications is 46 lb/in^2.

2. **HDPE:** HDPE pipe has been used extensively as subdrain and as agricultural drain tile. HDPE storm sewer pipe is also available. Unlike PVC pipe, the pipe stiffness for HDPE pipe varies depending on diameter. Use of HDPE pipe for storm sewer applications requires specific approval by the Engineer due to the increased potential for impact to the pipe envelope and subsequent damage to the pipe from adjacent or crossing excavation activity.

   In addition to pipe deflection, HDPE pipe must also be analyzed for several additional failure modes. These include wall thrust, buckling pressure, bending stress, and bending strain. In general, the limits on the depth of bury for HDPE pipe is not due to deflection, but wall thrust.

   One of HDPE pipe’s material properties is its tendency to creep, or permanently deform when stressed beyond a certain level for an extended time. If the wall thrust stresses at the springline of the pipe are high enough, the sidewall of the pipe can undergo permanent deformation. Wall thrust failures occur as rippling, buckling, or cracking at the springline of the pipe.

3. **Polypropylene:** Polypropylene pipe can be used for both sanitary sewer and storm sewer applications. It is made from polypropylene resin to form a pipe that has improved impact resistance and less susceptibility to brittleness than HDPE pipe.

   The minimum pipe stiffness allowed in the SUDAS Specifications is 46 psi. The smaller pipe diameters (12 inch to 30 inch) are double walled with a smooth interior and corrugated exterior.

In order to maintain the minimum pipe stiffness in the larger diameters (30 inch to 60 inch), the pipe is triple walled. Both the double wall and triple wall pipe have an integral bell and spigot joint.

Use of polypropylene pipe for storm sewer applications requires specific approval by the Engineer due to the increased potential for impact to the pipe envelope and subsequent damage to the pipe from adjacent or crossing excavation activity.

D. Flexible Pipe Bedding

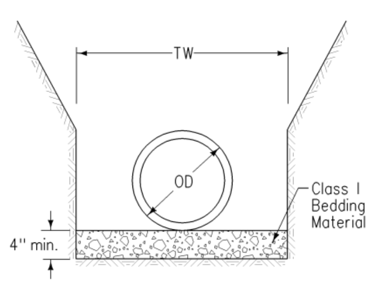
For most gravity installations with flexible pipe, granular bedding material is recommended. Suitable granular bedding material is self-compacting when placed in the trench. This ensures proper pipe support is provided in the area below the springline, where it is difficult to provide mechanical compaction. The bedding below the springline is critical for providing proper sidewall support for flexible pipes.

Granular bedding should be extended to the top of the pipe for storm sewers (Bedding Class F-2), and to 1 foot over the top of the pipe for sanitary sewer installations (Bedding Class F-3). The additional granular bedding material in this area protects the pipe from impact and movement during final trench backfill. In excavations where trench boxes are used, care must be taken to prevent disturbance of the pipe and bedding material when moving the trench box.

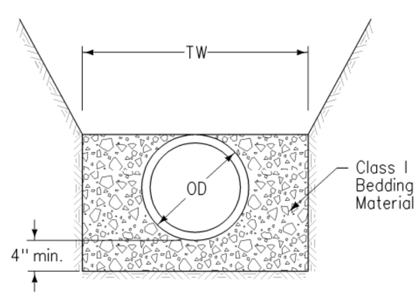
For gravity pipe installations using ductile iron pipe or other flexible water main materials, granular bedding material may not be required along the sides of the pipe, due to the additional pipe strength provided by these products. For these applications, granular bedding is only required under the pipe (Bedding Class F-1) to assist in achieving the proper grade and alignment.

Figure 3 illustrates the standard bedding classes for flexible pipe installations. Refer to Section 9B-4 - Ductile Iron Pipe, for bedding types for flexible pressure pipe (AWWA C900 / C905).

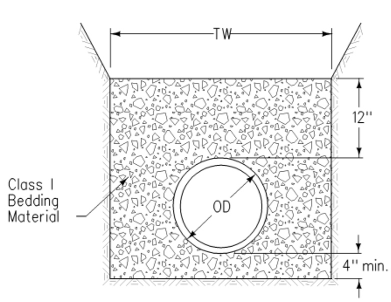
Figure 9B-3.03: Flexible Pipe Bedding Types



Class F-1



Class F-2



Class F-3

E. Trench Width

1. **Minimum:** Unlike rigid pipe, the load on a flexible pipe does not increase as trench width increases. While trench width does not affect the pipe load, it must be wide enough to properly place and compact the bedding material in the haunch and primary backfill areas of the pipe. Generally, this is considered to be 1.25 times the outside diameter of the pipe plus 12 inches, or the outside diameter of the pipe plus 18 inches, whichever is greater.

2. **Poor Soils:** As mentioned earlier in this section, a critical requirement for flexible pipe performance is sidewall support. In a typical installation, the thrust forces from the sidewall of the deflecting pipe are transferred through the granular bedding material to the trench walls. As these forces pass through the rock envelope, they are distributed over a larger area, reducing the pressure against the trench walls. The crushed stone bedding has a higher bearing capacity, or modulus of soil reaction, than the adjacent soil, allowing it to carry greater loads than the surrounding soil without deformation.

   In a typical installation, the granular bedding material reduces the pressure against the trench walls to an acceptable level. However, for installations with poor soil conditions, the in-situ soils may not provide adequate lateral support with a standard trench and pipe bedding. Examples of poor soil conditions include poorly compacted fill with a SPT blow count of five or less, peat, muck, or highly expansive soils. In these situations, additional trench width may be required. A wider trench, and thus a wider rock envelope, allows the thrust forces from the pipe sidewall to be distributed over an even larger area on the trench wall. By increasing the bearing area, the pressure on the trench wall can be reduced to a level that the in-situ soil can support. For conditions with poor soils, increasing the minimum trench width to two times the outside diameter of the pipe is recommended.

F. Pressure Pipe

Using flexible pipes for pressure applications such as water main or sanitary sewer force mains is also common. Unlike flexible pipes for gravity flow applications, pressure pipes are classified based upon the pressure rating of the pipe, rather than the pipe stiffness.

Flexible pressure pipes typically have a significantly thicker wall than gravity flow pipes. As such, the inherent stiffness of the pipe is also significantly greater. For example, C900, DR 18 pipe has an equivalent pipe stiffness of 360 lb/in^2.

Because of the increased pipe stiffness and relatively shallow depth of bury, bedding requirements for PVC water mains and force mains are less critical than flexible gravity pipe. Native soil can be used for bedding many PVC water main or force main installations. Likewise, the concern of adjacent excavations disturbing the pipes sidewall support is not an issue with PVC pressure pipes. Refer to Section 9B-4 - Ductile Iron Pipe, for typical pressure pipe installations.

Ductile Iron Pipe

A. Introduction

Ductile iron pipe is used primarily as water main, but is also used as force main and for some specialized gravity flow situations. Since the pipe is constructed of ductile iron, it can deflect without failing and behaves similar to a flexible plastic pipe. However, ductile iron pipe has some additional properties that warrant a slightly different design methodology than previously described for flexible plastic pipes.

B. Pipe Design

The first step in analyzing a ductile iron pipe for structural capacity is to determine what pipe thickness to use. Currently, there are two different pipe classifications for ductile iron pipe: Pressure Class and Thickness Class. SUDAS requires Thickness Class 52 pipe for all water main 24 inches or smaller. Unlike PVC pipe, ductile iron pipe does not follow a standardized diameter ratio (DR), so there is no easy method of determining the wall thickness based upon diameter. Pipe standard AWWA C151 indicates the nominal wall thicknesses for Class 52 pipe. However, the values listed in the AWWA standard are not the values used for design purposes. The AWWA values include a casting allowance to ensure that negative thickness deviations do not occur during the casting process. The casting allowance varies from 0.05 to 0.09 inches, depending on diameter. A service allowance of 0.08 inches is also included in the wall thickness to account for material loss from the pipe over its service life. These values are subtracted from the stated wall thickness to determine the design thickness.

Once the design thickness of the pipe is known, the pipe can be analyzed for deflection. Just like flexible pipes, ductile iron can undergo significant deflections without damage. However, the allowable deflection for ductile iron pipe is normally limited to 3%. This limitation is imposed to protect the cement-mortar lining on the inside of ductile iron water pipe.

In addition to deflection limitations, the ring bending stress in the pipe must also be determined. Maximum ring bending stress occurs at the invert of the pipe. If the stress exceeds the yield stress of the ductile iron material, the pipe will undergo permanent deformation.

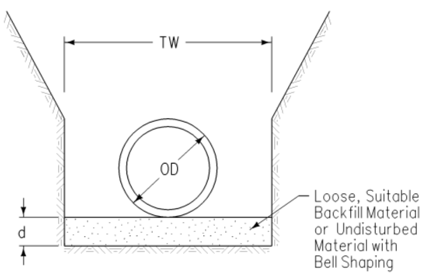
The equations and procedures for determining pipe deflection and ring bending stress are provided in the Ductile Iron Pipe Research Association's (DIPRA) publication "Design of Ductile Iron Pipe."

C. Bedding

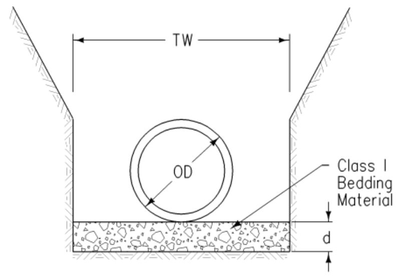
For shallow installations, ductile iron pipe can be installed without granular bedding material. However, the sidewall support from granular bedding material allows ductile iron pipe to carry greater loads then the pipe could by itself. This is an important consideration for deep installations. For example, the maximum depth of bury for a 24 inch Thickness Class 52 ductile iron pipe in a Class P-1 bedding (native soil) is 16 feet. The allowable depth of bury for the same pipe in a Class P-3 bedding (crushed stone encasement) is 38 feet.

Figure 9B-4.01 shows the standard bedding classes for pressure pipe installations. Refer to Section 9B-3 - Flexible Plastic Pipes, regarding bedding requirements for ductile iron pipe when used in a gravity flow installation.

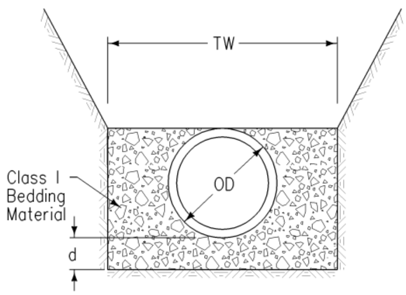
Figure 9B-4.01: Pressure Pipe Bedding Types



Class P-1



Class P-2



Class P-3

Depth of Bury Tables

A. General

The depth of bury tables on the following pages are based upon the design methodology from the various pipe material associations' design manuals. In order to develop the allowable depth of bury tables, numerous assumptions, including site characteristics and construction methods, were required for each pipe material. In general, when assumptions were required, the values that would provide more conservative results were selected.

For site conditions that differ from the assumptions used for the following tables, a separate calculation should be made with values appropriate to the specific conditions at the proposed project. In order to assist with the lengthy equations involved in some of the pipe design methods, SUDAS has developed a number of spreadsheets that will calculate the allowable depth of bury based upon site conditions entered by the user. These spreadsheets are available on the SUDAS website at www.iowasudas.org. The spreadsheets follow the design methodology of each pipe material association's design manual. Before using the spreadsheets, the designer should have a thorough understanding of the limitations of each design method.

B. Rigid Pipe Assumptions

The depth of bury calculations for clay pipe were done in accordance with the National Clay Pipe Institute's *Clay Pipe Engineering Manual*. The depth of bury calculations for concrete pipe were done in accordance with the ACPA *Concrete Pipe Design Manual*, utilizing the Marston-Spangler design method. The results of the depth of bury calculations for concrete and clay pipe indicated in Tables 9B-5.01 through 9B-5.05 were developed with the following assumptions:

- Saturated clay backfill – $k\mu'$=0.110
- Unit weight of backfill = 120 lb/ft^3
- $D_{0.01}$ pipe strength with a factor of safety of 1.0 (concrete pipe only)
- A minimum trench width of 54 inches (this is the smallest excavator bucket commonly used by some contractors).
- An HS-20 live load applied in an unpaved condition. If the pipe will not be subjected to live load, the minimum depth of bury does not apply.
- Maximum allowable depth of bury was cut off at 40 feet. Calculated values may exceed this depth, but were not shown. For depths greater than 40 feet, an independent analysis should be done using values for actual site conditions.

Table 9B-5.01: Allowable Depth of Bury for Class III (2000D) RCP

Pipe Diameter (inches)	Bedding Class (feet)				
	R-1	***R-2***	***R-3 and R-4***		
			A_s=0.0%	A_s=0.4%	A_s=1.0%
12	2 to 7	1 to 10	1 to 15	1 to 19	1 to 27
15	2 to 8	1 to 10	1 to 16	1 to 19	1 to 27
18	1 to 8	1 to 11	1 to 16	1 to 20	1 to 40
21	1 to 8	1 to 11	1 to 18	1 to 26	1 to 40
24	1 to 8	1 to 12	1 to 23	1 to 36	1 to 40
27	1 to 10	1 to 15	1 to 30	1 to 40	1 to 40
30	1 to 11	1 to 15	1 to 29	1 to 40	1 to 40
33	1 to 11	1 to 15	1 to 28	1 to 40	1 to 40
36	1 to 11	1 to 15	1 to 27	1 to 40	1 to 40
42	1 to 11	1 to 15	1 to 26	1 to 38	1 to 40
48	1 to 11	1 to 15	1 to 26	1 to 36	1 to 40
54	1 to 11	1 to 15	1 to 25	1 to 34	1 to 40
60	1 to 11	1 to 15	1 to 25	1 to 33	1 to 40
66	1 to 11	1 to 15	1 to 24	1 to 32	1 to 40
72	1 to 11	1 to 15	1 to 24	1 to 32	1 to 40

Table 9B-5.02: Allowable Depth of Bury for Class IV (3000D) RCP

Pipe Diameter (inches)	Bedding Class (feet)				
	R-1	***R-2***	***R-3 and R-4***		
			A_s=0.0%	A_s=0.4%	A_s=1.0%
12	1 to 12	1 to 15	1 to 23	1 to 28	1 to 40
15	1 to 12	1 to 16	1 to 23	1 to 30	1 to 40
18	1 to 13	1 to 16	1 to 29	1 to 40	1 to 40
21	1 to 13	1 to 18	1 to 40	1 to 40	1 to 40
24	1 to 16	1 to 23	1 to 40	1 to 40	1 to 40
27	1 to 19	1 to 30	1 to 40	1 to 40	1 to 40
30	1 to 19	1 to 29	1 to 40	1 to 40	1 to 40
33	1 to 19	1 to 28	1 to 40	1 to 40	1 to 40
36	1 to 19	1 to 28	1 to 40	1 to 40	1 to 40
42	1 to 18	1 to 27	1 to 40	1 to 40	1 to 40
48	1 to 18	1 to 26	1 to 40	1 to 40	1 to 40
54	1 to 18	1 to 25	1 to 40	1 to 40	1 to 40
60	1 to 18	1 to 25	1 to 40	1 to 40	1 to 40
66	1 to 18	1 to 25	1 to 40	1 to 40	1 to 40
72	1 to 18	1 to 24	1 to 40	1 to 40	1 to 40

Table 9B-5.03: Allowable Depth of Bury for Class V (3750D) RCP

Pipe Diameter (inches)	Bedding Class (feet)				
	R-1	***R-2***	***R-3 & R-4***		
			A_s=0.0%	***A_s=0.4%***	***A_s=1.0%***
12	1 to 18	1 to 23	1 to 35	1 to 40	1 to 40
15	1 to 19	1 to 24	1 to 40	1 to 40	1 to 40
18	1 to 19	1 to 30	1 to 40	1 to 40	1 to 40
21	1 to 25	1 to 40	1 to 40	1 to 40	1 to 40
24	1 to 34	1 to 40	1 to 40	1 to 40	1 to 40
27	1 to 40	1 to 40	1 to 40	1 to 40	1 to 40
30	1 to 40	1 to 40	1 to 40	1 to 40	1 to 40
33	1 to 40	1 to 40	1 to 40	1 to 40	1 to 40
36	1 to 40	1 to 40	1 to 40	1 to 40	1 to 40
42	1 to 37	1 to 40	1 to 40	1 to 40	1 to 40
48	1 to 35	1 to 40	1 to 40	1 to 40	1 to 40
54	1 to 33	1 to 40	1 to 40	1 to 40	1 to 40
60	1 to 32	1 to 40	1 to 40	1 to 40	1 to 40
66	1 to 31	1 to 40	1 to 40	1 to 40	1 to 40
72	1 to 31	1 to 40	1 to 40	1 to 40	1 to 40

Table 9B-5.04: Allowable Depth of Bury for Reinforced Concrete Arch Pipe

Pipe Size (inches)	**Equivalent Diameter (inches)**	**Pipe Class A-III (feet)**	**Pipe Class A-IV (feet)**
18 by 11	15	2 to 11	2 to 16
22 by 13	18	2 to 11	1 to 20
26 by 15	21	2 to 14	1 to 27
29 by 18	24	2 to 15	1 to 31
36 by 22	30	1 to 15	1 to 29
44 by 27	36	1 to 15	1 to 28
51 by 31	42	1 to 15	1 to 27
58 by 36	48	1 to 15	1 to 26
65 by 40	54	1 to 15	1 to 26
73 by 45	60	1 to 15	1 to 25
88 by 54	72	1 to 15	1 to 25

Table 9B-5.05: Allowable Depth of Bury for Extra Strength VCP

Pipe Diameter (inches)	**Bedding Class (feet)**				
	R-1	***R-2***	***R-3 & R-4***		
			A_s=0.0%	***A_s=0.4%***	***A_s=1.0%***
6	1 to 25	1 to 30	1 to 30	1to 30	1 to 30
8	1 to 20	1 to 26	1 to 30	1 to 30	1 to 30
10	1 to 18	1 to 23	1 to 30	1 to 30	1 to 30
12	1 to 16	1 to 20	1 to 30	1 to 30	1 to 30
15	1 to 15	1 to 19	1 to 28	1 to 30	1 to 30
18	1 to 14	1 to 18	1 to 30	1 to 30	1 to 30
21	1 to 15	1 to 22	1 to 30	1 to 30	1 to 30
24	1 to 18	1 to 28	1 to 30	1 to 30	1 to 30
27	1 to 20	1 to 30	1 to 30	1 to 30	1 to 30
30	1 to 19	1 to 29	1 to 30	1 to 30	1 to 30
33	1 to 20	1 to 30	1 to 30	1 to 30	1 to 30
36	1 to 20	1 to 30	1 to 30	1 to 30	1 to 30
39	1 to 19	1 to 29	1 to 30	1 to 30	1 to 30
42	1 to 18	1 to 26	1 to 30	1 to 30	1 to 30

C. Flexible Pipe Assumptions

The depth of bury calculations for PVC pipe were done in accordance with the Uni-Bell PVC Pipe Association’s *Handbook of PVC Pipe: Design and Construction*. The depth of bury calculations for HDPE pipe were done according to the Plastic Pipe Institute’s: *The Complete Corrugated Polyethylene Pipe Design Manual and Installation Guide*. The AASHTO design method was used for the determination of live load for both materials. The results of the depth of bury calculations for PVC and HDPE pipe indicated in Tables 9B-5.06, 9B-5.07, and 9B-5.08 were developed with the following assumptions:

PVC assumptions:

- Unit weight of backfill is 120 lb/ft^3
- Prism load for backfill
- Deflection lag factor (D_L) of 0.1
- Modulus of soil reaction (E’) of 0 psi, 1000, lb/in^2, and 1000 lb/in^2 for pipe classes F-1, F-2, and F-3 respectively.
- An HS-20 live load applied in an unpaved condition. If the pipe will not be subjected to live load, the minimum depth of bury does not apply.
- Maximum allowable pipe deflection of 5%. A value of 3% is used for design based upon the published deflection accuracy of $\pm$ 2% for dumped crushed rock bedding.
- Maximum allowable depth of bury was cut off at 40 feet. Calculated values may exceed this depth, but were not shown. For depths greater than 40 feet, an independent analysis should be done using values for actual site conditions.

HDPE assumptions:

- Unit weight of backfill is 120 lb/ft^3
- Prism load for backfill
- Water table 2 feet below ground surface
- Deflection lag factor (D_L) of 0.1
- Crushed rock bedding with a 1,000 lb/in^2 modulus of soil reaction (E’)

- An HS-20 live load applied in an unpaved condition. If the pipe will not be subjected to live load, the minimum depth of bury does not apply; however sufficient cover should be provided to protect the pipe from damage by ultraviolet radiation or maintenance equipment.
- Maximum allowable pipe deflection of 5%
- Pipe also checked for wall thrust, critical buckling pressure, bending stress, and bending strain.

Table 9B-5.06: Allowable Depth of Bury for Gravity Flow PVC Pipe - Bedding Class F-2 or F-3

Pipe Diameter (inches)	**ASTM (feet)**					
	D 3034 Solid Wall		***F 679 Solid Wall***	***F 949 Corrugated Exterior***	***F 1803 Closed Profile***	***D 2680 Composite***
	SDR 26	***SDR 35***	***SDR 35***			
8	2 to 28	2 to 24	---	2 to 24	---	2 to 32
10	2 to 28	2 to 24	---	2 to 24	---	2 to 32
12	2 to 28	2 to 24	---	2 to 24	---	2 to 32
15	2 to 28	2 to 24	---	2 to 24	---	2 to 32
18	---	---	2 to 24	2 to 24	---	---
21	---	---	2 to 24	2 to 24	2 to 24	---
24	---	---	2 to 24	2 to 24	2 to 24	---
27	---	---	2 to 24	---	2 to 24	---
30	---	---	2 to 24	2 to 24	2 to 24	---
33	---	---	2 to 24	---	---	---
36	---	---	2 to 24	2 to 24	2 to 24	---
42	---	---	2 to 24	---	2 to 24	---
48	---	---	2 to 24	---	2 to 24	---
54	---	---	---	---	2 to 24	---
60	---	---	---	---	2 to 24	---

Table 9B-5.07: Allowable Depth of Bury for AWWA C900/C905 PVC Pressure Pipe

Pipe Diameter (inches)	**Bedding Class (feet)**		
	P-1	***P-2***	***P-3***
4	2 to 19	2 to 40	2 to 40
6	2 to 19	2 to 40	2 to 40
8	2 to 19	2 to 40	2 to 40
10	2 to 19	2 to 40	2 to 40
12	2 to 19	2 to 40	2 to 40
14	2 to 19	2 to 40	2 to 40
16	2 to 19	2 to 40	2 to 40
18	2 to 19	2 to 40	2 to 40
20	2 to 19	2 to 40	2 to 40
24	2 to 19	2 to 40	2 to 40

Table 9B-5.08: Allowable Depth of Bury for HDPE Pipe - Bedding Class F-2 or F-3

Pipe Diameter (inches)	AASHTO M 294 (feet)
6	2 to 8
8	2 to 8
10	1 to 9
12	2 to 8
15	1 to 9
18	1 to 9
24	1 to 9
30	1 to 9
36	1 to 9
42	1 to 8
48	1 to 8
54	1 to 8
60	1 to 8

D. Ductile Iron Pipe Assumptions

The depth of bury calculations for ductile iron were done according to the DIPRA publication "Design of Ductile Iron Pipe." The results of the depth of bury calculations for ductile iron pipe indicated in Table 9B-5.09 were developed with the following assumptions:

- Unit weight of backfill is 120 lb/ft^3
- Prism load for backfill
- An HS-20 live load applied for all conditions
- Live load impact factor of 1.5
- Bedding classes P-1, P-2, and P-3 follow DIPRA laying conditions Type 2, Type 4, and Type 5, respectively.
- Maximum allowable pipe deflection of 3%
- 48,000 psi ring bending stress limit.
- Maximum allowable depth of bury was cut off at 40 feet. Calculated values may exceed this depth, but were not shown. For depths greater than 40 feet, an independent analysis should be done using values for actual site conditions.

Table 9B-5.09: Allowable Depth of Bury for Ductile Iron Pipe (Thickness Class 52)

Pipe Diameter (inches)	**Bedding Class (feet)**		
	P-1	***P-2***	***P-3***
4	2.5 to 40	2.5 to 40	2.5 to 40
6	2.5 to 40	2.5 to 40	2.5 to 40
8	2.5 to 40	2.5 to 40	2.5 to 40
10	2.5 to 36	2.5 to 40	2.5 to 40
12	2.5 to 31	2.5 to 40	2.5 to 40
14	2.5 to 26	2.5 to 40	2.5 to 40
16	2.5 to 23	2.5 to 37	2.5 to 40
18	2.5 to 20	2.5 to 34	2.5 to 40
20	2.5 to 18	2.5 to 32	2.5 to 40
24	2.5 to 16	2.5 to 29	2.5 to 38
30	2.5 to 13	2.5 to 23	2.5 to 31
36	2.5 to 13	2.5 to 22	2.5 to 30
42	2.5 to 13	2.5 to 21	2.5 to 29
48	2.5 to 13	2.5 to 19	2.5 to 27
54	2.5 to 13	2.5 to 19	2.5 to 27

Casing Pipe

A. General

Utilities must often be encased in a steel pipe when crossing under roadways or railroads. Steel casing pipe complying with the requirements of ASTM A252 (Standard Specification for Welded and Seamless Steel Pipe Piles) is generally used.

Depending on the timing of the installation, the casing pipe can be either installed in an open cut trench or by one of the trenchless techniques described in Chapter 14.

Regardless of the installation method, the casing pipe thickness and casing pipe diameter should be specified on the plans.

B. Casing Thickness

The casing pipe must have sufficient thickness to withstand both earth loads and any live loads imposed from traffic above. Table 9B-5.01 provides minimum recommended casing pipe thicknesses for both roadway and railroad installations. The roadway values are based upon common industry standards. The railroad values are based upon American Railway Engineering and Maintenance-of-Way Association (AREMA) design standards. Individual railroad standards may vary.

Table 9B-5.01: Minimum Casing Pipe Thickness

Nominal Diameter (inches)*	Roadway (inches)	Railroad (inches)
6 through 14	0.250	0.25
16	0.250	0.281
18	0.250	0.312
20	0.250	0.344
24	0.281	0.375
30	0.312	0.469
36	0.344	0.531
42	0.344	0.625
48	0.344	0.687
54		0.719
60		0.843
66		0.937
72		1.000

*Additional casing diameters are available.

Notes: Minimum thicknesses assume a minimum of 4.5 feet of cover over top of pipe.

C. Casing Diameter

The casing pipe should be sized to provide a minimum of 4 inches of clearance between the inside of the casing pipe and the largest outside diameter of the carrier pipe (including pipe bells) to allow for deflection of the casing pipe and installation of casing spacers.

Utility Cut Restoration

A. General

Utility cuts are made in existing pavement sections to install a myriad of utilities and to repair those that experience maintenance needs. Once a utility cut is made in the pavement, the restoration materials and process will have a significant impact on the life of the pavement patch. When a utility cut is made, the native material surrounding the perimeter of the trench is subjected to loss of lateral support. This leads to loss of material under the pavement and bulging of the soil on the trench sidewalls into the excavation. Subsequent refilling of the excavation does not necessarily restore the original strength of the soils in this weakened zone. The weakened zone around a utility cut excavation is called the "zone of influence." Poor performance of pavements over and around utility trenches on local and state systems often causes unnecessary maintenance problems due to improper backfill placement (i.e., under compacted, too wet, too dry). It has been reported that the life of a utility cut replacement patch is only 2 to 3 years. The costs of repairing poorly performing utility cut restorations can potentially be avoided with a better understanding of proper material selection and construction practices. In addition to the resources spent by the public agency to maintain the pavement patch area, there is a significant impact to the traveling public due to rough streets and the traffic interruptions that occur frequently when maintenance activities are occurring.

The improper use and placement of backfill materials and failure to provide for the loss of lateral support of the trench walls are the primary causes of pavement patch failure.

While planning of utility modifications can be accommodated as part of a larger project, frequently these excavations occur at odd-hours and with no advance notice to repair a facility (i.e., water main break). It is therefore important to plan ahead to help ensure that desirable methods are used to restore utility trenches, even when weather, timing, or other factors may be less than ideal.

B. Background

The Iowa Highway Research Board (IHRB) commissioned two projects focusing on how best to reconstruct utility trenches. The goal of the projects has been to mitigate the negative effects utility trenches have on the surrounding roadway pavement. The two studies are described below.

- IHRB Project TR-503 (2005) Utility Cut Repair Techniques - Investigation of Improved Cut Repair Techniques to Reduce Settlement in Repaired Areas
- IHRB Project TR-566 (2010) Utility Cut Repair Techniques - Investigation of Improved Utility Cut Repair Techniques to Reduce Settlement in Repaired Areas, Phase II

The above reports can be accessed at the following websites:

- www.intrans.iastate.edu
- www.iowadot.gov/operationsresearch/reports.aspx

The research identified the following problems with current trench restoration methods:

- Large equipment bearing on the trench edges (causing damage to the trench sidewalls and the remaining pavement)

- 2 to 4 foot lifts of backfill material
- Sporadic compaction of the backfill lifts
- Utilizing native, saturated material in the excavation in an attempt to clean the excavation site
- General lack of density and moisture quality control

The research identified three modes of failure for the utility trenches.

- Settlement of utility cut restoration, caused by poor compaction and wet/frozen conditions
- A "bump" forming over the restoration, resulting from uplift or settlement of surrounding soil
- Weakening of the surrounding soils

Many of the studied patches showed signs of failure within 2 years.

C. Factors Affecting Patch Performance

1. **Compaction:** Proper compaction of the non-manufactured backfill material is a critical element of good trench construction. Use of granular backfill has previously been thought of as a means to achieve an acceptable level of trench compaction with a minimal level of effort; however, that is not the case. Even with granular materials, the material should be placed in lifts not exceeding 12 inches in thickness. Each lift of granular material should receive an appropriate level of compactive effort to achieve a minimum relative density of 65%. If cohesive soils are used in the top 2 feet to match existing subgrade materials, the soil should be placed in 8 inch lifts and compacted to 95% of Standard Proctor Density for that soil.

   Backfill materials are often compacted using large compaction equipment, which is placed close to the edges of the cut, resulting in damage to pavement surfaces around the perimeter of the excavation. Note Figure 9D-1.01. It is important to keep equipment away from the edges of the trench.

Figure 9D-1.01: Cracking Pavement Surrounding the Utility Cut Area Because of Construction Equipment Getting Too Close to the Edge of the Open Cut

Source: IHRB Project TR-566

Figure 9D-1.02: Large Lift Thicknesses Used in Utility Cut Trench Backfill Material Placement

Source: IHRB Project TR-566

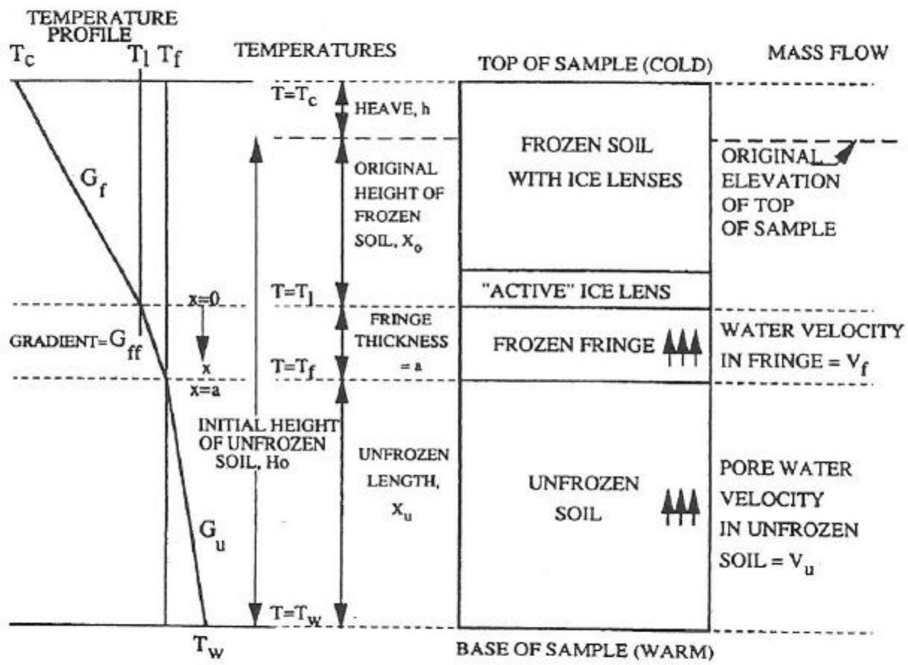
2. **Use of Appropriate Backfill Material:** Class I and Class II backfill material, according to SUDAS Specifications Section 3010, should be used to place backfill material in the trench. The Engineer can change the gradation to meet locally acceptable materials, including crushed concrete pavement. Backfill materials classified as SM (silty sands) or GC (clayey gravel) with less than 35% sand will achieve relative densities of dense to very dense without a significant amount of compaction. In addition to non-manufactured soils, manufactured products such as flowable mortar and controlled low strength material (CLSM) can be used. These manufactured products do not require compaction, but they do cost more.

3. **Frost Heave:** One of the major impacts to trench performance is seasonal effects such as frost heave. Frost is known to cause two main problems. First, frost changes the stiffness of the soil structure during freeze and thaw cycles. As frost forms, the pavement structure stiffens. When frost thaws, the increase in water content in the soil causes the pavement structure to weaken. Second, displacements are caused by the formation of ice lens and the pressure on related structures, which are normal to the growth of the lens.

   As frost forms in a soil, a frozen fringe develops (see Figure 9D-1.03). This fringe develops at freezing temperatures and extends downward. The frozen fringe forms below where an active ice lens is forming. As freezing temperatures penetrate deeper in a soil, the frozen fringe and zone of active lens formation also migrate downward. The active ice lens layer in the soil is also a boundary where the permeability of the soil decreases because the pores begin filling with ice. This boundary prevents water from traveling upward beyond the active lens zone. Because of this, no additional ice lens will form above the active ice lens zone. The downward movement of the frozen fringe affects the size of the ice lens. When the front advances rapidly through a soil, the lens will be thin; however, when the frozen fringe remains at a stationary point because of the heat flow balance, a larger lens will form.

   As ice lenses form, they exert an outward pressure on the pore. When the pressures are greater than the overburden pressures, the soil will heave. The heave (expansion of soil) occurs at the frost line, which is assumed to be at freezing. Frozen soils above the frost line do not expand because there is no influx of moisture.

Figure 9D-1.03: Frost Heave in Idealized One-Dimensional Soil Column



Source: IHRB Project TR-566

Soils do not need to be saturated to experience frost formation. Conversely, when a soil is at freezing temperatures, not all of the water is frozen.

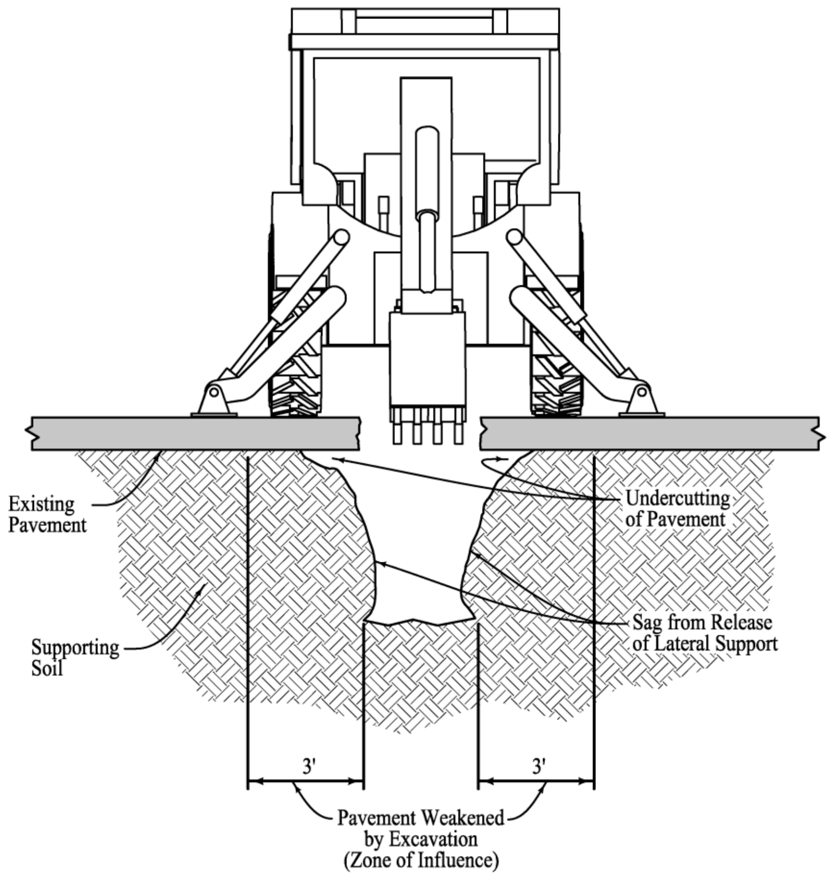
The frost susceptibility of a soil is a function of particle and void size. Gravels have large voids and large particles. This allows water to flow freely through the soil. When water does freeze in gravel, the void size is large enough so that as the frozen fringe passes through, the lens cannot grow large enough to displace the particles. Clays, at the other end of the gradation chart, have very small particles and small voids. Less force is required to displace smaller soil particles than larger particles, such as gravels. This make soils with high percentages of fines more susceptible to frost.

4. **Zone of Influence:** The zone of influence is an area immediately adjacent to and outside the trench where the soils are adversely affected by the nearby excavation. The effects in this zone are caused by changes to the soil that remains in place during a trench excavation. The trench walls, usually close to vertical, are not capable of withstanding their normal loads and tend to experience sloughing due to loss of lateral support. In all cases, there is subsidence that creates a weakened plane in the soil. This weakened plane, if it is not addressed, is one of the primary reasons for trench/pavement patch failure. This can lead to undercutting of the existing pavement that was original planned to remain in place. These effects are detailed in Figure 9D-1.04. The damage to the supporting nature of the soils is very difficult to repair and results in an isolated weak column of soil surrounding the entire excavation.

The research has pointed out that the zone of influence can extend between 2 and 3 feet from the top of the trench wall. Thus, prior to replacement of the pavement, the area should be cut back a minimum of 3 feet and to a depth of 2 feet. This top area should then be filled with backfill materials matching the existing subgrade/subbase to provide uniform pavement support.

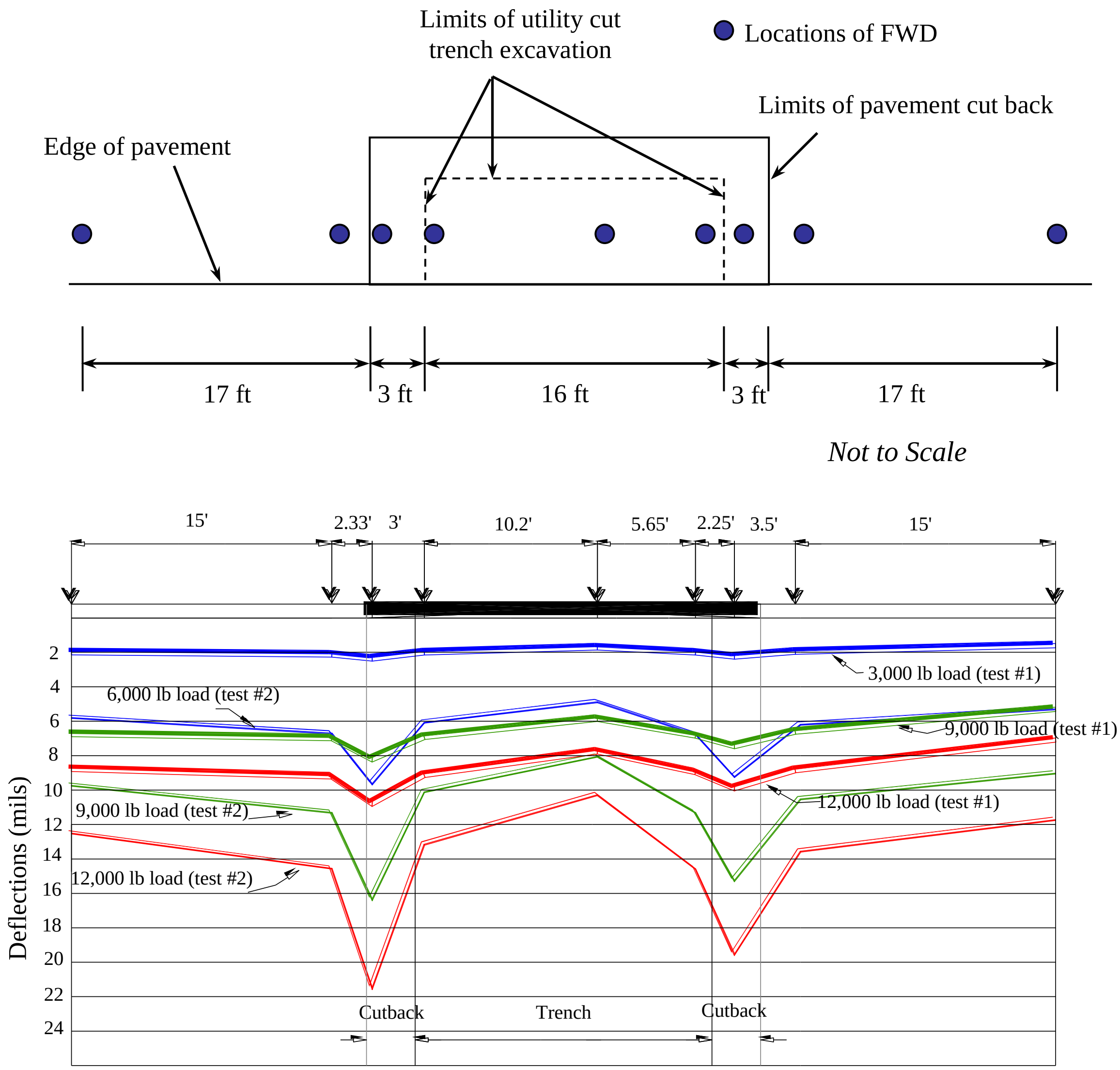
As can be seen in Figure 9D-1.05, the zone of influence, in this case approximately five to six feet in width, performed significantly poorer than both the existing, undisturbed pavement section, and the properly constructed trench backfill. This increased deflection implies that the pavement is moving relative to the surrounding pavement, increasing the likelihood of crack formation and premature pavement deterioration.

Figure 9D-1.04: Overstressing of the Pavement and Natural Materials Adjacent to the Trench



Source: IHRB Project TR-503

Figure 9D-1.05: Locations and Results of FWD Tests Performed at a Utility Cut Location Showing Deflection Within the Zone of Influence



Note: 1) 1 mil=0.001 inches
2) Deflections reflect point where FWD drops weight
3) Weights were dropped on centerline of trench in the longitudinal direction of the pavement

Source: IHRB Project TR-566

The charts above were recorded using a Falling Weight Deflectometer (FWD). This tool, providing a non-destructive measurement, drops a weight from a controlled height and measures the resulting pavement deflection using an array of sensors placed along a line. This line is usually orientated along the direction of travel for the roadway. Figure 9D-1.07 shows the Iowa DOT's FWD.

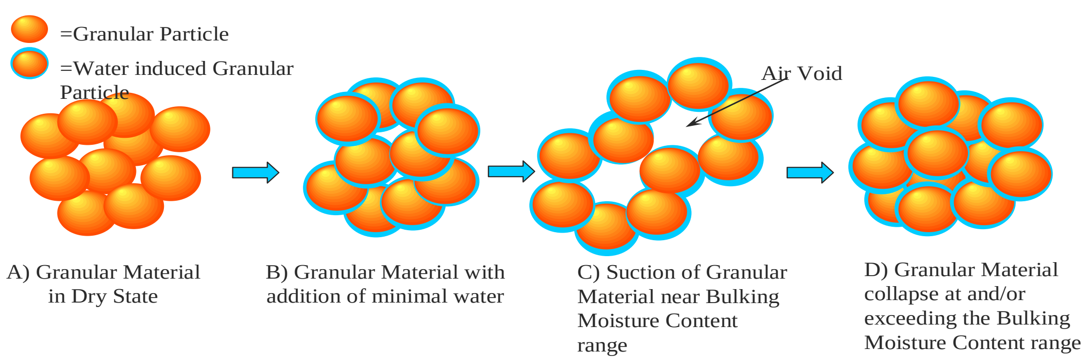
Figure 9D-1.06: Iowa DOT FWD Equipment Showing Sensor Configuration

Source: IHRB Project TR-566

5. **Moisture Content of Backfill Material and Collapse Potential:** It is critical that the bulking moisture content of the granular material be exceeded in order to achieve a dense backfill condition. Bulking is a phenomena that occurs in most granular materials in which the capillary action between soil particles that are surrounded by water hold the particles together in a honeycombed structure as noted below. The material starts out dry. With the addition of some water, the soil particles are surrounded. As more water is introduced, generally to a moisture content of approximately 6% to 10%, suction forms between the soil particles that creates tension and air voids. With the addition of more water, generally above 10%, the tension is released. This rearrangement of particles is referred to as collapse of granular materials. This phenomena is shown in Figure 9D-1.07.

   When the tension releases, the collapse occurs, leading to a denser material, and reduced trench settling. Thus, it is very important that the granular backfill materials be placed in the trench above the bulking moisture content. The bulking moisture content range (i.e., the range of moisture contents to avoid can be determined in the laboratory and the range is defined as the moisture content for the maximum collapse potential plus or minus 2%.

Figure 9D-1.07: Bulking Moisture Content



Source: IHRB Project TR-503

6. **Quality Control:** If granular materials are used for the primary backfill material as recommended, the level of density must be checked and compared to the relative density of the material. Use of Standard Proctor Density does not provide an appropriate level of results for the density of granular materials. See Chapter 6 - Geotechnical for more information on relative density and Proctor density. Use of the Dynamic Cone Penetrometer (DCP), which involves measurements of penetration of the rod into the backfill material, is another method of determining density. The greater the number of blows to penetrate a given depth, the stiffer the material. The Clegg Hammer operates on the same general principle.

D. Recommended Utility Trench

The recommended trench configuration and recommended best practices to increase performance of pavement patches over utility trench repairs are shown below. It is important to note that all of these recommendations must be implemented in order to have improved pavement life.

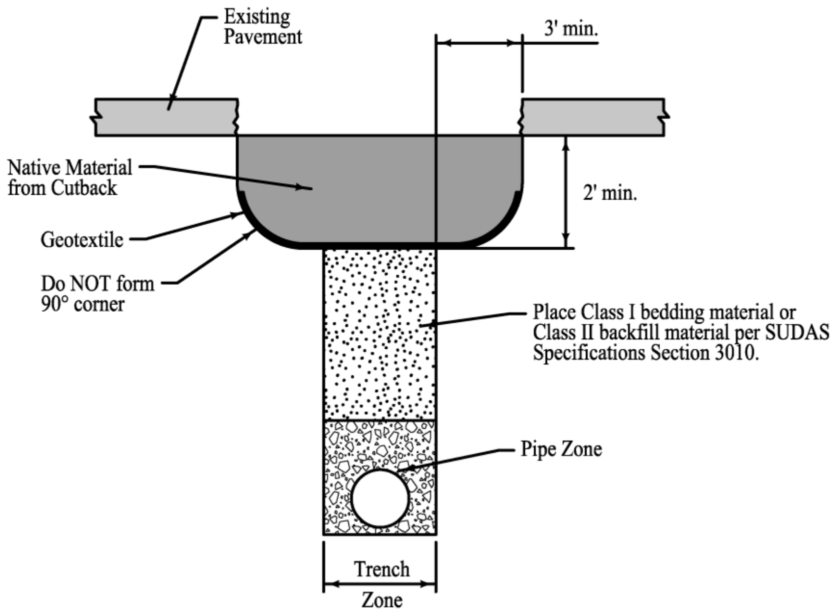
1. **Equipment:**
   - Throughout construction of the trench, including excavation and placement of backfill material, keep equipment and materials as far away from the trench area as possible to minimize trench wall sloughing.
   - The smallest equipment which can satisfactorily perform the job should be used to minimize the adverse effects caused by equipment loading near the edges of the excavation.

2. **Trench Excavation:**
   - Soils excavated from the trenches or other soils should not be mixed with the granular backfills unless previous laboratory testing yielding a range of recommended moisture content and densities to be achieved in the field are conducted.
   - The T-section should be modified to use walls that are beveled outward to facilitate compaction of backfill. Beveled edges will reduce the amount of disturbance to the surrounding soil and also eliminate the vertical excavation, which makes compacting the backfill more difficult.
   - Refrain from using saturated material from the excavation.

3. **Backfill:**
   - Reduce lift thickness to 8 inches to 12 inches for backfill materials
   - The standard vertical-walled cross-section with 1 inch clean limestone is recommended as a construction practice.
   - Quality control measures should be implemented in the field. These should include methods to ensure compaction and moisture requirements are met. This includes achieving at least medium relative density with moisture contents above the bulking moisture content for cohesionless soils and above 95% of Standard Proctor and ± 2% of optimum moisture content for cohesive soils.
   - Place geotextiles in the bottom of the cutback area prior to placement of subgrade/subbase material.

4. **Pavement Surfacing:**
   - Saw the pavement full depth to create the cutback area three feet from edge of the original trench.
   - Recompact the top 12 inches of trench backfill material after removal of cutback materials.
   - PCC patches seem to perform better regardless of the existing pavement type. This is due in part to the difficulties in completing uniform compaction of HMA in relatively small areas.

Figure 9D-1.08: Recommended Trench Reconstruction



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CHAPTER 10
Street Tree Criteria

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General Information

A. Concept

A street tree is any tree with a trunk located 50% or more within the land lying between property lines on either side of all public streets, boulevards, and alleys, including public easements. With narrower rights-of-way and the increasing use of underground utilities, the available space within a public right-of-way to plant trees is diminishing. Placement of trees outside of the public right-of-way on private property will still maintain all of the aesthetic and environmental advantages of trees. In addition, the placement outside of the public right-of-way will prevent future complications of sight distance, utility conflicts, and construction conflicts. Thus it is recommended that new trees not be placed in the public right-of-way.

If trees are placed in the public right-of-way, the principal considerations in design of the placement of street trees are their relation to horizontal and vertical clear zones. No street tree should be placed in the horizontal clear zone or triangular sight distance as described in Chapter 5. The minimum vertical clearance for mature trees should be 14 feet above the street grade, 10 feet above recreational trails, and 8 feet above sidewalks. Special considerations must be given to clearances to overhead utility lines, driveways, traffic signs, and underground utilities. If at all possible, street trees should not be placed over buried utilities (public or private).

B. Conditions

1. **Design Standards:**

    a. SUDAS Design Manual

    b. Recognized design publications for street trees

    c. In case of a conflict between the above design standards, the Jurisdictional Engineer should be contacted for clarification

2. **Construction Standards:** Use the most recent edition of the SUDAS Standard Specifications together with the latest contract supplementary information.

3. **Project Submittals:** If street trees are allowed by the Jurisdiction and if project submittals are required, a street tree planting layout showing the quantity, species/cultivar, and location of all trees must be submitted for review. This plan is to be approved by the Jurisdiction prior to the tree planting and a permit issued if the proposed trees are within the public right-of-way.

4. **Ownership:** If the tree is located in the public right-of-way or publicly owned property, Section 364.12 of the Iowa Code requires the Jurisdiction to remove deadwood or diseased trees. If the street tree(s) are located outside of public property or right-of-way, the responsibility and ownership is that of the landowner.

5. **Establishment and Warranty Periods:** The establishment period is 1 year after the installation has been accepted by the Engineer. Care and maintenance of all plants will be the responsibility of the Contractor during that time. The Engineer has the option to include an additional year, which is called the warranty period. If specified, the warranty period begins immediately after the establishment period and continues for another year. Check with the Jurisdiction for their requirements.

Street Tree Design

A. Area Requirement per Tree

At least 9 square feet of ground is required for each tree and the trunk of street trees should be no closer than 2.5 feet from impervious surface material.

B. Spacing

For planning purposes, the ideal spacing should be 50 feet apart or no closer than the distance of their full spread from the next tree in the parking. Spacing as close as 30 feet may be allowed by the Jurisdiction for species/cultivars or ornamental trees that have appropriate mature branch spreads.

C. Location within Public Right-of-way

The following criteria are for the location of street trees that are located in the street right-of-way. Jurisdictions may require additional street right-of-way to provide clearances to underground or overhead utilities. The mature tree trunk size should be taken into account when placing the tree. The criterion does not include street trees located within medians. Special designs that meet the required clear zone must be used when locating trees within medians.

1. Minimum distance of 5 linear feet from water service stop boxes.
2. Minimum distance from the edge of the traveled way according to Chapter 5 - Roadway Design.
3. Minimum distance of 10 linear feet from hydrants, poles, transformers, telephone junction boxes, manholes, and driveway approaches.
4. Minimum distance from street lights of 25 linear feet or the width of spread of the mature tree, whichever is greater.
5. In central business districts where traffic speeds are low, a minimum distance of 3 feet from the back of curb should be used for street trees if a minimum distance of 8 feet exists for right-of-way from the back of curb.
6. No trees should be in the horizontal clear zone or triangular sight distance area. (See Chapter 5 - Roadway Design).
7. Do not plant street trees in any public right-of-way that has less than 12 feet from back of curb or edge of pavement to the property line on each side of the street.

D. Tree Size

Street trees should be a minimum of 1 inch diameter for ornamental and 1 1/2 inch diameter for shade trees or as specified and measured at 6 inches above grade after planting unless smaller trees are allowed.

All underground utilities or any other improvements, either private or public, will be located before excavation is done. Information concerning contacting Iowa One Call will be included in the contract documents. The Iowa One Call phone number is 811 or 800-292-8989.

E. Selection of Trees

The species of trees listed are recommended for street tree use. Note: Where it is not recommended that any trees be planted under overhead utility lines, some jurisdictions may allow plantings of low growing trees. Other species, or different varieties of the listed species, may be used with approval of the Jurisdiction. Certain species listed may not be allowed by all Jurisdictions.

Table 10B-1.01: Selection of Trees*

Common Name	Genus Name	Minimum Spacing (feet)	Mature Height (feet)	Mature Spread (feet)
European Hornbeam**	Carpinus betulus	40	40	30
Hackberry	Celtis occidentalis	40	75	50
Ginkgo (male only)	Ginkgo biloba	50	60	35
Sycamore	Platanus occidentalis	40	100	50
Callery Pear	Pyrus calleryana	35	60	60
American Hophornbeam** (Ironwood)	Ostrya virginiana	25	40	20
Maple				
Freeman Maple	Acer X freemanii	30	50	45
Norway Maple	Acer platanoides	65	15	30
Black Maple	Acer nigrum	40	65	60
Sugar Maple	Acer saccharum	45	80	50
Greencolumn Maple	Acer nigrum 'Greencolumn'	25	50	20
Honeylocust				
Honeylocust, Thornless Skyline Honeylocust	Gleditsia triacanthos i. cv.	30	60	30
Moraine Honeylocust	Gleditsia triacanthos i. cv.		60	40
Imperial Honeylocust	Gleditsia Triacanthos var. inermis 'Imperial'	30	25	30
Shademaster Honeylocust	Gleditsia Triacanthos var. inermis 'Shademaster'	40	45	40
Oak				
Swamp White Oak** (High PH sensitive)	Quercus bicolor	50	75	60
Northern Red Oak	Quercus rubra	50	75	70
Burr Oak	Quercus macrocappa	40	75	50
English Oak	Quercus robur	55	75	50
Scarlet Oak** (High PH sensitive)	Quercus coccinea	50	60	50
Linden				
American**	Tilia americana	35	70	45
Littleleaf**	Tilia cordata	30	50	35
Silver	Tilia tomentosa	50	50	40
American	Tilia americana 'Fastigiata'	30	50	30
Greenspire	Tilia cordata 'Greenspire'	30	45	30
Crimean	Tilia x euchlora	35	30	60

* Monoculture plantings may result in insect problems
** Salt Sensitive

Table 10B-1.01: Selection of Trees (Continued)

Common Name	Genus Name	Minimum Spacing (feet)	Mature Height (feet)	Mature Spread (feet)
Crabapple* **				
Adams	Malus 'Adams'	25	20	20
Adirondack	Malus Adirondack	20	18	10
Pink Spires	Malus 'Pink Spires'	15	12	10
Snowdrift	Malus 'Snow Drift'	20	20	15
Spring Snow	Malus 'Spring Snow'	20	20	15
White Candle	Malus 'White Candle'	12	18	8

***Dwarf species

F. Trees that Should Not be Planted in Public Right-of-way

American Elm
Mulberry
Black Locust
Russian Olive
Bolleana Poplar
Conifers
Silver Maple

Box Elder
European Mountain Ash
Catalpa
Tree of Heaven
Weeping Birch
White Ash

Cotton-Bearing Cottonwood
White Poplar
Willows
Austrian Pine
Lombardy Poplar
Green Ash

G. Guideline for Selection of Nursery Trees

1. There should be no roots greater than 1/10 the trunk diameter circling more than one-third the way around in the top half of the root ball. Roots larger than this may be cut provided they are smaller than one-third the trunk diameter. There should be no kinked roots greater than 1/5 the trunk diameter. Roots larger than this can be cut provided they are less than one-third the trunk diameter.

2. Plants should be in a healthy, vigorous condition and essentially free of dead or broken branches, scars that are not completely healed, frost cracks, disfiguring knots, broken or abraded bark, redundant leaders or branches, rubbing branches or aberrations of any kind. Plants should not have multiple leaders, unless that is their natural form.

3. Ensure trees are rooted into the root ball so that soil or media remains intact and trunk and root ball move as one when lifted. The trunk should bend when gently pushed, not pivot at or below soil line.

4. The point where the top-most root in the root ball emerges from the trunk should be visible at the soil surface.

5. Comply with ANSI Z60.1 for the relationship between caliper, height, and root ball size, as shown in Table 10B-1.02.

6. There should be one dominant leader more-or-less straight to the top of the tree with the largest branches spaced at least 6 inches apart. There can be a double leader in the top 10% of the tree.

7. The tree canopy should be symmetrical, free of large voids, and typical of the species or cultivar. Live crown ratio (distance from bottom of canopy to tree top/tree height) should be at least 60%.

8. Branches should be less than 2/3 the trunk diameter, free of bark inclusions, and more-or-less radially distributed around the trunk.

9. Trees greater than 1 1/2 inches caliper should be able to stand erect without a supporting stake.

10. Ensure the trunk and main branches are free of wounds (except for properly-made pruning wounds), damaged areas, conks, bleeding, and signs of insects or disease.

11. In areas near overhead utility lines, the mature height of the tree should be a minimum of 10 feet lower than the overhead lines.

12. If any of the above conditions are not met, trees may be rejected.

Table 10B-1.02: Caliper/Rootball/Height Relationship

Caliper (inches)	Average Height (feet)	Minimum Rootball Diameter (inches)
1	8 to 10	16
1 1/2	10 to 12	20
2	12 to 14	24
2 1/2	12 to 14	28
3	14 to 16	32
3 1/2	14 to 16	38
4	16 to 18	42

Source: American Standard for Nursery Stock (ANSI Z60.1)

H. Staking of Trees

Depending on the size of the trees identified to be planted, the Jurisdictional Engineer should designate if staking is required. Generally if plant stock is delivered with well developed root balls, and if properly planted, it will not require staking.

References

The following references can be found from ISU Extension (www.extension.iastate.edu):

1. Guidelines for Selecting Trees
2. Street Trees for Iowa
3. Low-Growing Trees for Urban and Rural Iowa

The following references can be found from ISU Forestry Extension (www.extension.iastate.edu/forestry/publications/list.html):

1. Identification of Conifer Trees in Iowa
2. Soils and Trees
3. Tips for Proper Planting of Containerized Trees
4. Transplanting Trees and Shrubs
5. Watering Newly-Planted Trees
6. Preventing Construction Damage to Trees

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CHAPTER 11
Street Lighting

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Chapter 11 - Street Lighting

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General Information

A. General

Darkness brings increased hazards to users of urban streets because it reduces the distance they can see. The nighttime fatal accident rate on unlighted roadways is about three times the daytime rate, based on proportional vehicular miles of travel. This ratio can be reduced when proper fixed street lighting systems are installed.

Good visibility under night conditions is one of the fundamental requirements enabling motorists to move on roadways in a safe and coordinated manner during the nighttime hours. Properly designed and maintained street lighting will produce quick, accurate, and comfortable visibility at night, which will safeguard, facilitate, and encourage both vehicular and pedestrian traffic. Other objectives of street lighting include:

- Improvement of traffic flow at night by providing light, beyond that provided by vehicle lights, which aids drivers in orienting themselves, delineating roadway geometries and obstructions, and determining relationship to other motorists.
- Aid in police protection and enhanced sense of personal security.
- Promotion of business commerce and the use of public facilities during the nighttime hours.

Street lighting design is concerned with the selection and location of lighting equipment to provide improved visibility and increased safety while making the most efficient use of energy with minimum expenditure. This chapter focuses on the street lighting design approach of urban local, collector, and arterial streets. This chapter does not include guidelines for rural or freeway roadway types.

This chapter makes use of state-of-the-art lighting science and internationally and nationally recommended street lighting design practices to facilitate the quality and energy efficient design of street lighting on Iowa's urban roadways. This design guidance relies on roadway lighting guidelines issued by the Illuminating Engineering Society of North America (IESNA). IESNA is considered the nation's technical authority on illumination. The independent, member-based professional organization synthesizes research, investigations, and discussions to develop lighting design recommendations intended to promote good lighting practice. Many of the items in this chapter are references from ANSI / IESNA RP-8-00, *American National Standard Practice for Roadway Lighting,* (RP-8) publication, reaffirmed in 2005.

B. Industry Outlook

At this time, high pressure sodium (HPS) is the predominant type of light source used for street lighting. It is still viewed in many states, including Iowa, as the industry standard for energy efficient street lighting and is used as the benchmark in qualifying other types of lighting sources as energy efficient. However, the practical production of light from energy such as electricity is currently undergoing significant technological change.

On the forefront of the technological development is the advent and use of the light emitting diode (LED) for street lighting purposes. LEDs themselves are currently not as efficient at turning electrical power into light compared to HPS lamps. However, because of the difference in construction of and the control of light from an LED luminaire, LED luminaires used in street lighting practical applications are realizing energy efficiencies greater than HPS. Also, LEDs are expected to last longer than current lighting sources. Although the approach, appropriate use, and performance standards of LEDs are still being developed by professionals in the lighting industry, it is widely agreed upon that the technology is here to stay. This chapter will touch upon the design considerations, advantages, and potential disadvantages of LED lighting.

As will be discussed in Section 11C-1, there are two basic concepts of lighting design, the illumination concept and the luminance concept. The current edition of RP-8 discusses and supports both design concepts. There are currently discussions by lighting professionals whether the next edition of RP-8 will transition to and favor the luminance concept. However, at this time the illumination design method remains predominant in the United States. For the purpose of this chapter, the illumination method will be the only design concept discussed.

C. Iowa Code

Although there are many options for the type of luminaire to be used for street lighting projects, Iowa Code states all new or replaced luminaires shall be replaced with high pressure sodium lighting or lighting with equivalent or better energy efficiency. Following are excerpts from the current Iowa Code that pertain to publicly owned exterior lighting. Many of the lighting terms used in the following cited Iowa Code sections will be defined in the definition list and detailed further in this chapter.

1. **Facilities Owned By Cities:** Iowa Code Section 364.23 below pertains to facilities owned by cities. It is understood the reference to “era or period lighting” is in relation to architectural or ornamental lighting of historical significance, often found in downtown locations.

   “**364.23 Energy-efficient Lighting Required:** All city-owned exterior flood lighting, including but not limited to street and security lighting but not including era or period lighting which has a minimum efficiency rating of fifty-eight lumens per watt and not including stadium or ball park lighting, shall be replaced, when worn-out, exclusively with high pressure sodium lighting or lighting with equivalent or better energy efficiency as approved in rules adopted by the utilities board within the utilities division of the department of commerce. In lieu of the requirements established for replacement lighting under this section, stadium or ball park lighting shall be replaced, when worn-out, with the most energy-efficient lighting available at the time of replacement which may include metal halide, high-pressure sodium, or other light sources which may be developed.”

2. **Facilities Owned By Public Utilities:** Iowa Code Section 476.62 below pertains to facilities owned by public utilities.

   “**476.62 Energy-efficient Lighting Required:** All public utility-owned exterior flood lighting, including but not limited to street and security lighting, shall be replaced when worn-out exclusively with high pressure sodium lighting or lighting with equivalent or better energy efficiency as approved in rules adopted by the board.”

3. **Utilities Board Rules:** Iowa Administrative Code (IAC) 199-35.15 (476) contains the rules adopted by the Utilities Board within the Utilities Division of the Department of Commerce that are referenced in the two Iowa Code sections stated above and pertain to exterior lighting energy efficiency. It is understood one of the five conditions of IAC 199-35.14(476) must be met in order to use a light source other than high-pressure sodium for exterior lighting applications.

"**199-35.15(476) - Exterior Flood Lighting**

35.15(1) - Newly Installed Lighting: All newly installed public utility-owned exterior flood lighting shall be high-pressure sodium lighting or lighting with equivalent or better energy efficiency.

35.15(2) - In-service Lighting Replacement Schedule: In-service lighting shall be replaced with high-pressure sodium lighting or lighting with equivalent or better energy efficiency when worn out due to ballast or fixture failure for any other reason, such as vandalism or storm damage. A utility shall file with the board as part of its annual report required in 199-Chapter 23 a report stating progress to date in converting to high-pressure sodium lighting or lighting with equivalent or higher energy efficiency.

35.15(3) - Efficiency Standards: Lighting other than high-pressure sodium has equivalent or better energy efficiency if one or more of the following can be established:

a. For lamps less than 120 watts, the lumens-per-watt lamp rating is greater than 77.1, or
b. For lamps between 120 and 500 watts, the lumens-per-watt lamp rating is greater than 96, or
c. For lamps greater than 500 watts, the lumens-per-watt lamp rating is greater than 126, or
d. The new lighting uses no more energy per installation than comparable, suitably sized high-pressure sodium lighting, or
e. The new lighting consists of solid-state lighting (SSL) luminaries that have an efficacy rating equal to or greater than 66 lumens per watt according to a Department of Energy (DOE) Lighting Facts label, testing under the DOE Commercially Available LED Product Evaluation and Reporting Program (CALiPER), or any other test that follows Illuminating Engineering Society of North America LM-79-08 test procedures."

Prior to the fall of 2010, the language in IAC 199-35.15(3) was different and used strictly the bare lamp efficacy rating of HPS lamps as the basis of comparison to qualify other lighting source types as energy efficient. Because of the way LED lighting is constructed and produces light, the IAC excluded the use of LED lighting even though it could be demonstrated that in many street lighting applications, current LED lighting was more energy efficient. Therefore, the IAC was revised in the fall of 2010 to the language shown above. The IAC still sets HPS lighting as the energy efficient standard; however, other lighting source types can be used if they pass one of the five stated conditions. The first three conditions (a, b, and c) are a modification from the IAC prior to 2010 and generally apply to high intensity discharge (HID) or other single-lamp type luminaires.

Condition 'd' is intended to apply to lighting replacement or retrofit applications. Again, the IAC uses HPS as the comparison standard. Bear in mind the condition says "suitably sized" HPS lighting. For a defined project area, this requires the designer to compare the energy consumption of the proposed lighting system type (other than HPS) to the energy consumption of HPS lighting if it is properly applied meeting the same illumination criteria. The designer should be forewarned to not necessarily use the existing lighting system, particularly if it is HPS, as the basis of energy consumption for the replacement project because the project area may be over lit by the existing lighting. It is generally understood that the illumination criteria published in RP-8 for roadway lighting is to be used in the comparison process.

Condition 'e' is intended to apply strictly to LED lighting when installed in new lighting project applications. This requires the luminaires to have a luminaire efficacy rating of at least 66 lumens per watt as established by a proper industry testing procedure.

D. Definitions

Average Maintained Illuminance: The average level of horizontal illuminance on the roadway pavement when the output of the lamp and luminaire is diminished by the maintenance factors; expressed in average footcandles for the pavement area.

Ballast: A device used with an electric-discharge lamp to obtain the necessary circuit conditions (voltage, current, and wave form) for starting and operating the lamp.

Bracket or Mast Arm: An attachment to a lighting standard or other structure used for the support of a luminaire.

Candela (cd): The unit of luminous intensity. Formerly the term "candle" was used. Refer to Figure 11A-1.01.

Coefficient of Utilization Curve (CU): This curve shows the percentage of the total light output that will fall on the roadway. Mounting height and horizontal dimensions transverse to the roadway relative to the luminaire position must be known to apply the curve. Refer to Figure 11C-1.02.

Efficacy (Luminous Efficacy): The quotient of the total luminous flux delivered from a light source divided by the total power input to the light source. It is expressed in lumens per watt (l/w).

Footcandle (fc): One footcandle is the illumination incident on a surface one square foot in area on which there is uniformly distributed a luminous flux of one lumen. Footcandle is the English unit for illumination. The metric or SI unit is lux. One footcandle equals 10.76 lux. Refer to Figure 11A-1.01.

Foot-lambert (fl): The unit of photometric brightness (luminance). It is equal to $1/\pi$ candela per square foot. One foot-lambert equals 3.426 candelas per square meter.

High Intensity Discharge (HID): A term applied to a category of electric lamps that produce light by means of an electric arc sustained between tungsten electrodes housed inside a translucent or transparent fused quartz or fused alumina arc tube filled with gas and metal salts. The gas facilitates the arc's initial strike. Once the arc is started, it heats and evaporates the metal salts forming a plasma, which greatly increases the intensity of light produced by the arc and reduces its power consumption. High intensity discharge lamps are a type of arc lamp.

Horizontal Footcandle: One lumen distributed uniformly over a horizontal surface one square foot in area. Thus, horizontal footcandle is a measure of illumination from light that strikes a horizontal surface such as the pavement.

Illuminance: The density of the luminous flux incident on a surface. It is the quotient of luminous flux by area of the surface when the latter is uniformly illuminated (measured in footcandles). Refer to Figure 11A-1.01

Initial Lamp Lumens: Manufacturer's published initial bare lamp lumen output of a new lamp.

Isocandela Diagram: A series of lines plotted in appropriate coordinates to show directions in space at which the candlepower is the same.

Isofootcandle Diagram: This diagram is available from the manufacturer of the light source and shows the horizontal footcandles on the pavement surface at various points away from the source. Mounting height must be known to properly use the diagram. Refer to Figure 11C-1.02.

Lamp Lumen Depreciation Curve (LLD): This curve gives information on the relationship between length of service and light output. All lamps deteriorate with time, and total light output becomes less. Refer to Figure 11B-1.01.

Lamp: A generic term for a man-made source of light that is produced either by incandescence or luminescence.

Lighting Standard: The pole with or without bracket or mast arm used to support one or more luminaires.

Lighting Unit: The assembly of pole or standard with bracket and luminaire.

Longitudinal Roadway Lines (LRL): A set of horizontal lines running parallel to the curb line or edge of pavement that establish a coordinate system for roadway lighting analysis. Refer to Figure 11B-1.03.

Lumen (lm): A unit of measure of luminous flux or flow of light from a light source. One lumen is the luminous flux emitted within a unit solid angle (one steradian) by a point source having a uniform luminous intensity of one candela. Refer to Figure 11A-1.01.

Luminaire: A complete lighting assembly consisting of a lamp or lamps together with the parts designed to distribute the light, to position and protect the lamps, and to connect the lamps to the power supply.

Luminaire Dirt Depreciation Curves (LDD): These curves give information on the relationship of light output depreciation due to accumulated dirt on the luminaire and lamp optical surfaces. An estimate of the dirt level present in the environment is needed to apply the curves. Refer to Figure 11B-1.02.

Luminance (L): The luminous intensity of a surface in a given direction per unit of projected area of the surface as viewed from that direction (measured in foot-lamberts).

Lux (lx): The International System (SI) unit of illumination. One lux is the illumination incident on a surface one square meter in area on which there is uniformly distributed a luminous flux of one lumen. One lux equals .0929 footcandle.

Maintenance Factor (MF): A depreciation factor that is the product of the Lamp Lumen Depreciation Factor (LLD) and the Luminaire Dirt Depreciation Factor (LDD). This factor is applied to the initial average footcandles to account for dirt accumulation and lamp depreciation at some predetermined point after installation.

Mean Lamp Lumens: Average quantity of light output (lumens) over the life of the lamp. High pressure sodium, LED, and incandescent lamps are measures for mean lumens at 50% of lamp life. Fluorescent and metal halide lamps are measured for mean lumens at 40% of rated lamp life.

Mounting Height (MH): The vertical distance between the roadway surface and the center of the apparent light source of the luminaire (fixture elevation relative to the roadway surface).

Nadir: A point directly below an observer or object. In lighting, the point vertically below a luminaire's lamp source center with the luminaire mounted in standard position with zero tilt or roll. Refer to Figure 11B-1.05.

Overhang: The transverse horizontal dimension of the position of the luminaire relative to the edge of the roadway or back of curb of the street. Positive overhang is in the direction toward the street center. Negative overhang is in the direction away from the street center.

Roadway Width: The curb to curb width for urban roadway sections and edge to edge pavement width for rural roadway sections.

Steradian: The unit measure of solid angle defined as the conical or pyramid shape that subtends an area on a sphere surface equal to the radius squared. Refer to Figure 11A-1.01.

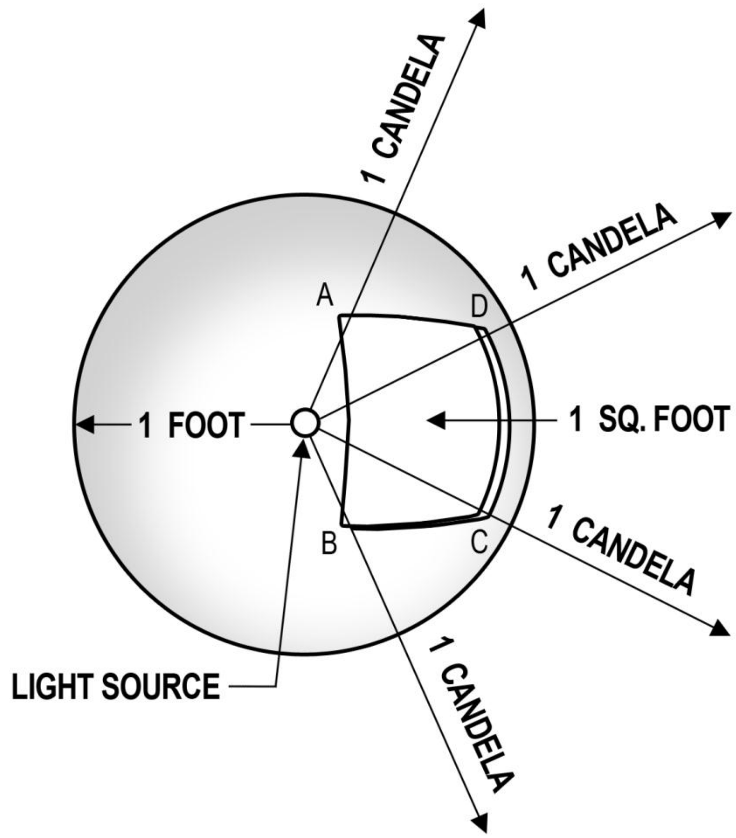
Spacing: The distance between successive lighting units measured longitudinally along the centerline of the roadway.

Transverse Roadway Lines (TRL): A set of horizontal lines running perpendicular to the curb line or edge of pavement that establish a coordinate system for roadway lighting analysis. Refer to Figure 11B-1.03.

Vertical footcandle: One lumen distributed uniformly over a vertical surface one square foot in area. Thus, vertical footcandle is a measure of illumination from light that strikes a vertical surface such as curbs, piers, retaining walls, or other objects with a vertical surface.

Watt: The measure of power or the rate of flow of energy per time. One watt equals the flow of one joule of energy per second. Watts are also equivalent to volts multiplied by amps.

Figure 11A-1.01: Lighting Units Definition Diagram



Relationship between candelas, lumens, and footcandles: A uniform point source (luminous intensity or candlepower equal to one candela) is shown at the center of a sphere of unit radius whose interior surface has a reflectance of zero. The illuminance at any point on the sphere is one footcandle (one lumen per square foot) when the radius is one foot. The solid angle subtended by the area A,B,C,D is one steradian. The flux density is therefore one lumen per steradian, which corresponds to a luminous intensity of one candela as originally assumed. The sphere has a total area of 4π (or 12.57) square feet and there is a luminous flux of one lumen falling on each unit area. Thus, the source provides a total of 12.57 lumens.

Source: Adapted from *ANSI / IES RP-8-00* (R2005)

E. References

Illuminating Engineering Society of North America. *American National Standard Practice for Roadway Lighting*. ANSI / IENSA RP-8-00, (R2005).

Iowa Administrative Codes, 2011.

Luminaires

A. Lighting Sources

Since the development of street lighting, there have been many electrical lighting source types used to illuminate public streets, the first being the incandescent lamp. Other lamp types developed over time were fluorescent, and high intensity discharge (HID) types such as mercury vapor, metal halide (MH), low pressure sodium, and high pressure sodium (HPS). Most recently the solid state light emitting diode (LED) has become a viable choice because of its efficiency to create and apply light in street applications. Because of the enactment of the Iowa Code in 1989 mandating outdoor lighting efficiency, the Code revisions in 2010, and other application considerations, the most practical choices today are metal halide, high pressure sodium, and LED. For a comparison of these source types, refer to Table 11B-1.01.

Table 11B-1.01: Typical Street Lighting Performance Values

Lamp Type and Wattage	Initial Lamp Lumens	Lamp Efficacy (l/w)	Lamp and Ballast Watts	Lamp and Ballast Efficacy (l/w)	Luminaire Optical Efficiency (%)	Overall System Efficacy (l/w)	Average Life (hrs)
250W High Pressure Sodium (HPS)	28,000	112	295	95	85	80.7	24,000
250W Metal Halide (MH)	21,500	86	285	75.5	85	64.2	20,000
180W Light Emitting Diode (LED)	13,100	73	204	64.5	--	64.5	70,000

For HPS and MH, the performance of the light source will vary with wattage size. Typically, the larger the size, the better the efficacy or lumens per watt of the lamp. This is not the case for LED luminaires. Since an LED light assembly is comprised of multiple small LED lamps each having the same efficacy and larger LED luminaires just contain more of the same individual lamps, the efficacy ratio tends to remain the same over the luminaire size range.

For comparison purposes, the table contains a 250 watt HPS lamp, a 250 watt MH lamp, and a 180 watt LED luminaire. The 180 watt LED size was chosen based on application experience. This LED luminaire puts out less total lumens than either of the other two, but because of superior optical efficiency and control, this size luminaire will produce similar street illumination results as a 250W HPS luminaire.

The efficacy ratio suffers as all of the luminaire losses are considered. For the HPS and MH cases, the initial efficacy is based on the lamp input wattage. The efficacy ratio drops when the ballast wattage is included in the calculation. The efficacy ratio drops again when the inherent lumen losses of the luminaire optics are considered. LEDs are rated differently. The initial lumen output is that measured from the entire luminaire assembly at the outset. Therefore, this value has already considered the lamp intensity and any luminaire optical losses. Only the driver wattage needs to be included to arrive at the overall system efficacy.

Another comparison is that LEDs are projected to last significantly longer than HPS or MH. The 70,000 hour life equates to almost 16 years for a street light averaging 12 hours of burn time per night. At this time, LED lighting has not been in practical application for this long. Manufacturers base the rated life on projections from laboratory testing. Due to longer life, LED lighting has the potential for significant maintenance savings.

B. Light System Depreciation

Lighting system depreciation is the loss or degradation of the light output of a luminaire over time with the same power input. The primary factors of lighting system depreciation are lamp lumen depreciation (LLD) and luminaire dirt depreciation (LDD). The light source types considered in this chapter suffer degradation of their light output over their lifetime. A typical range for LLD is from 0.9 to 0.78.

All luminaire assemblies are susceptible to dirt ingress, which absorbs/blocks/disperses light produced by the lamp and prevents it from reaching the intended destination. Some judgment is required to evaluate the luminaire enclosure for contamination protection and the environment to which the luminaire will be exposed. A typical range for LDD is 0.95 to 0.78.

The product of these two factors is referred to as the Maintenance Factor (MF). This factor multiplied by the initial light source lumen output gives the maintained lumen output value, which is the expected performance of the lighting system near the end of its rated life. The maintained lumens value is what is used in lighting design photometric calculations. Typical maintenance factors used are:

High Pressure Sodium:	0.75 to 0.80
Metal Halide:	0.70 to 0.78
LED:	0.75 to 0.80

Figures 11B-1.01 and 11B-1.02 depict lamp lumen and dirt depreciation curves.

Figure 11B-1.01: Typical Lamp Lumen Depreciation

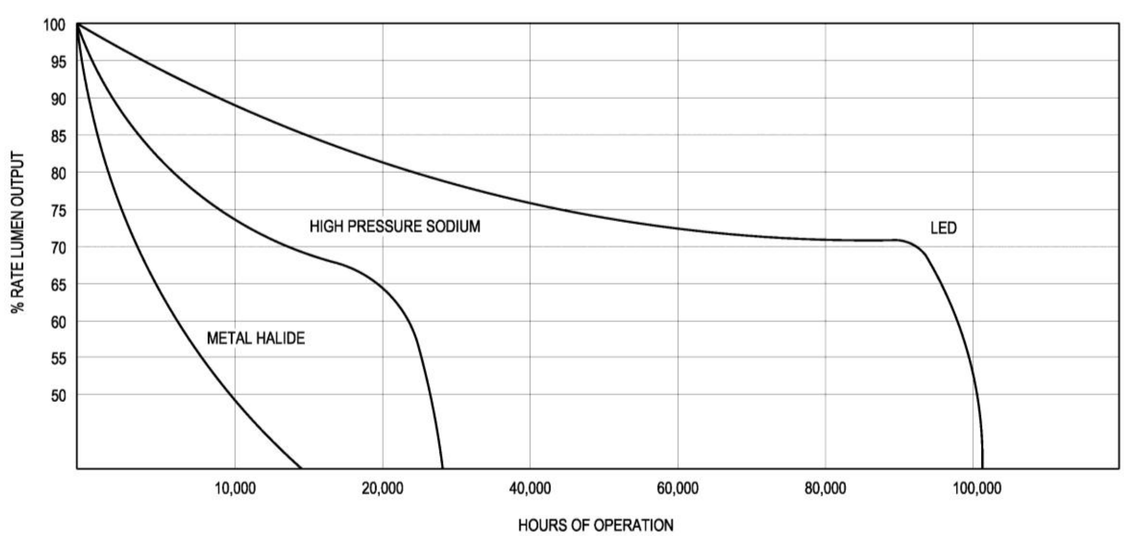
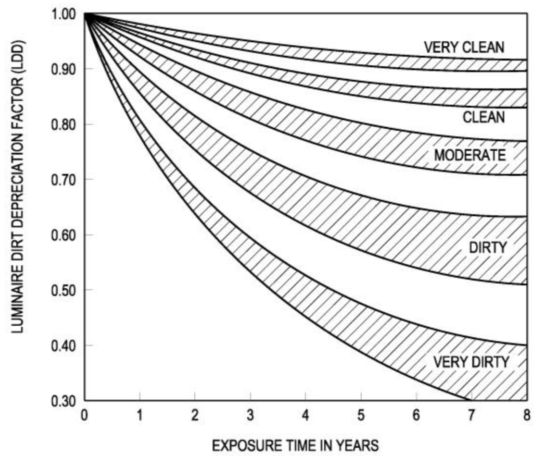


Figure 11B-1.02: Typical Luminaire Dirt Depreciation

Select the appropriate curve according to the type of ambient conditions as described by the following examples:

Very Clean - No nearby smoke or dust generating activities and a low ambient contaminant level. Light traffic. Generally limited to residential or rural areas. The ambient particulate level is no more than 150 micrograms per cubic meter.



Clean - No nearby smoke or dust generating activities. Moderate to heavy traffic. The ambient particulate level is no more than 300 micrograms per cubic meter.

Moderate - Moderate smoke or dust generating activities nearby. The ambient particulate level is no more than 600 micrograms per cubic meter.

Dirty - Smoke or dust plumes generated by nearby activities may occasionally envelope the luminaires.

Very Dirty - As above but the luminaires are commonly enveloped by smoke or dust plumes.

Source: Adapted from *Roadway Lighting Handbook*

C. Luminaire Light Distribution Classifications

The Illuminating Engineering Society of North America (IESNA) has developed classification categories and parameters to describe the photometric properties of luminaires. The classifications assist lighting designers in choosing the proper luminaires to accomplish the street lighting task. The categories are lateral light distribution, vertical light distribution, and cutoff rating.

1. **Lateral Light Distribution:** The lateral light distribution classification describes where the light from a luminaire falls into the street surface in relation to the street width, or in other words, how far the light reaches or lands across the street. The classification rating depends on the lateral distance, measured in multiples of luminaire mounting height (mh), where the half-maximum candela trace lands in relation to the location of the luminaire. Refer to Figure 11B-1.03. Following are the IES lateral distribution types and their definitions:

   Type I: Half-maximum candela trace falls between 1 mh on the house side and 1mh on the street side of the luminaire position.

   Type II: Trace falls between 1 mh and 1.75 mh on the street side of the luminaire position.

   Type III: Trace falls between 1.75 mh and 2.75 mh on the street side of the luminaire position.

   Type IV: Trace falls beyond 2.75 mh on the street side of the luminaire position.

   Type V: Has distribution that is circularly symmetrical around the luminaire position.

   The most popular types used for public streets and roads are Types II, III, and IV. Type V distribution is more popularly used in parking or area lighting applications. Type I distribution is used when the luminaire is positioned in the center median of a narrow roadway such as a boulevard driveway.

2. **Vertical Light Distribution:** The vertical light distribution describes where the maximum light intensity (maximum candela) falls longitudinally up and down the street measured in multiples of mounting height in relation to the location of the luminaire (refer to Figure 11B-1.03). Following are the IES vertical distribution types and their definitions:

   Very Short: The maximum intensity point lands 0 to 1.0 mh each way longitudinally from the luminaire position.

   Short: The maximum intensity point lands between 1.0 mh and 2.25 mh each way longitudinally from the luminaire position.

   Medium: The maximum intensity point lands between 2.25 mh and 3.75 mh each way longitudinally from the luminaire position.

   Long: The maximum intensity point lands between 3.75 mh and 6.0 mh each way longitudinally from the luminaire position.

   Very Long: The maximum intensity point lands beyond 6.0 mh each way longitudinally from the luminaire position.

   On the basis of vertical light distribution, the theoretical maximum spacing for a vertical distribution type is such that the maximum candlepower beams from adjacent luminaires are joined on the roadway surface. With this assumption, the maximum luminaire spacing for each distribution type is:

   Very Short: 2.0 mounting heights

   Short: 4.5 mounting heights

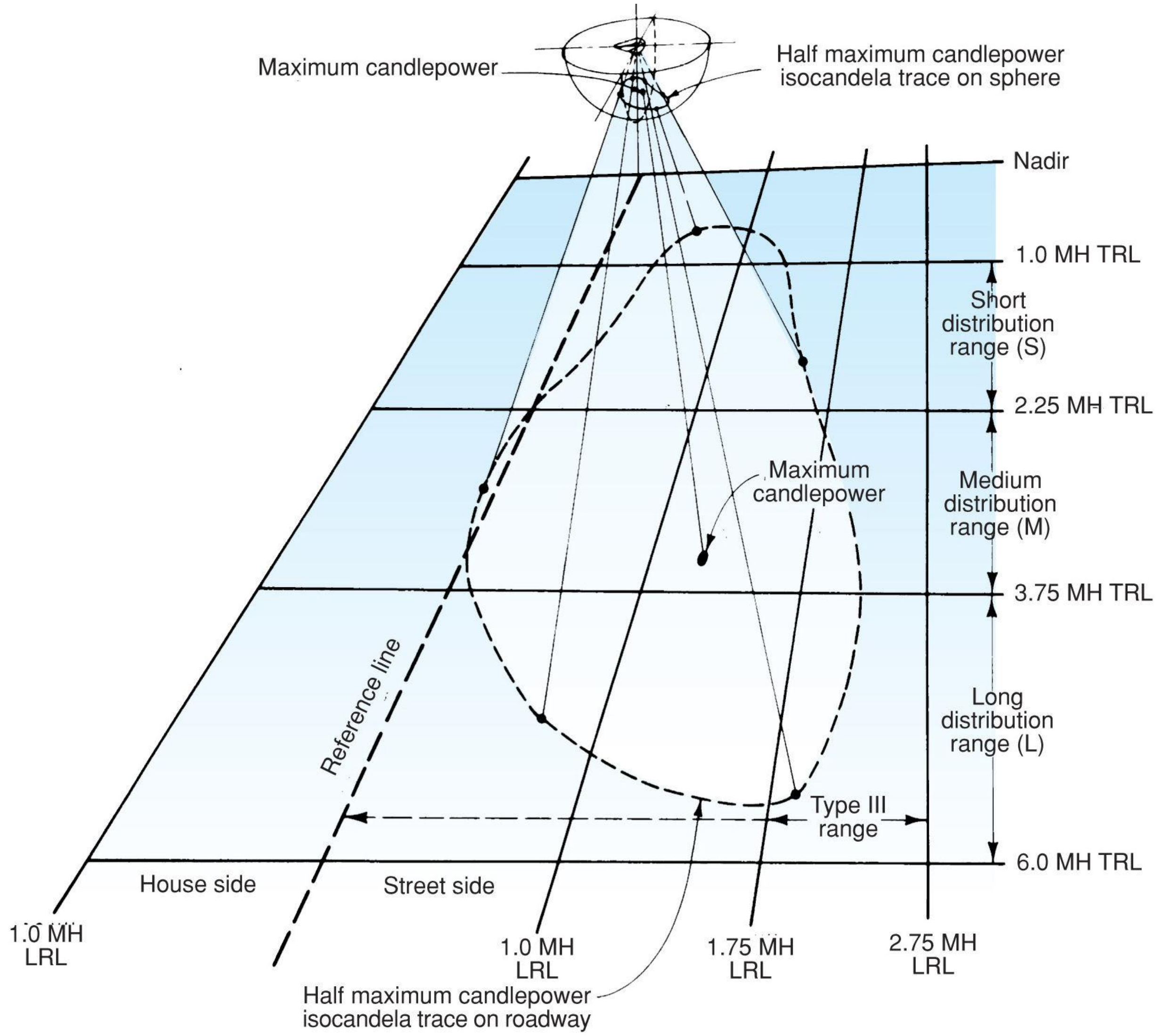
   Medium: 7.5 mounting heights

   Long: 12.0 mounting heights

   Very Long: Beyond 12.0 mounting heights

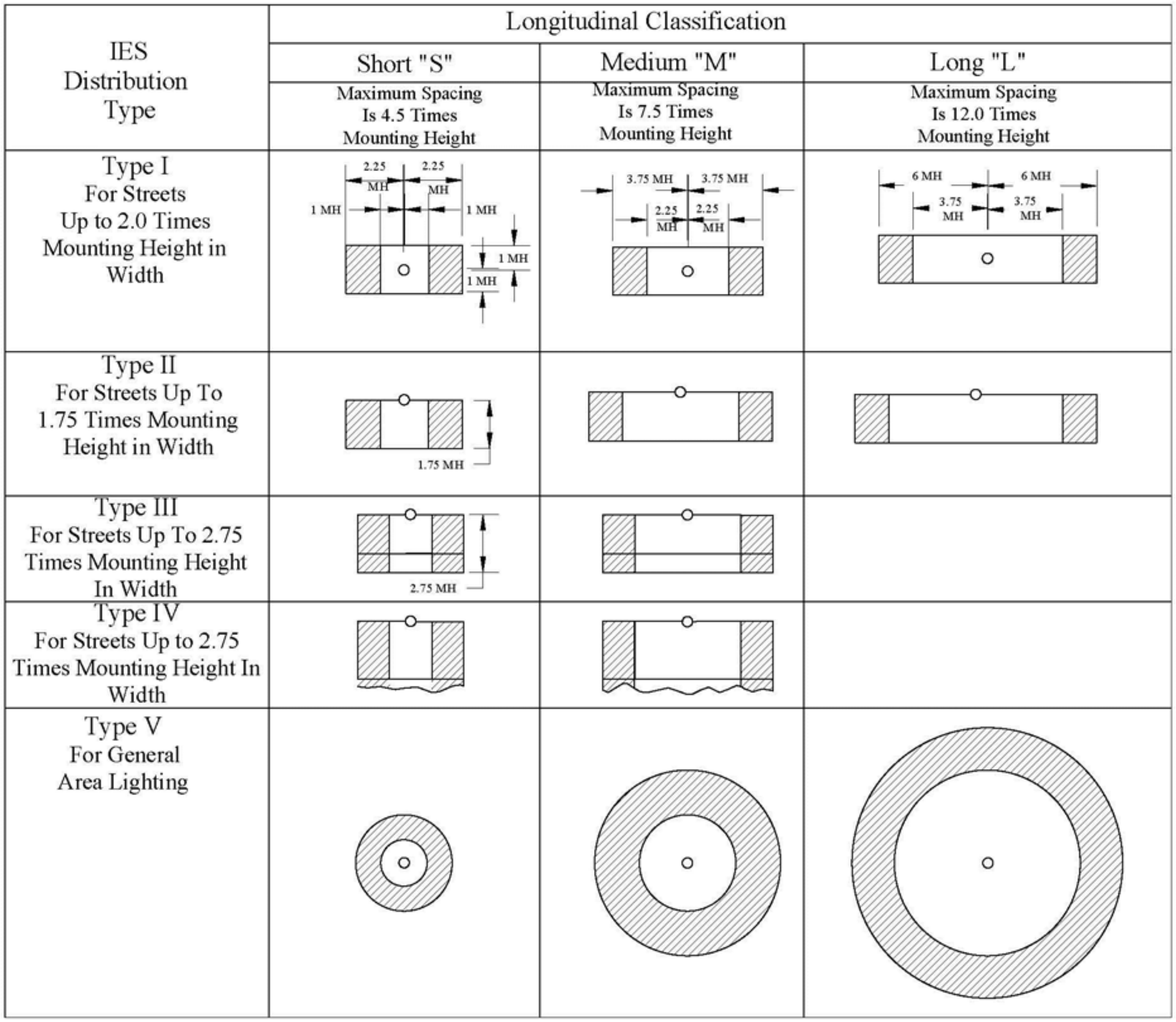
   From a practical standpoint, the medium distribution is predominantly used in practice, and the spacing of luminaires normally does not exceed five to six mounting heights. Short distributions are not used extensively for reasons of economy, because extremely short spacing and more lighting assemblies are required. At the other extreme, the long distributions are not used to any great extent because the high beam angle of maximum candlepower often produces excessive glare, as further described by the cutoff rating of a luminaire.

Figure 11B-1.03: IES Light Distribution - Illumination Zone Grid



Source: *IES Lighting Handbook*

Table 11B-1.02: IES Distribution Summary Diagrams



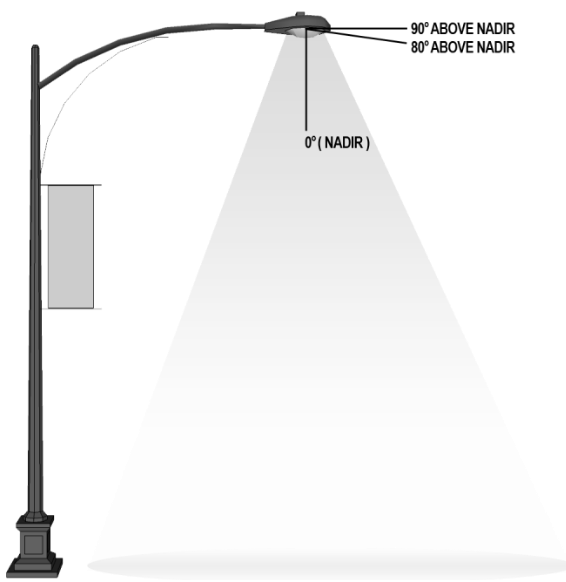
IES Distribution Type	Longitudinal Classification		
	Short "S"	Medium "M"	Long "L"
	Maximum Spacing Is 4.5 Times Mounting Height	Maximum Spacing Is 7.5 Times Mounting Height	Maximum Spacing Is 12.0 Times Mounting Height
Type I For Streets Up to 2.0 Times Mounting Height in Width	2.25 MH; 2.25 MH; 1 MH; 1 MH; 1 MH; 1 MH	3.75 MH; 3.75 MH; 2.25 MH; 2.25 MH	6 MH; 6 MH; 3.75 MH; 3.75 MH
Type II For Streets Up To 1.75 Times Mounting Height in Width	1.75 MH		
Type III For Streets Up To 2.75 Times Mounting Height In Width	2.75 MH		
Type IV For Streets Up to 2.75 Times Mounting Height In Width			
Type V For General Area Lighting			

3. **Cutoff Rating:** Disability and discomfort glare are largely a result of light emission into the driver's eye. This is largely caused by high-angle light (zone between 80 degrees to 90 degrees above nadir) emanating from a luminaire. Refer to Figure 11B-1.04. Also a concern is the amount of light emanating from the luminaire above 90 degrees from nadir (horizontal plane at the luminaire). This light contributes to sky glow. For design purposes, it is necessary that luminaires be classified according to their relative glare effects. Thus, luminaires are classified by IES as follows:

Full Cutoff: A luminaire light distribution is classified as full cutoff when the luminous intensity (candela) at or above 90 degrees from nadir is zero, and the candela per 1,000 bare lamp lumens does not exceed 100 (10%) at or above a vertical angle of 80 degrees above nadir. This applies to all lateral angles around the luminaire.

Cutoff: A luminaire light distribution is classified as cutoff when the luminous intensity (candela) per 1,000 bare lamp lumens does not exceed 25 (2.5%) at or above 90 degrees from nadir, and does not exceed 100 (10%) at or above a vertical angle of 80 degrees above nadir. This applies to all lateral angles around the luminaire.

Figure 11B-1.04: Luminaire Cutoff Diagram



Semicutoff: A luminaire light distribution is classified as semicutoff when the luminous intensity (candela) per 1,000 bare lamp lumens does not exceed 50 (5%) at or above 90 degrees from nadir, and does not exceed 200 (10%) at or above a vertical angle of 80 degrees above nadir. This applies to all lateral angles around the luminaire.

Noncutoff: A luminaire light distribution where there is no candela limitation in the zone above maximum candela.

As noted above, the metrics related to cutoff classifications for High Intensity Discharge (HID) products are based on candela (intensity) values at specific vertical angles of 80 and 90 degrees when expressed as a percentage of "rated lamp lumens". If LED fixtures are to be evaluated, a problem is developed because LED fixtures are rated in absolute format where there is no lumen rating. This difference can lead to problems if LED luminaires are compared to HID luminaires.

In order to address this difference, the IESNA has published TM-15-11 which uses the parameters of backlighting, uplighting, and glare (BUG) to determine the lumen distributions in specific areas. Design software programs are available that use the BUG rating system. Designers should consult the updated Appendix A of the TM-15-11 document if their design evaluation includes both HID and LED fixtures so the proper comparisons can be made.

D. References

Federal Highway Administration. *Roadway Lighting Handbook*. 1978.

Illuminating Engineering Society of North America (IESNA). *IES Lighting Handbook*. 9th Edition.

LED Lighting

A. LED vs. HPS Lighting

The predominant light source type for city street lighting has been high pressure sodium (HPS) for many years. In fact, HPS type street lighting has been mandated by Iowa Code (with some exceptions) since 1989 as the first choice standard for energy efficient street lighting. Other light source types were allowed to be used if they were shown to have equal or better energy efficiency than HPS.

Prior to the fall of 2010, the efficiency comparison test was based on bare lamp efficacy defined by the Code as lamp output lumens divided by lamp input watts. This efficiency definition did not take into account ballast energy losses or the luminaire's optical efficiency. The only possible practical choices for street lighting luminaires during this time were high intensity discharge (HID) type light sources (high pressure sodium, metal halide, mercury vapor). Since these luminaire types are constructed and essentially produce and control light the same way, the bare lamp efficacy test produced an apples-to-apples comparison. Of these, HPS always had the highest lamp efficacy and by Code is the only source allowed.

Not all of the light produced by the lamp exits the luminaire. HID luminaires generally are about 70% to 85% efficient at allowing the light produced by the lamp out of the luminaire. Also, not all of the light emanating from the luminaire illuminates the desired subject. A significant quantity of the light produced lands directly below the luminaire creating a "hot spot" resulting in excess illumination and wasted light. Also depending on the optics of the luminaire, some of the light is directed above 90 degrees from nadir, which results in fugitive light and undesirably brightens the sky at night.

With the advent of LED lighting, this lighting source type has challenged HPS as the most efficient. However, LED lighting was not immediately recognized as more efficient because LED luminaire efficacy is measured differently, and because of the method of comparison defined in the Iowa Code, LED luminaires were not legal to use for city street lighting. As a result, the Iowa Code was modified to allow their use if it can be shown that they are more energy efficient than HPS fixtures; see Section 11A-1, C. This process involves the regulated street lighting utility obtaining approval from the Iowa Utilities Board. This approval process does not apply for city owned street lights.

LED luminaires are constructed and deliver light differently than HID luminaires. A typical HID street lighting luminaire consists of a single lamp light source surrounded by an optical reflector and optical lens refractor to bounce and/or bend the light from the lamp and direct it onto the street as evenly as possible. The lamp is not necessarily manufactured by the luminaire manufacturer, but the lamp sizes and wattages are standardized throughout the industry. The lamp efficacy of a 250W HPS from one manufacturer is very nearly the same as that from another manufacturer. The same is true for the efficiency of the ballasts.

LED luminaires are constructed using many individual LED lamps assembled into an array and are energized by an on-board power supply commonly called a driver. Luminaires may contain as few as ten or as many as 100 or more LEDs depending on the intended function of the luminaire. The LEDs are typically individually aimed to produce the desired overall illumination pattern. Therefore, the optical control of the light from an LED luminaire is much more precise. For a street lighting application, more lamps are aimed longitudinally up and down the street and less directly below the luminaire. This results in much greater illumination uniformity on the street with less light production as compared to HPS luminaires. It significantly reduces the wasteful “hot spot” directly below the luminaire. The optical efficiency of LED luminaires is about 90% to 95% compared to the 70% to 85% stated above for HID luminaires.

Since the LEDs and the luminaire are an integral assembly, the concept of bare lamp lumens and lamp lumen efficacy is much less meaningful for LED lights. Instead, the lighting industry has chosen absolute lumens as the accepted measure of light output. The parameter of absolute lumens is defined as the measure of the total luminous flux emanating from a luminaire assembly (using any light source, not just LED). This measurement therefore takes into account both the light source and the luminaire assembly efficiency and gives you the total useable light output. It does not describe which way the light is going or whether the light intensity is concentrated in a particular direction or evenly distributed. Dividing the absolute lumens value by the luminaire assembly total input watts gives the luminaire efficacy rating. This is a much more accurate description of the overall efficiency of a luminaire and its ability to convert electrical power into useable light, and takes into account all of the parasitic losses inherent in a luminaire assembly (lamp power-to-light conversion, ballast or driver efficiency, and luminaire optical efficiency).

The ultimate comparison between luminaires is found in their application to a given task and the ability to produce the target illumination (footcandles) and uniformity using the least amount of energy for the application. Currently HPS luminaires still have higher efficacy ratings than do LED luminaires. However, since LED luminaires possess superior optical light control and produce less waste light, street lighting applications using LED lighting typically consume less energy compared to using HPS lighting.

The parameter more popularly being used to compare overall application is watts per average delivered footcandles of illumination. For a given project area, divide the total power draw in watts for the project area by the calculated average footcandles of illumination. This is a better method of comparing the energy efficiency of different lighting systems based on the delivered illumination on a surface rather than just the production of light.

B. LED Lighting Advantages

Besides reduction in energy consumption, there are other advantages to using LED lighting over HPS.

1. Compared to HPS luminaires with the same photometric classification, the application of LED luminaires to achieve a given set of illumination criteria may result in one or more of the following:
    - Better uniformity ratio at the target average illumination level
    - Lower mounting heights resulting in less costly lighting structures
    - Greater spacing between luminaires resulting fewer lighting structures and luminaires

2. The light produced by LEDs is whiter and provides significantly better color rendering of objects. There are studies demonstrating that whiter light improves the visual ability of the human eye. There are discussions among the lighting professionals that lower illumination levels may be acceptable and provide equal visibility using the whiter light of LEDs (or metal halide) as compared to the more yellow light of HPS.

3. The rated service life of LED lighting is projected to be from 50,000 to as much as 100,000 burn-time hours, which is considerably longer than HPS or other lighting types. HPS lighting sources typically have a rated service life of 24,000 burn-time hours. This would significantly reduce street light maintenance costs.

4. The components of LED solid state lighting are recyclable and contain less toxic heavy metal elements.

5. LED luminaires are dimmable. For example, this would allow a street lighting installation serving a business district to illuminate the street at a higher illumination level during evening business hours and dim to a lower allowed minimum illumination level after business hours, which would conserve energy.

6. LED luminaires are instant-on. This feature lends them to the use of occupancy or motion sensor controls to save energy. While this may not be practical for street lighting applications, it could have potential use in parking lot applications.

C. LED Lighting Disadvantages

1. Currently LED luminaires cost more than HPS luminaires.

2. LEDs themselves do not tolerate heat and need to be kept cool during operation. However, luminaire assemblies with good thermal management design can sufficiently control diode junction temperature making LED lighting a practical choice. This issue is partially mitigated by the lower ambient temperature conditions of nighttime operation.

3. Since LED luminaire wattages are not standardized, they do not readily fit into electric utility tariff rate programs and have not been incorporated into utility-owned street lighting stock.

Facility Design

A. General

The basic goal of street lighting is to provide patterns and levels of pavement luminance to provide a safer night driving environment and reduce conflict between motorists and pedestrians. A driver's eye discerns an object on or near the street due to contrast between the brightness of the object and the brightness of the background or pavement, or by means of surface detail, glint, shadows, or detection of motion.

Lighting design is concerned with the selection and location of lighting equipment so as to provide improved visibility and increased safety while making the most efficient use of energy with minimum expenditure for the lighting equipment. There are two basic concepts of lighting design - the illumination concept and the luminance concept.

The illumination concept, which is almost universally used in the United States, is based on the premise that by providing a given level of illumination and uniformity of distribution, satisfactory visibility can be achieved. The luminance concept is based on the premise that visibility is related to the luminance of the pavement compared to the luminance of the objects on the pavement. Calculations to determine the luminance of pavement or objects require the estimation of the reflectivity of varying pavement surfaces and objects within the driver’s field of vision. These reflectivity values can be difficult to estimate and can vary widely.

The luminance concept is fairly popular in parts of Europe and is being promoted by lighting professionals in the United States. At this time, ANSI/IESNA RP-8-00, R2005 (RP-8) supports both lighting design concepts. However, it is believed the next revision of RP-8 will favor the luminance concept. Although other design concepts are discussed in RP-8, such as Small Target Visibility, the illuminance concept design method remains predominant in the United States. Therefore, the illuminance method will be the only design concept discussed in this chapter.

B. Design Process

By definition, lighting design according to the illumination method relies on the “illumination” or amount of light flux reaching the pavement from the lighting source (quantity) and the uniformity of that illumination on the pavement surface (quality). The steps in the design process are as follows:

- Determination of the design illumination and uniformity criteria by assessing the facility to be lighted.
- Selecting the type of light source.
- Selecting light source size and mounting height.
- Selecting luminaire light distribution type.
- Determining luminaire spacing and location.
- Checking for design adequacy.

These steps are arranged in the order in which they are usually encountered in the design process.

1. **Design Criteria:** The first task of the lighting designer is to research and determine if any requirements (such as ordinances, resolutions, or policies) pertaining to street lighting are in effect in the jurisdiction. Many municipalities have no requirements at all. Some may have adopted a published standard in its entirety or have adopted it with some variations. Others may have developed prescriptive guidelines that, for a given street type, specifically describe the luminaire size and type, specific mounting height, and pole spacing. Still others may have developed a combination of these depending on the street type. Finally, a municipality may have requirements that do not deal directly with the amount of light on the street. Rather, they may simply be lighting limitations such as maximum footcandle levels at property or right-of-way lines to control light trespass, or allow only cutoff type luminaires to control sky glow or excessive glare.

   The designer's first obligation is to conform to state codes and jurisdictional requirements, but in the absence of such requirements, it is recommended that the designer follow a nationally recognized written street lighting design standard such as RP-8.

   To perform street lighting design, two parameters need to be considered - illumination level and uniformity. The amount of illumination at any given point on a street surface is expressed in footcandles (fc). Since the luminous flux from street lighting is typically not distributed evenly over the pavement surface, the illumination is expressed in average footcandles when describing the level of illumination over a defined area. This parameter describes the "quantity" of light provided.

   While the average amount of illumination on the street surface may be satisfactory, the lighting distribution may consist of very high (bright) and very low (dim) localized illumination areas. A driver traveling down a street illuminated in this manner will experience difficulty seeing the street and other objects due to the inability of the eye to rapidly adjust to the varying light conditions. Therefore, another parameter is needed to describe the evenness or uniformity of the applied lighting. This parameter is known as the uniformity ratio of the illumination distribution and is defined as either the ratio of maximum-to-minimum footcandle values or the ratio of the average-to-minimum footcandle values over the project area. The most popular choice is the average-to-minimum ratio. This parameter describes the "quality" of the illumination distribution. A ratio of 1:1 represents perfectly uniform illumination distribution. A real-life example of this is moonlight at night from a full moon overhead. The illumination level of moonlight is approximately 0.5 fc but it is almost perfectly uniform.

   The Illuminating Engineering Society of North America has established acceptable illumination levels and uniformity ratios for various public street types. See Table 11C-1.01. To obtain the recommended average illumination and uniformity ratio for a given street, there are three classifications that need to be determined - the street use, the pavement type, and the level of pedestrian conflict associated with the street.

Table 11C-1.01: Illuminance Method - Recommended Values

Street and Pedestrian Conflict Area		Pavement Classification (Minimum Maintained Average Values)			Uniformity Ratio E_{ave}/E_{min}	Veiling Luminance Ratio L_{max}/L_{avg}
Street	***Pedestrian Conflict Area***	***R1 fc***	***R2 and R3 fc***	***R4 fc***		
Freeway Class A	N/A	0.6	0.9	0.8	3.0	0.3
Freeway Class B	N/A	0.4	0.6	0.5	3.0	0.3
Expressway	High	1.0	1.4	1.3	3.0	0.3
	Medium	0.8	1.2	1.0	3.0	0.3
	Low	0.6	0.9	0.8	3.0	0.3
Major (Arterial)	High	1.2	1.7	1.5	3.0	0.3
	Medium	0.9	1.3	1.1	3.0	0.3
	Low	0.6	0.9	0.8	3.0	0.3
Collector	High	0.8	1.2	1.0	4.0	0.4
	Medium	0.6	0.9	0.8	4.0	0.4
	Low	0.4	0.6	0.5	4.0	0.4
Local	High	0.6	0.9	0.8	6.0	0.4
	Medium	0.5	0.7	0.6	6.0	0.4
	Low	0.3	0.4	0.4	6.0	0.4

Pedestrian Conflict Area Classifications:

- High - Areas with significant numbers of pedestrians expected to be on the sidewalks or crossing the streets during darkness. Examples are down-town retail areas, near theaters, concert halls, stadiums, and transit terminals.
- Medium - Areas where lesser numbers of pedestrians utilize the streets at night. Typical are down-town office areas, blocks with libraries, apartments, neighborhood shopping, industrial, older city areas, and streets with transit lines.
- Low - Areas with very low volumes of night pedestrian usage. These can occur in any of the cited street classifications but may be typified by sub-urban single family streets, very low density residential developments, and rural or semi-rural areas.

Source: Adapted from *ANSI / IES RP-8-00* (R2005)

Table 11C-1.02: Street Surface Classifications

Class	Q_o*	Description	Mode of Reflectance
R1	0.10	PCC street surface. Asphalt street surface with a minimum of 12% of the aggregates composed of artificial brightener (e.g., Synopal) aggregates (e.g., labradorite, quartzite).	Mostly diffuse
R2	0.07	Asphalt street surface with an aggregate composed of minimum 60 percent gravel [size greater than 1 cm (0.4 in.)]. Asphalt street surface with 10% to 15% artificial brightener in aggregate mix. (Not normally used in North America).	Mixed (diffuse and specular)
R3	0.07	Asphalt street surface (regular and carpet seal) with dark aggregates (e.g., trap rock, blast furnace slag); rough texture after some months of use (typical highways).	Slightly specular
R4	0.08	Asphalt street surface with very smooth texture.	Mostly specular

* Q_o = representative mean luminance coefficient

Source: *ANSI / IES RP-8-00* (R2005)

a. **Street Use:** While the street types in Table 11C-1.01 vary from high speed freeways down to low speed local streets, this chapter is only concerned with the major (also known as arterial), collector, and local street classifications. Some jurisdictions have already classified their streets and it is recommended to follow these classifications first. If the jurisdiction has not established classifications, refer to the descriptions in Chapter 5 - Roadway Design to determine the classification of the subject street.

b. **Pavement Type:** Pavement types are classified into four categories, R1 through R4. For the purposes of determining lighting criteria, two of the pavement classifications, R2 and R3, are combined, forming three illumination classifications. Refer to Table 11C-1.02 to determine the pavement type classification of the subject street.

c. **Pedestrian Conflict:** Pedestrian conflict is categorized into three classifications - high, medium, and low. The level of pedestrian conflict is almost entirely driven by the land use adjoining the street and the potential of the land use to cause pedestrian traffic during nighttime hours. For example, pedestrian conflict would be low for a local residential street as compared to a high pedestrian conflict level for a local street next to a movie theater. Refer to the pedestrian conflict classification descriptions following Table 11C-1.01 to determine the potential pedestrian conflict for the subject street.

   Using the defined classifications, determine the recommended illumination and uniformity ratio for the subject street. The illumination values listed represent average maintained footcandles over the street surface. The uniformity ratio is average footcandles divided by the minimum footcandle value. These values represent the minimum illumination and the maximum uniformity ratio recommended. The designer may consider more illumination and/or better uniformity for the street if it would better serve expected activity along the route.

2. **Selecting the Type of Light Source:** The vast majority of street lighting in municipalities is owned, operated, and maintained by the local electric utility. The cost for the installation, energy, and maintenance is paid for by the municipality in monthly installments based on established utility tariff rates for the type of lighting units installed. These rates are regulated and set by the utility with approval from the Iowa Utilities Board. If the street lighting to be installed on a particular project will be utility owned, the lighting equipment will need to be selected from that available from the utility. While the utility maintains a stock of various lighting source types, the only allowable type for public street lighting is HPS unless certain other conditions or exceptions can be met per Iowa Code.

   Currently, electric utilities do not maintain a stock of LED lighting luminaires for two reasons: the LED lighting package sizes have not been standardized, and LED technology is in a rapid state of flux. The energy consumption of any given LED package size may or may not fit in the utility’s current tariff rate structure. Because they are regulated, utilities are not at liberty to create custom tariff rates to fit random load sizes. Also because of the rapid change in the industry, LED luminaire costs are varying widely and are considerably more expensive than HPS luminaires. This will likely change in the future when LED technology plateaus, cost compared to performance stabilizes, and the industry introduces more standardization. For now, LED lighting must be owned by the customer and must be on metered electric services.

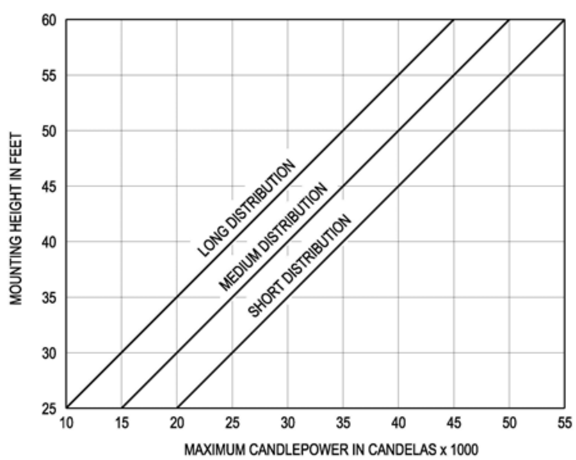
   If the street lighting will be owned and operated by the municipality, the choice of light source type is a little more open, but again, the installation still needs to meet Iowa Code. HPS lighting is a stable technology that can be installed economically with little risk from unknowns. LED lighting on the other hand, has more unknowns, such as will LEDs last as long as predicted and what will be the true cost of maintenance in the future. In spite of this, LEDs are seeing more use in applications, and with proper layout design, are proving to provide better quality lighting for less energy consumption compared to HPS.

If initial cost is an important parameter, currently HPS will have lower installation cost. If life-cycle cost is the deciding factor, then LED will likely win out, but the designer will have to develop layouts for each type to make the comparison. If the color of the light and color rendering of objects are important, LED will be the choice. In the future with the increase in performance of LEDs, the confirmation of LED rated life and the initial cost of LED luminaires nearing the cost of traditional HPS luminaires, LED will become the primary lighting source.

3. **Selecting Light Source Size and Mounting Height:** The distance the lamp/luminaire is mounted above the street will affect the illumination intensity, uniformity of brightness, area covered, and relative glare of the unit. Higher mounted units will provide greater coverage, more uniformity, and reduction of glare, but a lower illumination level. The illumination of an object from a light source varies inversely to the square of the distance from the light source, so doubling the distance will reduce the illumination on the object to one fourth of the original value. Therefore, greater mounting heights will require larger wattage luminaires. It is necessary to weigh the effects of larger wattage luminaires against a greater number of smaller units at lower mounting heights with an increase in glare potential.

   Mounting heights of street luminaires vary from 15 feet to more than 100 feet above the street surface. Conventional municipal street lighting utilizes mounting heights of 25 to 50 feet. Generally, the greater the target uniformity ratio, the shorter the mounting height and vice versa. Local street lighting uses 25 to 30 feet mounting heights while collector and major streets will use 30 to 40 feet mounting heights. The lower mounting heights may require the use of luminaires with a semi-cutoff distribution or better to minimize glare. Figure 11C-1.01 shows minimum mounting heights for various maximum candela levels and vertical light distributions.

Figure 11C-1.01: Minimum Mounting Height vs. Maximum Candela



Source: Adapted from *Roadway Lighting Handbook*

4. **Selecting Luminaire Light Distribution Type:** Selection of the luminaire light distribution type (lateral, vertical, and cutoff) for a given street lighting application depends on several elements, the mounting height, the pole placement pattern, the cross sectional geometry of the street, and any jurisdictional ordinances that control or limit light trespass, glare, or sky glow.

   Table 11B-1.02 is a guide to selecting which lateral light distribution(s) are best suited for the street width and pole placement pattern. This is only a guide. While lighting distribution types are defined, luminaires that fit into a type still vary between manufacturers. A Type II from one manufacturer may provide better illumination than a Type III from another for a wider street. For the given street width, pole pattern, and mounting height, the distribution pattern from the Type II may “fit” together better and provide more uniform light.

   The designer may select the first luminaire that meets the illumination criteria. However this may not be optimum selection based on defined goals of the project. Street lighting design is an iterative process if optimization is to be achieved.

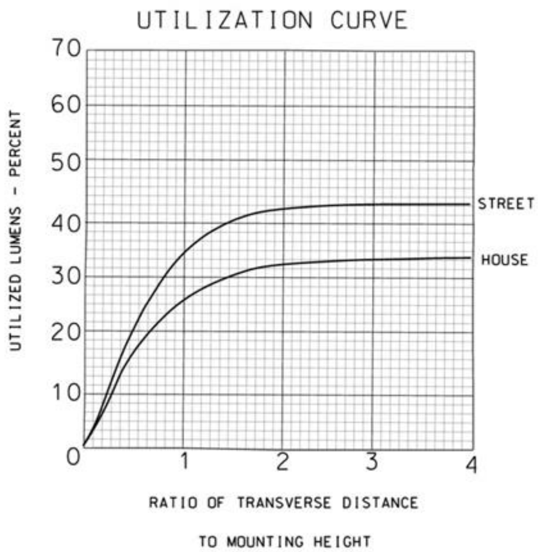
5. **Determining Luminaire Spacing and Location:** The most common lateral location of street lighting luminaires is positioned over the curb line or edge of pavement (zero overhang). This is also the base line for luminaire design. Since it would be impractical to place light poles directly at the edge of the street, lighting support structures typically consist of a poles fitted with mast arms to set the poles back away from and provide clearance for traffic and pedestrians. Streets typically have defined clear zones behind the curb or pavement edge, the width of which depends on the street characteristics. The designer needs to consider setback to determine if a mastarm of sufficient length is available to place the luminaire at the street edge. Luminaires positioned with excessive negative overhang will likely require shorter longitudinal luminaire spacing to compensate.

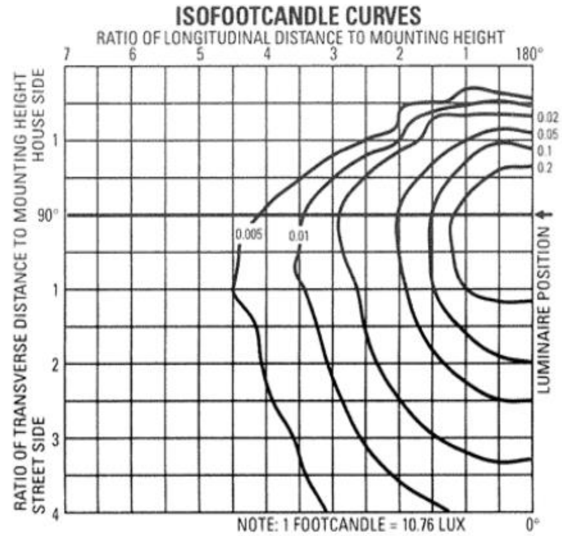
   Section 11B-1 discusses theoretical maximum longitudinal luminaire/pole spacing for a given vertical light distribution. However, this spacing may not be practical to fit the site. The designer needs to consider how the street interfaces with the adjoining property features. These factors include location of sidewalks, bike trails, driveways, alleys, and cross streets. Many times, particularly in residential areas, it is desirable to place the light poles in line with the side property lines.

6. **Checking for Design Adequacy:** All of the above selected elements are formed into a design concept or model. The next step is to perform calculations to verify the chosen equipment and layout to meet the design criteria. For many years, manual calculations were the only methods used to determine the resulting design illumination and uniformity. Since the advent of the computer, numerous software programs have been developed and are available to automate the calculation process.

   a. **Manual Calculation Method:** The most popular manual calculation method is the coefficient of utilization and isofootcandle plot method. As the name implies, two pieces of graphical information are required, a coefficient of utilization curve and an isofootcandle plot. These are developed by luminaire manufacturers and are required for the calculation process. Examples of such are shown in Figure 11C-1.02. The coefficient curve is a quantitative description of the percentage of total lumens emitted from the fixture that will land on or be utilized to illuminate the street below based on the street width and relative position of the luminaire to the street.

Figure 11C-1.02: Typical Luminaire Utilization and Isofootcandle Plots





Rather than repeat the process here, the designer is recommended to visit and access Minnesota DOT Street Lighting Design Manual, Sections 4 and 5. The discussion in this document provides a good step-by-step description of the manual calculation process.

b. Computer Modeling Method: All that is required is to obtain a lighting application software program to run on the computer to have the tools to model lighting installations and perform photometric calculations. There are numerous programs available, both purchased and free. Some software packages can be very sophisticated with the ability to create such things as shade plots and shade and shadow renderings to closely represent what the human eye would see. For the design purposes described herein, all that is required of the software is to take luminaire photometric data and perform point-by-point calculations on a defined plane and be able to export the numerical results.

The first requirement is to create a computer model of the street to be lit. For most situations, this involves defining the width and length of the street. Most of the lighting programs have drawing tools to create the model directly in the program. If an electronic representation of the street is available from a computer-aided design file such as that created by AutoCAD or Microstation, this can be imported into the lighting program to form the model. Once this is done, the designer will “place” luminaires spatially above the model surface locating them with the desired mounting height and overhang from the street edge.

For each luminaire type to be considered, the designer needs to acquire a photometric file that describes the photometry or lighting distribution characteristics of the luminaire. These files are generated by the manufacturer through laboratory testing. They are text files containing a defined array of light intensity values (candela) in standardly defined spatial directions emanating from the luminaire. The files are commonly referred to as IES photometric files (or IES files) since the standard was developed by the Illuminating Engineering Society of North America (IESNA). The files are readily available from the manufacturer’s website at no cost.

The files are imported into the program to model the performance of the selected luminaires. The candela values in the file are typically based on a default lamp lumen value of 1,000 lumens. The designer will be required to input the proper initial lamp lumen value, which will scale the intensity values accordingly. For LED luminaires however, the file usually contains the actual initial lumen value of the luminaire assembly since the LEDs are not necessarily a removable modular element of the luminaire. In any case, the designer is cautioned to verify the proper lumen value is used. Also, the designer will need to enter the lumen maintenance factor for each luminaire model.

The final task is to define a calculation area by drawing a region on the street model surface. The width of the area could be back of curb to back of curb for example, or it could be right-of-way to right-of-way to calculate the illumination from building face to building face in a downtown business district. Within this area, the designer will create a calculation grid that is a defined set of points on the surface, at which the footcandle illumination level will be calculated. Typical calculation point grids are a 10 feet by 10 feet or a 5 feet by 5 feet rectangular array. More points in the calculating area will usually yield more accurate results but require more computer processing time. For a small area, this is not a problem, but if the designer has created a large area, the time may be significant.

The program utilizes the superposition principal to perform the calculation. The program will step through each point and calculate the illumination contribution at that point on the model surface from each luminaire defined in the model. Each of these contribution values are simply added together to get the overall illumination at that point. Once all of the points are calculated, the program determines the average illumination value of all of the points in the grid, giving the average illumination of the entire surface. The program then uses the point with the lowest footcandle value to calculate the average-to-minimum uniformity ratio.

A clear advantage of using computer modeling is the ease in which the designer can make changes to the luminaire layout model and obtain the illumination results for different scenarios. For example, the designer could change luminaire, type, wattage, mounting height, or position; or any combination of these to optimize the lighting design and minimize the energy consumption.

Most available lighting design software packages contain pre-defined street models or “wizards” for quick luminaire spacing optimization. This allows a designer to simply input a luminaire at a mounting height, a street width, a mounting pattern (one-side, each side staggered, etc.), and target illumination criteria, and have the program calculate the optimum longitudinal luminaire spacing.

C. References

Federal Highway Administration. *Roadway Lighting Handbook*. 1978.

Illuminating Engineering Society of North America. *American National Standard Practice for Roadway Lighting*. ANSI / IENSA RP-8-00, (R2005).

Minnesota Department of Transportation. *Roadway Lighting Design Manual.* 2003.

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CHAPTER 12
Sidewalks and Bicycle Facilities

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Chapter 12 - Sidewalks and Bicycle Facilities

12A Sidewalks

12B Bicycle Facilities

General Sidewalk Requirements

A. Introduction

Sidewalks are an integral component of the transportation system. They provide a designated area, separated from the roadway, for pedestrians to use for both travel and recreation. Along roadways where pedestrians are present or anticipated, consideration should be given to constructing sidewalks on both sides of the road to minimize conflicts between vehicles and pedestrians.

Where sidewalks are provided, they must be constructed so they are accessible to all potential users, including those with disabilities. Design standards for pedestrian access routes are provided in Section 12A-2.

B. Sidewalk Classes

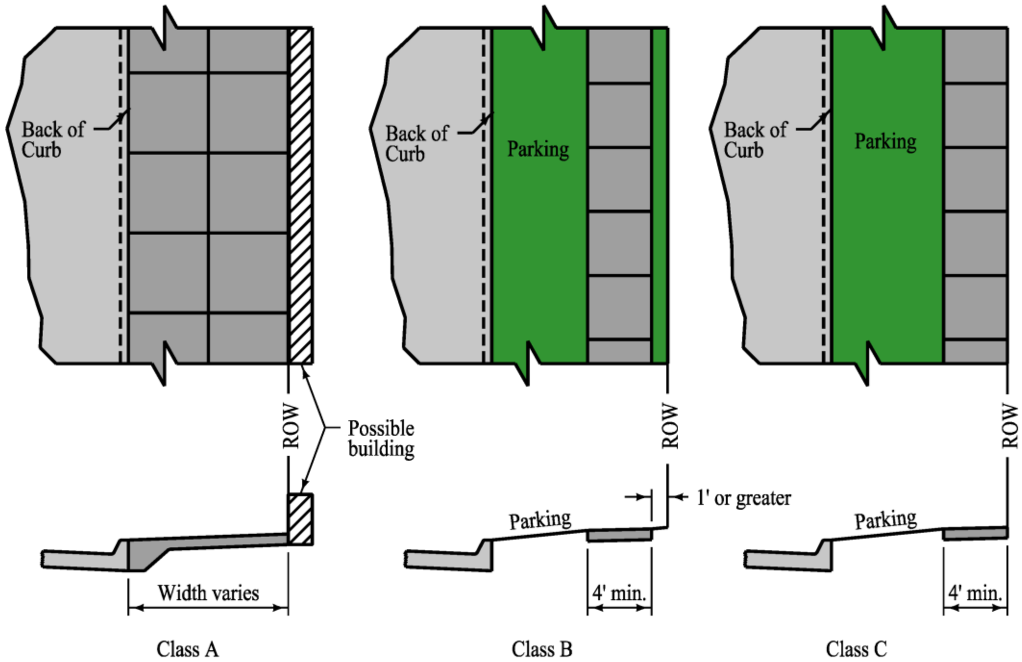
SUDAS identifies three classes of sidewalks, which are described below. Class B and C sidewalks provide a grass strip between the back of curb and the sidewalk, often referred to as the "parking."

1. **Class A:** Class A sidewalks begin at the back of curb and generally extend to the right-of-way line. These types of sidewalks are typical in downtown areas. Consideration must be given to the location of street signs, street lighting, utilities, mailboxes, snow storage, and other obstacles when utilizing Class A sidewalk.

2. **Class B:** Class B sidewalks are constructed with the back edge of the sidewalk 1 foot or more off of the right-of-way line.

3. **Class C:** Class C sidewalks have the back edge of the sidewalk on the right-of-way line.

Figure 12A-1.01: Classes of Sidewalk



C. Accessible Sidewalk Design

It has been common practice to place the responsibility for sidewalk ramp layout on the contractor or construction inspector. This has resulted in the sidewalk, curb ramps, driveway crossings, etc. being designed in the field, often with mixed accessibility results. As public right-of-way accessibility comes under greater scrutiny, it is increasingly important that newly constructed or altered sidewalks meet accessibility requirements. Therefore, sidewalks, curb ramps, and street crossings shall be included as part of the design process and the details of those designs shall be included in the contract documents as appropriate. Projects reviewed or let by the Iowa DOT will require use of S sheets according to the Iowa DOT Design Manual Section 1F-18.

D. Construction Requirements

1. **Sidewalk Thickness:** Sidewalks should be constructed of PCC with a minimum thickness of 4 inches. Where sidewalks cross driveways, the minimum thickness is 6 inches, or the thickness of the driveway, whichever is greater.

2. **Obstructions:** All obstructions are to be removed or relocated except for those that are impractical to move. In new development areas, these items should never occur, but in older, established areas, they will have to be addressed. In the case where the sidewalk is shifted to avoid an obstacle, use of a minimum 2:1 taper to and from the obstruction with a straight section adjacent to the obstruction should be considered. Flatter tapers may be used if space is available and user volume is high.

3. **Construction Tolerances:** Dimensions are subject to conventional industry tolerances except where dimensions are stated as a range, minimum, or maximum. Conventional industry tolerances include tolerances for field conditions and tolerances that may be a necessary consequence of a particular manufacturing process. Conventional industry tolerances do not apply to design work; see PROWAG R103.1. Designing features to the target values, rather than the allowable maximum or minimum, allows for appropriate construction tolerances and field adjustment during construction while maintaining compliance with PROWAG.

Accessible Sidewalk Requirements

A. Introduction

SUDAS and Iowa DOT jointly developed this section based on the July 26, 2011 "Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way." This section was developed in accordance with Federal regulations (23 CFR 652 and 28 CFR 35) and is the standard for use by all governmental entities in the State of Iowa. A local jurisdiction may elect to produce their own standards; however, these will require review and approval by FHWA and/or the United States Department of Justice.

Where sidewalks are provided, they must be constructed so they are accessible to all potential users, including those with disabilities. This section establishes the criteria necessary to make an element physically accessible to people with disabilities. This section also identifies what features need to be accessible and then provides the specific measurements, dimensions, and other technical information needed to make the feature accessible. The requirements of this section were developed based on the following documents:

1. **ADAAG:** The "Americans with Disabilities Act Accessibilities Guidelines" (ADAAG) was written by the US Access Board and adopted by the Department of Justice (DOJ) in 2010. This document includes a broad range of accessibility guidelines including businesses, restaurants, public facilities, public transportation, and sidewalks. These standards were originally adopted in 1991 and have been expanded and revised several times.

2. **PROWAG:** The July 26, 2011 "Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way" was written by the US Access Board and is also known as the Public Right-of-Way Accessibility Guidelines or PROWAG. PROWAG provides more specific information than the ADAAG for transportation facilities within the right-of-way including pedestrian access routes, signals, and parking facilities. The PROWAG requirements are currently in the development and adoption process and have not been officially adopted by the Department of Justice; however, the Federal Highway Administration has issued guidance that the draft version of the PROWAG "are currently recommended best practices, and can be considered the state of the practice that could be followed for areas not fully addressed" in the existing ADAAG requirements.

   Due to the widespread acceptance of the PROWAG, and their pending adoption in the future, the standards of this chapter are based upon the PROWAG requirements. The designer is encouraged to reference the complete PROWAG document for additional information (www.access-board.gov). References to the PROWAG in this section are shown in parentheses, e.g. (R302.7). Buildings and other structures not covered by PROWAG must comply with the applicable requirements of the ADAAG. For parks, recreational areas, and shared use paths, refer to other sections within this chapter.

B. Transition Plan

The ADA law passed in 1990 required public entities with more than 50 total employees to develop a formal transition plan identifying the steps necessary to meet ADA accessibility requirements for all pedestrian access routes within their jurisdiction by upgrading all noncompliant features. Recognizing that it would be difficult to upgrade all facilities immediately, the law provided the opportunity to develop a transition plan for the implementation of these improvements. Covered entities had until 1992 to complete a transition plan. In addition, any local public agency that is a recipient of US DOT funds must have a transition plan. For those agencies that have not completed a transition plan, it is critical that this process be completed. Although the transition plan may cover a broader scope, this section will only cover requirements within the public right-of-way.

Key elements of a transition plan include the following:

- Identifying physical obstacles in the public agency's facilities that limit the accessibility of its programs or activities to individuals with disabilities
- A detailed description of the methods that will be used to make the facilities accessible
- A schedule for taking the steps necessary to upgrade pedestrian access in each year following the transition plan
- Identification of the individual responsible for implementation of the plan

The document: *ADA Transition Plans: A Guide to Best Management Practices* (NCHRP Project No. 20-7 (232)) provides guidance for the development and update of transition plans. The document also assists communities in prioritizing required improvements for accessibility.

Public entities not required to have a formal transition plan are required to address noncompliant pedestrian access routes.

C. Definitions

Accessible: Facilities that comply with the requirements of this section.

Alteration: An alteration is a change that affects or could affect the usability of all or part of a building or facility. Alterations of streets, roadways, or highways include activities such as reconstruction, rehabilitation, resurfacing, widening, and projects of similar scale and effect.

Alternate Pedestrian Access Route: A route provided when a pedestrian circulation path is temporarily closed by construction, alterations, maintenance operations, or other conditions.

Curb Line: A line at the face of the curb that marks the transition between the curb and the gutter, street, or highway.

Cross Slope: The grade that is perpendicular to the direction of pedestrian travel.

Crosswalk: See pedestrian street crossing.

Curb Ramp: A ramp that cuts through or is built up to the curb. Curb ramps can be perpendicular, parallel, or a combination of parallel and perpendicular curb ramps.

Detectable Warning: Detectable warnings consist of small, truncated domes built in or applied to a walking surface that are detectable by cane or underfoot. On pedestrian access routes, detectable warning surfaces indicate the boundary between a pedestrian route and a vehicular route for pedestrians who are blind or have low vision.

New Construction: Construction of a roadway where an existing roadway does not currently exist.

Pedestrian Access Route: A continuous and unobstructed path of travel provided for pedestrians with disabilities within, or coinciding with, a pedestrian circulation path.

Pedestrian Circulation Path: A prepared exterior or interior surface provided for pedestrian travel in the public right-of-way.

Pedestrian Street Crossing: A marked or unmarked route, providing an accessible path to travel from one side of the street to the other. Pedestrian street crossings are a component of the pedestrian access route and/or the pedestrian circulation path.

Running Slope: The grade that is parallel to the direction of pedestrian travel.

PROWAG: The Public Right-of-way Accessibility Guidelines establish the criteria for providing a feature within the public right-of-way that is physically accessible to those with physical disabilities.

Scope of the Project: Work that can reasonably be completed within the limits of the project. This is not defined by the written project scope; however, it focuses on whether the alteration project presents an opportunity to design the altered element, space, or facility in an accessible manner.

Structurally Impracticable: Something that has little likelihood of being accomplished because of those rare circumstances when the unique characteristics of terrain prevent the incorporation of full and strict compliance with this section. Applies to new construction only.

Technically Infeasible: With respect to an alteration of an existing facility, something that has little likelihood of being accomplished because existing structural conditions would require removing or altering a load-bearing member that is an essential part of the structural frame; or because other existing physical or site constraints prohibit modification or addition of elements, spaces, or features that are in full and strict compliance with the requirements of this section. (2010 ADAAG 106.5)

Turning Space: An area at the top or bottom of a curb ramp, providing a space for pedestrians to stop, rest, or change direction.

D. Applicability

1. **New Construction:** Newly constructed facilities within the scope of the project shall be made accessible to persons with disabilities, except when a public agency can demonstrate it is structurally impracticable to provide full compliance with the requirements of this section. Structural impracticability is limited to only those rare situations when the unique characteristics of terrain make it physically impossible to construct facilities that are fully compliant. If full compliance with this section is structurally impracticable, compliance is required to the extent that it is not structurally impracticable. [2010 ADAAG 28 CFR 35.151(a)]

2. **Alterations:** Whenever alterations are made to the pedestrian circulation path, the pedestrian access route shall be made accessible to the maximum extent feasible within the scope of the project. If full compliance with this section is technically infeasible, compliance is required to the extent that it is not technically infeasible. [2010 ADAAG 28 CFR 35.151(b)] Alterations shall not gap pedestrian circulation paths in order to avoid ADA compliance.

Resurfacing is an alteration that triggers the requirement for curb ramps if it involves work on a street or roadway spanning from one intersection to another. Examples include, but are not limited to, the following treatments or their equivalents:

- New layer of surface material (asphalt or concrete, including mill and fill)
- Reconstruction
- Concrete pavement rehabilitation and reconstruction
- Open-graded surface course
- Microsurfacing and thin lift overlays
- Cape seals (slurry seal or microsurfacing over a new chip seal)
- In-place asphalt recycling

[DOJ/U.S. DOT Glossary of Terms and DOJ/U.S. DOT Technical Assistance; June 28, 2013]

Where elements are altered or added to existing facilities, but the pedestrian circulation path is not altered, the pedestrian circulation path is not required to be modified (R202.1). However, features that are added shall be made accessible to maximum extent feasible. The following are examples of added features:

- Installation of a traffic sign does not require sidewalk improvements; however, the sign cannot violate the protruding objects requirements.
- Installation of a traffic or pedestrian signal does not require sidewalk improvements; however, the signal must be accessible.
- Installation of a bench adjacent to the pedestrian access route would not require sidewalk improvements, but the bench cannot be placed in a manner that would reduce the sidewalk width below the minimum requirement.

3. **Maintenance:** Accessibility improvements are not required for work that is considered maintenance. Examples of work that would be considered maintenance include, but are not limited to, the following items.

- Painting pavement markings, excluding parking stall delineations
- Crack filling and sealing
- Surface sealing
- Chip seals
- Slurry seals
- Fog seals
- Scrub sealing
- Joint crack seals
- Joint repairs
- Dowel bar retrofit
- Spot high-friction treatments
- Diamond grinding
- Minor street patching (less than 50% of the pedestrian street crossing area)
- Curb and gutter repair or patching outside the pedestrian street crossing
- Minor sidewalk repair that does not include the turning space and curb ramps
- Filling potholes

If a project involves work not included in the list above, or is a combination of several maintenance items occurring at or near the same time, the agency administering the project is responsible for determining if the project should be considered maintenance or an alteration. If either of these two situations is determined to be maintenance, the agency administering the project must document the reasons for this determination. If the project is defined as maintenance, federal funding and Farm-to-Market funds cannot be used.

When a maintenance project modifies a crosswalk, installation of curb ramps at the crosswalks is recommended, if none already exists. The other accessibility improvements of this section are also recommended, but not required with such projects.

4. **Technical Infeasibility:** Examples of existing physical or site constraints that may make it technically infeasible to make an altered facility fully compliant include, but are not limited to, the following:
   - Right-of-way availability. Right-of-way acquisition in order to achieve full compliance is not mandatory, however, it should be considered. Improvements may be limited to the maximum extent practicable within the existing right-of-way.
   - Underground structures that cannot be moved without significantly expanding the project scope.
   - Adjacent developed facilities, including buildings that would have to be removed or relocated to achieve accessibility.
   - Drainage cannot be maintained if the feature is made accessible.
   - Notable natural or historic features that would have to be altered in a way that lessens their aesthetic or historic value.
   - Underlying terrain that would require a significant expansion of the project scope to achieve accessibility.
   - Street grades within the crosswalk exceed the pedestrian access route maximum cross slopes, provided an engineering analysis has concluded that it cannot be done without significantly expanding the project scope (for example, changing from resurfacing an intersection to reconstructing that intersection).

5. **Safety Issues:** When accessibility requirements would cause safety issues, compliance is required to the maximum extent practicable.

6. **Documenting Exceptions:** If the project cannot fully meet accessibility requirements because the accessibility improvements are structurally impracticable, technically infeasible, or safety issues, a document should be developed to describe how the existing physical or site constraints or safety issues limit the extent to which the facilities can be made compliant. This document should identify the specific locations that cannot be made fully compliant and provide specific reasons why full compliance cannot be achieved. It is recommended that this document be retained in the project file. For local agency projects administered through Iowa DOT, an "Accessibility Exceptions Certification" (Form 517118) with supporting documentation shall be signed by a registered professional engineer or landscape architect licensed in the State of Iowa and submitted to the Iowa DOT administering office. The certification shall be as prescribed by Iowa DOT Local Systems I.M. 1.080. For Iowa DOT projects, contact the Office of Design, Methods Section.

   Note: Documenting exceptions does not remove an agency's responsibility to consider making accessibility improvements the next time the facility is altered because physical or site constraints and safety issues may change over time. The determination of exceptions and corresponding documentation needs to be made each time a facility is altered, based on the existing conditions and the scope of the proposed project.

7. **Reduction in Access:** Regardless of whether the additions or alterations involve the modification of the existing pedestrian circulation path, the resulting work cannot have the result of reducing the existing level of accessibility below the minimum requirements. For example, the installation of a bench cannot have the effect of reducing the width of the pedestrian access route to 3 feet (4 feet is the minimum). Likewise, the construction of an overlay cannot result in a street cross slope of more than 5%, nor have a lip at the curb ramp that exceeds 1/2 inch.

Pedestrian facilities may be removed if they are being re-routed for safety reasons, or terminated because they do not connect to a destination or another pedestrian circulation path.

8. **Addition of Pedestrian Facilities:** If a sidewalk exists on both sides of the street, curb ramps shall be installed on both sides when the street is altered. PROWAG does not require construction of pedestrian facilities where none currently exists, although the jurisdiction's transition plan may require them.

9. **Utility Construction:** If the pedestrian circulation path is disturbed during utility construction, the requirements of this section and Section 12A-4 shall apply.

E. Standards for Accessibility

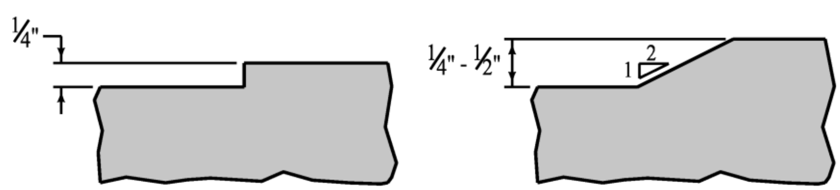
The following section summarizes the design standards for the elements of an accessible pedestrian access route. The minimum and maximum values stated are taken from the PROWAG. Target values are also provided. Designing features to the target values, rather than the allowable maximum or minimum, allows for appropriate construction tolerances and field adjustment during construction while maintaining compliance with the PROWAG standards.

1. **General Requirements:** These requirements apply to all parts of the pedestrian access route.

   a. **Surfacing:** PROWAG requires all surfaces to be firm, stable, and slip resistant (R302.7). All permanent pedestrian access routes, with the exception of some Type 2 shared use paths (see Section 12B-2), shall be paved. When crossing granular surfaced facilities, consider paving wider than the pedestrian access route; see the shared use path section.

   b. **Changes in Level:** Changes in level, including bumps, utility castings, expansion joints, etc. shall be a maximum of 1/4 inch without a bevel or up to 1/2 inch with a 2:1 bevel. Where a bevel is provided, the entire vertical surface of the discontinuity shall be beveled (R302.7.2).

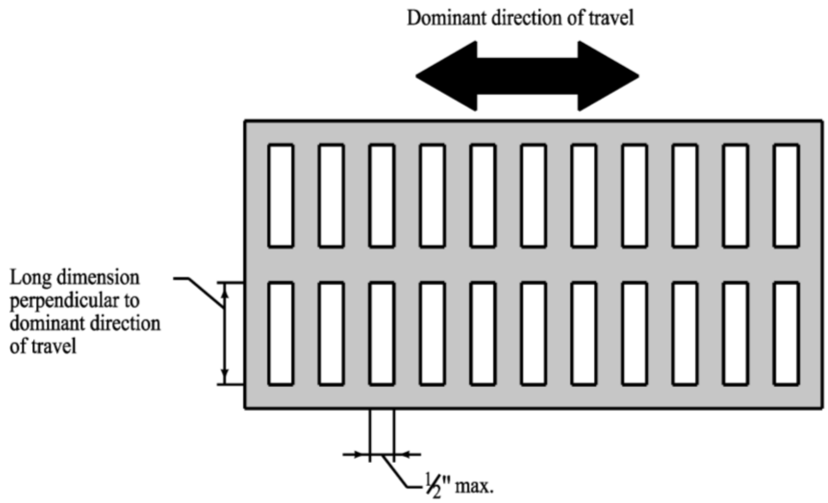
Figure 12A-2.01: Vertical Surface Discontinuities



   c. **Horizontal Openings:** Horizontal openings shall not allow passage of a sphere more than 1/2 inch in diameter. Elongated openings in grates shall be placed so the long dimension is perpendicular to the dominant direction of travel. The use of grates within the pedestrian access route is discouraged; however, where necessary, the grate should be located outside of curb ramp runs, turning spaces, and gutter areas if possible. (R302.7.3)

   It should be noted that none of the standard SUDAS/Iowa DOT intake grates meet the requirements for use within a pedestrian access route; therefore, a special design is required.

Figure 12A-2.02: Horizontal Openings



2. **Standard Sidewalk:** Sidewalks solely serving private residences are not required to follow these requirements.

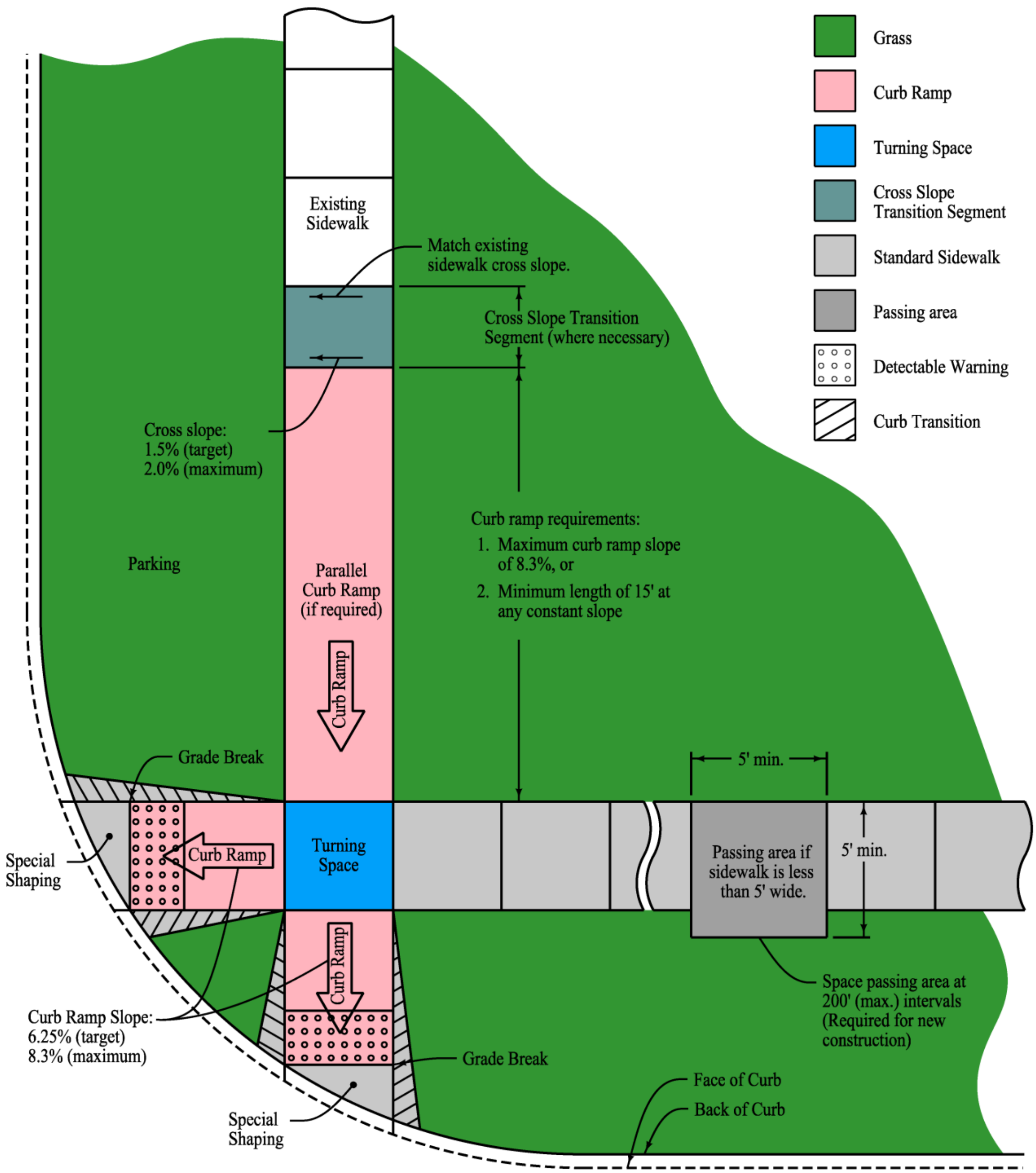
   a. **Cross Slope:** The maximum cross slope is 2.0% with a target value of 1.5% (R302.6).

   b. **Running Slope:** Sidewalks with a running slope of 5% or less are acceptable. However, where the sidewalk is contained within the street right-of-way, the grade of the sidewalk shall not exceed the general grade of the adjacent street (R302.5). For design, consider the general grade of the adjacent street to be within approximately 2% of the profile grade of the street.

   c. **Width:** The minimum width of the pedestrian access route is 4 feet. Five foot sidewalks are encouraged and may be required by the Jurisdiction. Iowa DOT will design 5 foot sidewalks unless otherwise requested. (R302.3)

   d. **Passing Spaces:** Where the clear width of the pedestrian access route is less than 5 feet, passing spaces are required at maximum intervals of 200 feet. The passing space shall be 5 foot minimum by 5 foot minimum. Passing spaces may overlap with the pedestrian access route. (R302.4). Driveways may be used as passing spaces, as long as the 2.0% maximum cross slope is not exceeded.

Figure 12A-2.03: Standard Sidewalk and Curb Ramp Elements

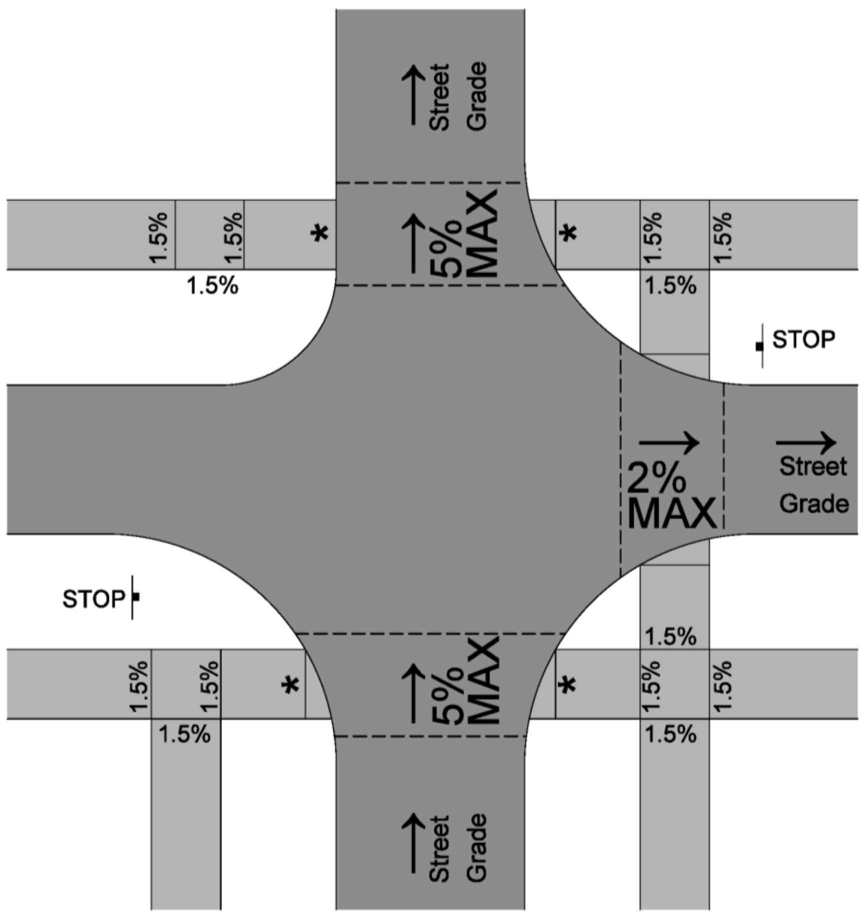


3. Pedestrian Street Crossings:

a. Cross Slope: The longitudinal grade of a street becomes the cross slope for a pedestrian street crossing. PROWAG has maximum limits for the cross slope of pedestrian street crossings, which vary depending on the location of the crossing and the type of vehicular traffic control at the crossing. These requirements, in effect, limit the longitudinal grade of a street, or require a "tabled crosswalk" at the intersection. (R302.6)

1) **Intersection Legs with Stop or Yield Control:** For pedestrian street crossings across an intersection leg with full stop or yield control (stop sign or yield sign), the maximum cross slope is 2.0% (maximum 2.0% street grade through the crossing).
2) **Intersection Legs without Stop or Yield Control:** For pedestrian street crossings across an intersection leg where vehicles may proceed without slowing or stopping (uncontrolled or signalized), the maximum cross slope of the pedestrian street crossing is 5.0% (maximum 5.0% street grade through the crossing).
3) **Midblock Pedestrian Street Crossings:** At midblock crossings, the cross slope of the pedestrian street crossing is allowed to equal the street grade.

Figure 12A-2.04: Example Street Intersection

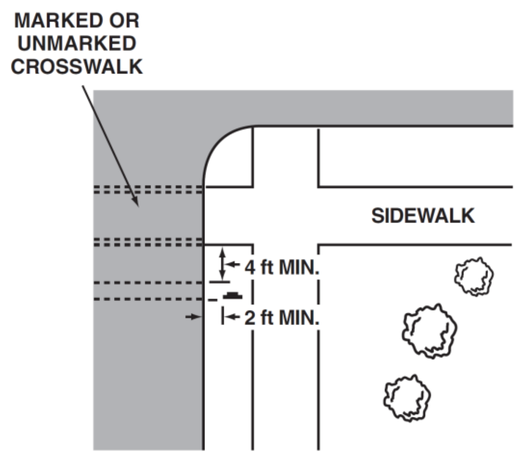


* Match pedestrian street crossing cross slope or flatter

b. Running Slope: The running slope of the pedestrian street crossing is limited to a maximum of 5.0% (maximum street cross slope or superelevation of 5.0%) (R302.5.1).

c. Location: Driver anticipation and awareness of pedestrians increases as one moves closer to the intersection. Therefore, curb ramps and pedestrian street crossings should be located as close to the edge of the adjacent traveled lane as practical. Where a stop sign or yield sign is provided, MUTCD requires the pedestrian street crossing, whether marked or unmarked, be located a minimum of 4 feet from the sign, between the sign and the intersection. It is recommended stop and yield signs be located no greater than 30 feet from the edge of the intersecting roadway; however, MUTCD allows up to 50 feet. Consult MUTCD for placement of curb ramps and pedestrian street crossings at signalized intersections.

Figure 12A-2.05: Pedestrian Street Crossing Location



Source: MUTCD, FHWA

d. Medians and Pedestrian Refuge Islands: Medians and pedestrian refuge islands in pedestrian street crossings shall be cut through level with the street or complying with the curb ramp requirements. The clear width of pedestrian access routes within medians and pedestrian refuge islands shall be 5.0 feet minimum (R302.3.1). If a raised median is not wider than 6 feet, it is recommended the nose not be placed in the pedestrian street crossing.

4. Curb Ramps:

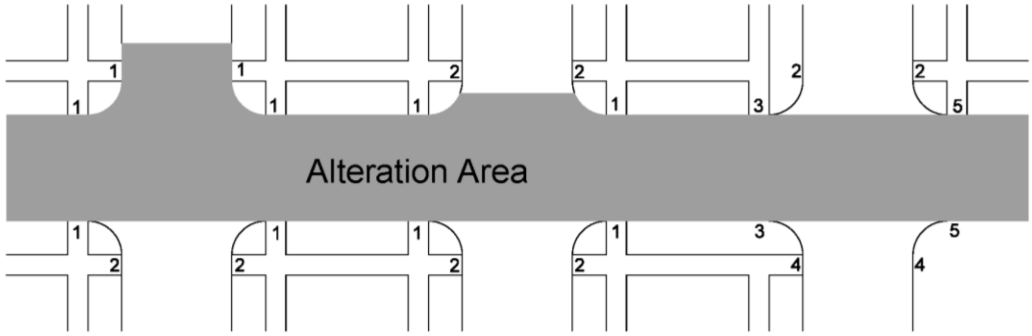
a. General: There are two types of curb ramps: perpendicular and parallel. Perpendicular curb ramps are generally perpendicular to the traffic they are crossing with the turning space at the top. Parallel curb ramps have the turning space at the bottom. Parallel curb ramps may be used where the sidewalk begins at or near the back of curb and there is little or no room between the sidewalk and curb for a perpendicular curb ramp.

A separate curb ramp is required at each pedestrian street crossing for new construction. Parallel ramps with a large turning space, as shown in Figure 12A-2.08, are allowed. For alterations, follow the new construction requirements if possible; however, a single diagonal curb ramp is allowed but not recommended where existing constraints prevent two curb ramps from being installed.

For transitions into and out of driveways, curb ramp requirements may be used.

For curb ramps within and near an alteration area, see Figure 12A-2.06.

Figure 12A-2.06: Curb Ramps for Alterations



1. Required.
2. Strongly recommended.
3. Required due to barriers in the path of travel between the sidewalk on one side of the street to the sidewalk on the other side of the street.
4. Recommended, but not required because it is outside the alteration area. Consider based on pedestrian usage, safety, and land development.
5. Install both sides or remove the existing one, based on pedestrian usage, safety, and land development.

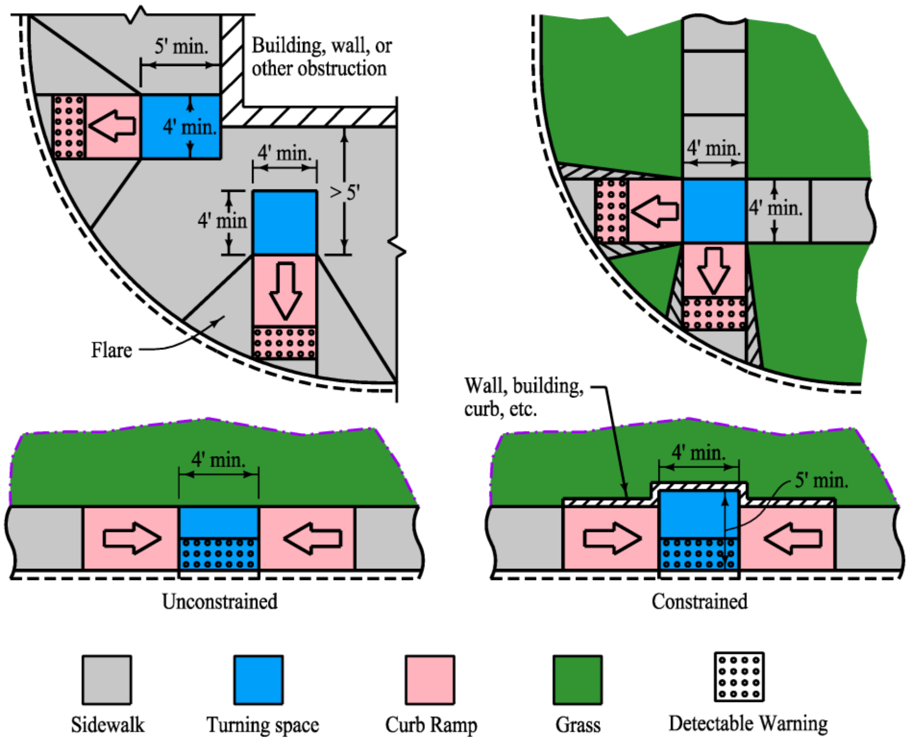
b. Technical Requirements:

1) **Cross Slope:** The maximum cross slope is 2.0% with a target value of 1.5%; however, for intersection legs that do not have full stop or yield control (i.e. uncontrolled or signalized) and at mid-block crossings, the curb ramp cross slope is allowed to match the cross slope in the pedestrian street crossing section. See "pedestrian street crossings" for additional details. (R304.5.3)
2) **Running Slope:** Provide curb ramps with a target running slope of 6.25% and a maximum slope of 8.3%; however, curb ramps are not required to be longer than 15 feet, regardless of the resulting slope. (R304.2.2 and R304.3.2)
3) **Width:** The minimum width of a curb ramp is 4 feet, excluding curbs and flares. If the sidewalk facility is wider than 4 feet, the target value for the curb ramp is equal to the width of the sidewalk. (R304.5.1)
4) **Grade Breaks:** Grade breaks at the top and bottom of curb ramps must be perpendicular to the direction of the curb ramp run. Grade breaks are not allowed on the surface of curb ramp runs and turning spaces. (R304.5.2)
5) **Flared Sides:** For perpendicular curb ramps on Class A sidewalks, or configurations where the pedestrian circulation path crosses the curb ramp, PROWAG requires the flares along the sides of the curb ramp to be constructed at 10% or flatter. (R304.2.3) This allows pedestrians to approach the curb ramp from the side and prevents a tripping hazard. It is recommended to design these flares at a slope between 8% and 10%, which will clearly define the curb ramp from the sidewalk.
6) **Clear Space:** At the bottom of perpendicular curb ramps, a minimum 4 foot by 4 foot area must be provided within the width of the pedestrian street crossing, but wholly outside of the parallel vehicle travel lanes. (R304.5.5)
7) **Turning Space:** Turning spaces allow users to stop, rest, and change direction on the top or bottom of a curb ramp (R304.2.1 and R304.3.1).
   a) **Placement:** A turning space is required at the top of perpendicular curb ramps and at the bottom of parallel curb ramps.
   b) **Slope:** The maximum cross slope and running slope is 2.0% with a target value of 1.5% (R304.2.2 and R304.3.2). When turning spaces are at the back of curb, cross slopes may be increased to match allowable values in the pedestrian street crossing section (R304.5.3).

c) **Size:** The turning space shall be a minimum of 4 feet by 4 feet. Where the turning space is constrained on one or more sides, provide 5 feet in the direction of the pedestrian street crossing.

8) **Special Shaping Area:** Transition area between the back of curb and the grade break. The longest side cannot exceed 5 feet.

Figure 12A-2.07: Curb Ramp Turning Spaces



c. Curb Ramp Design Considerations:

1) **Combination Curb Ramps:** For many intersection configurations, a perpendicular curb ramp will not provide enough length to establish the top turning space at the sidewalk elevation; in these situations, a parallel curb ramp is often required to transition from the turning space up to the sidewalk elevation. The use of a perpendicular curb ramp from the curb to the turning space in conjunction with a parallel curb ramp between the turning space and the sidewalk elevation is referred to as a combination curb ramp. When transitioning from a turning space to sidewalk elevation on a steep street, it is not necessary to chase the grade. As noted in the technical requirements above, a parallel curb ramp is not required to exceed 15 feet in length, regardless of the resulting curb ramp slope. In practice, the parallel curb ramp should be extended to the next joint beyond 15 feet.

2) **Cross Slope Transition Segment:** When connecting to existing construction that is out of cross slope compliance, the cross slope transition should be completed beyond the parallel curb ramp or turning space; this recommendation eliminates the need to list this curb ramp in the transition plan. It is recommended this cross slope transition take place at 1% per foot or less. Typically, this can be accomplished in a single panel.

3) **Parking Slope:** In situations where the length of the perpendicular curb ramp is insufficient to bring the turning space up to sidewalk elevation, consider lowering the sidewalk and flattening the parking slope.

5. **Blended Transitions:** A blended transition is allowed but not recommended. Design and constructability is difficult to meet compliance requirements. In lieu of a blended transition, a curb ramp or standard sidewalk should be used.

6. **Detectable Warnings:**

   a. **General:** Detectable warning surfaces are detected underfoot or with a cane by blind and low vision individuals. The warnings indicate the location of the back of curb. Detectable warnings also provide a visual queue to pedestrians with low vision and aid in locating the curb ramp across the street. For these reasons, the detectable warning shall contrast visually (light on dark or dark on light) from the surrounding paved surfaces (R305.1.3).

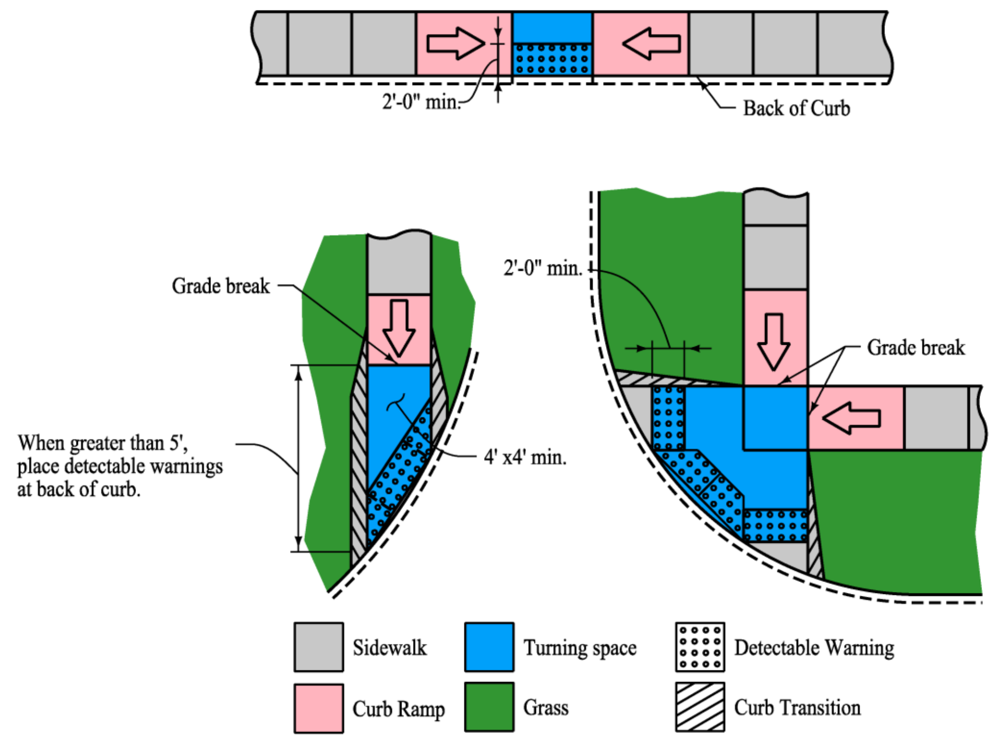
   b. **Location:** Detectable warnings shall be installed at all pedestrian street crossings and at-grade rail crossings (R208.1). Detectable warning surfaces should not be provided at crossings of residential driveways since the pedestrian right-of-way continues across the driveway. Where commercial driveways are provided with yield control, stop control, or traffic signals at the pedestrian access route, detectable warnings should be installed at the junction between the pedestrian access route and the driveway (Advisory R208.1).

   c. **Size:** Detectable warning surfaces shall extend a minimum of 2 feet in the direction of pedestrian travel and extend the full width of the curb ramp or pedestrian access route (R305.1.4).

   d. **Dome Orientation:** On curb ramps, the rows of truncated domes should be aligned perpendicular to the grade break so pedestrians in wheelchairs can track their wheels between the domes. On surfaces less than 5% slope, dome orientation is less critical.

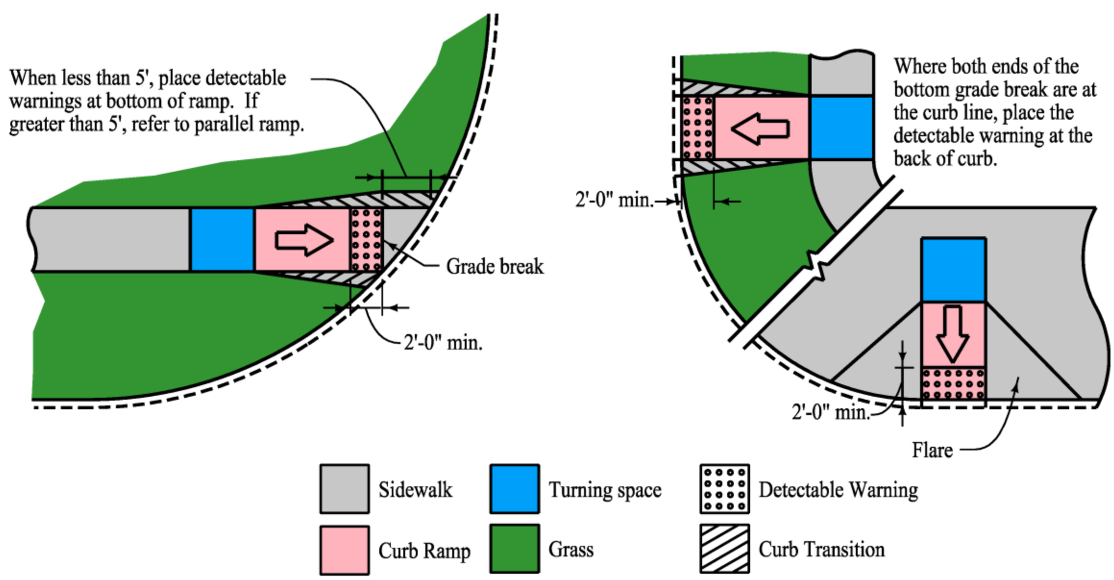
   e. **Parallel Curb Ramps:** On parallel curb ramps, detectable warning shall be placed on the turning space at the back of curb (R305.2.2).

Figure 12A-2.08: Detectable Warnings on Parallel Curb Ramps



f. **Perpendicular Curb Ramps:** Placement of detectable warning varies based upon location of grade break as shown in Figure 12A-2.09.

Figure 12A-2.09: Detectable Warnings on Perpendicular Curb Ramps



g. **Refuge Islands:** Where refuge islands are 6 feet wide or greater from back of curb to back of curb, detectable warning shall be placed at the edges of the pedestrian island and separated by a minimum 2 foot strip without detectable warnings. Where the refuge island is less than 6 feet wide, a 2 foot strip without detectable warnings cannot be installed. In these situations, detectable warnings shall not be installed at the island and the pedestrian signal must be timed for full crossing. (R208.1 and R208.2)

h. **Rural Cross-section:** Detectable warnings should be placed similar to urban layouts, except at the edge of shoulder instead of the back of curb.

F. Bus Stop

1. **Bus Stop Pads:** New and altered bus stop pads shall meet the following criteria.
   - Provide a firm, stable, and slip resistant surface (R308.1.3.1).
   - Provide a minimum clear length of 8 feet (measured from the curb or roadway edge) and minimum clear width of 5 feet (measured parallel to the roadway) (R308.1.1.1).
   - Connect the pad to streets, sidewalks, or pedestrian circulation paths with at least one accessible route (R308.1.3.2).
   - The slope of the pad parallel to the roadway will be the same as the roadway to the maximum extent practicable (R308.1.1.2).
   - Provide a desirable cross slope of 1.5% up to a maximum cross slope of 2.0% perpendicular to the roadway (R308.1.1.2).

2. **Bus Shelters:** Where new or replaced bus shelters are provided, install or position them to allow a wheelchair user to enter from the public way. An accessible route shall be provided from the shelter to the boarding area. (R308.2)

G. Accessible Pedestrian Signals

An accessible pedestrian signal is an integrated device that communicates information about the WALK and DON'T WALK intervals at signalized intersections in a non-visual format (i.e. audible tones and vibrotactile surfaces) to pedestrians who have visual disabilities. Consistency throughout the pedestrian system is very important. Contact the Jurisdictional Engineer regarding the standards and equipment types that should be incorporated into the design of the accessible pedestrian system. Where new or altered pedestrian signals and pushbuttons are provided they shall comply with MUTCD 4E.08 through 4E.13. Operable parts shall comply with R403. (R209.1)

1. **New Pedestrian Signals:** Each new traffic signal project location should be evaluated to determine the need for accessible pedestrian signals. An engineering study should be completed that determines the needs for pedestrians with visual disabilities to safely cross the street (MUTCD 4E.09). The study should consider the following factors:
   - Potential demand for accessible pedestrian signals
   - Requests for accessible pedestrian signals by individuals with visual disabilities
   - Traffic volumes when pedestrians are present, including low volumes or high right turn on red volumes
   - The complexity of the signal phasing, such as split phasing, protected turn phases, leading pedestrian intervals, and exclusive pedestrian phases
   - The complexity of the intersection geometry

   If a pedestrian accessible signal is warranted, audible tones and vibrotactile surfaces should be included. Pedestrian push buttons should have locator tones for the visually impaired individual to be able to access the signal.

2. **Existing Pedestrian Signals:** Excluding routine maintenance or repairs due to accidental damage, when the existing pedestrian signal controller and software are altered, or the pedestrian signal head is replaced, the pedestrian signals shall include accessible pedestrian signals and pushbuttons. (R209.2)

   If pedestrian signals are non-compliant, upgrades are recommended but not required when alterations are being made to the pedestrian circulation path.

H. On-Street Parking

- When on-street parking is marked or metered, provide accessible parking spaces according to Table 12A-2.01 (R214 and R309.1).

Table 12A-2.01 On-Street Accessible Parking Spaces

Total Number of Marked or Metered Parking Spaces on the Block Perimeter	Minimum Required Number of Accessible Parking Spaces
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 and over	4% of total

- Identify accessible parking spaces by displaying signs with the International Symbol of Accessibility (R411).
- Comply with R403 Operable Parts for parking meters and pay stations that serve accessible parking spaces.
- Locate accessible parking spaces where the street has the least crown and grade (R309.1).
- Accessible parking spaces located at the end of the block can be served by the curb ramps or blended transitions at the pedestrian street crossing (R309.4).
- Keep sidewalks adjacent to parallel accessible parking spaces free of signs, street furniture, and other obstructions. Locate curb ramps or blended transitions so the van side-lift or ramp can be deployed to the sidewalk (R309.2)
- At parallel accessible parking spaces, locate parking meters at the head or foot of the parking space (R309.5.1). Ensure information is visible from a point located 3.3 feet maximum above the center of the clear space in front of the parking meter or parking pay station (R309.5.2).
- For areas where the sidewalk width or available right of way exceeds 14 feet, provide an access aisle 5 feet wide at street level the full length of the parallel parking space and connect it to a pedestrian access route (R309.2.1). When an access aisle is not provided due to the sidewalk or right-of-way not exceeding 14 feet, locate the accessible parallel parking space at the end of the block face (R309.2.2)
- Provide an 8 feet wide access aisle the full length of the parking space for perpendicular or angled accessible parking spaces. Two accessible parking spaces are allowed to share a common access aisle (R309.3).
- For perpendicular or angled spaces, connect the access aisle to the pedestrian access route with a curb ramp. Do not locate curb ramps within the access aisle (R309.4).

Protruding Objects

A. Introduction

This section provides guidance to comply with section R402 of PROWAG. The pedestrian area is any prepared area available for pedestrians (equivalent to the pedestrian circulation path as defined in PROWAG). A protruding object is any obstacle that reduces the clearance width and/or the clearance height within a pedestrian area. The pedestrian area is not limited to the sidewalk or the pedestrian access route intended by the designer. The pedestrian area includes any areas that may be perceived as a pedestrian walking space, including adjacent parking lots and paved frontage.

Common protruding objects include:

- Signs and Sign poles
- Landscaping and branches
- Utility boxes or poles and their stabilizing wires
- Mailboxes (public and private)
- Trash cans
- Transit shelters
- Bike racks
- Planters
- Fire hydrants
- Parking meters
- Benches
- Public Art

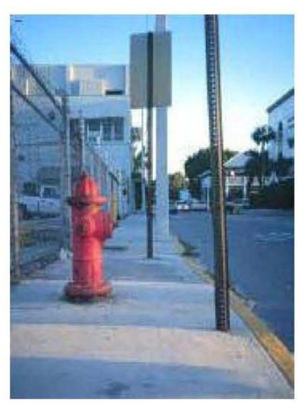
B. Protruding Object Locations

1. **Outside the Pedestrian Area:** A protruding object can result in narrow passing spaces, reduced access, and injury. Therefore, protruding objects should be placed completely outside of the pedestrian area whenever possible.

2. **Within the Pedestrian Area:** Ideally, the full width of the pedestrian area should be free of protruding objects and the pedestrian access route would be clearly separated from other paved surfaces. However, if some obstacles must be located within the pedestrian area, they should all be placed either right or left of center to provide a consistent pedestrian access route. Figure 12A-3.01 shows an acceptable pedestrian area with obstacles aligned, providing a consistent pedestrian access route. Figure 12A-3.02 shows an undesirable pedestrian area with a poorly defined pedestrian access route. The pedestrian access route within the pedestrian area must meet guidelines defined in this chapter. Special sidewalk treatments (such as brick pavers or stamped concrete) are recommended to provide a different surface texture to differentiate between the object corridor and the pedestrian access route.

Figure 12A-3.01: Acceptable Pedestrian Area

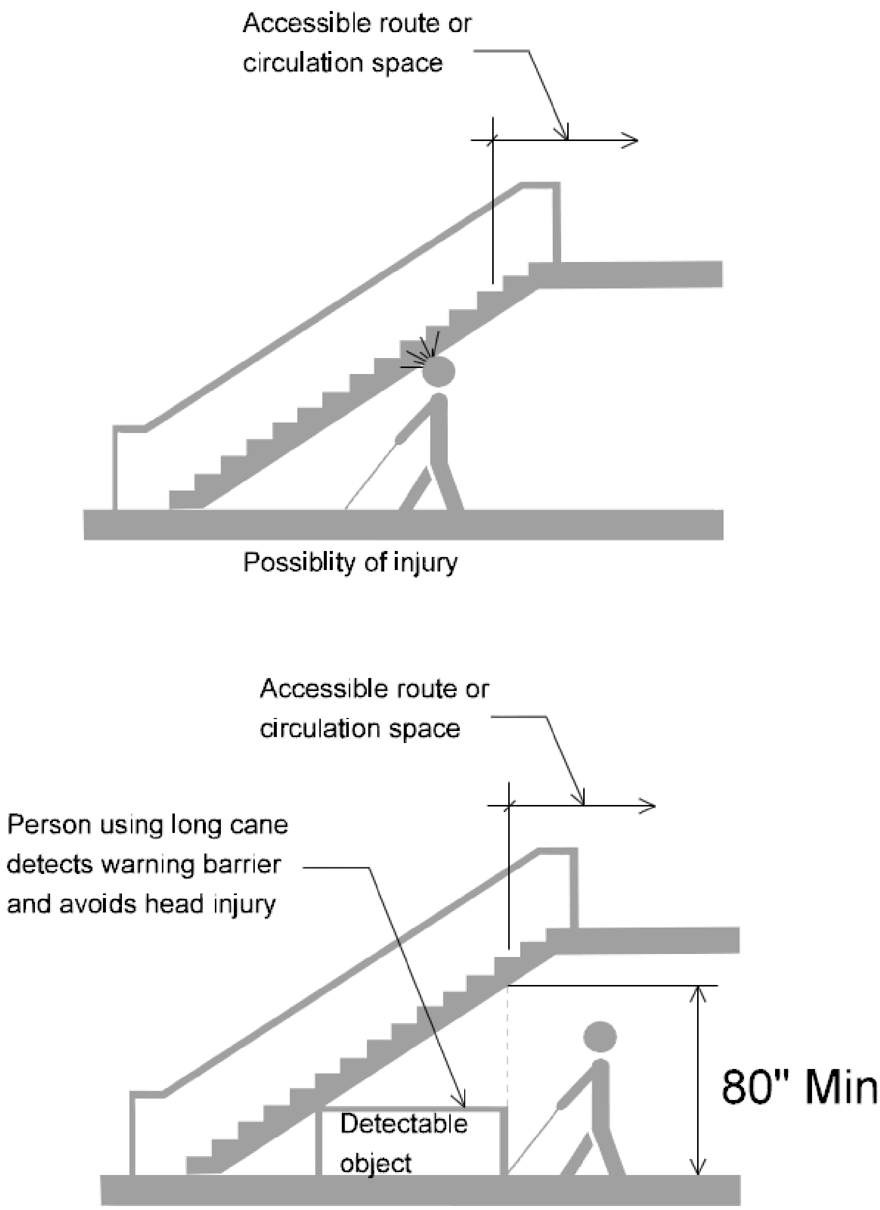
Figure 12A-3.02: Undesirable Pedestrian Area



C. Clearance

1. **Vertical Clearance:** Vertical clearance is minimum unobstructed vertical passage space required along the entire width of the pedestrian corridor. A minimum vertical clearance of 80 inches must be provided or the object must be shielded with a barrier. The leading edge of the barrier shall be a maximum of 27 inches above the finished surface. See Figure 12A-3.03.

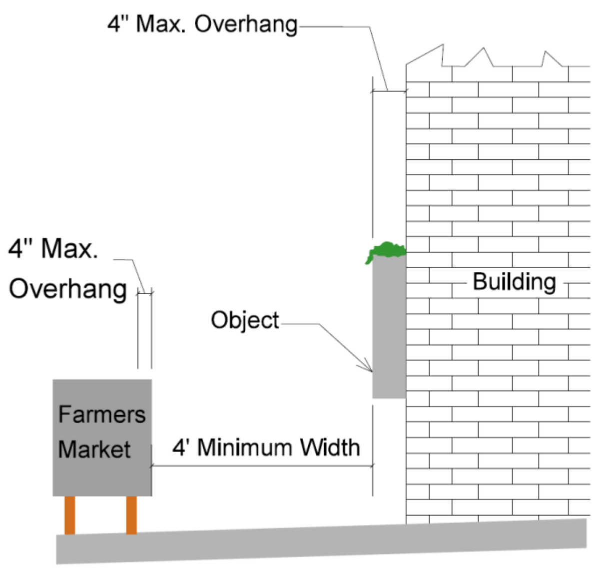
Figure 12A-3.03: Shielding for Vertical Clearance Obstacles



2. **Horizontal Clearance:** Objects mounted at or below 27 inches may extend from a fixed structure into the pedestrian area, provided the remaining sidewalk width complies with Section 8A-2. Objects that extend below 27 inches are easily detectable by most pedestrians.

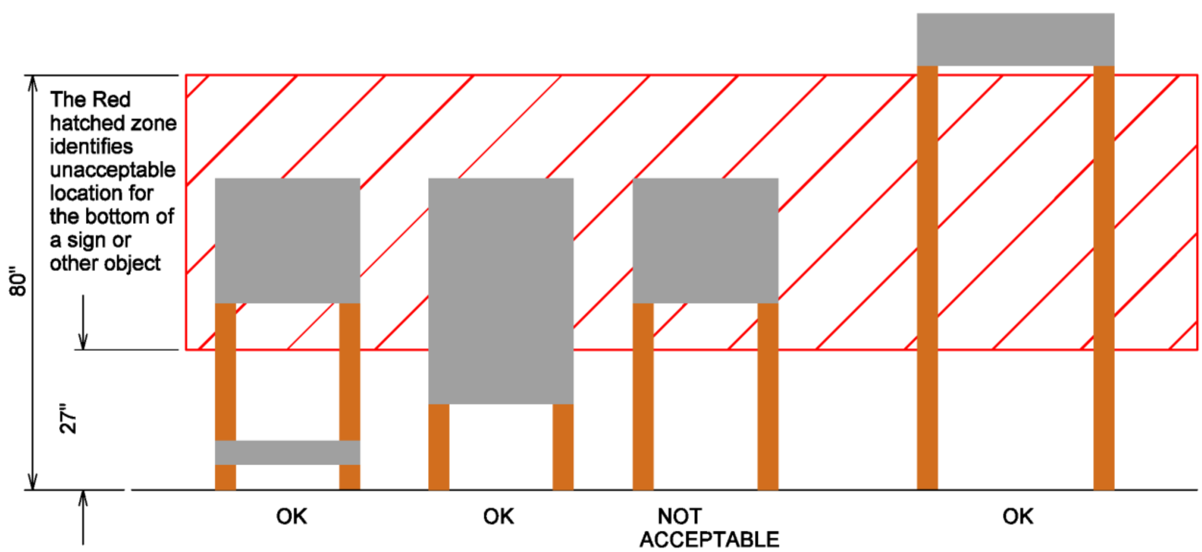
Objects that extend into the pedestrian area at a height above 27 inches are not easily detected with a cane and pedestrians may walk into them. This type of object cannot extend into the pedestrian corridor more than 4 inches from its base. The base shall be at least 2.5 inches in height. See Figure 12A-3.04.

Figure 12A-3.04: Vertical Clearance



3. **Objects Mounted Between Posts:** Where an object is mounted between posts or pylons and the clear distance between the posts or pylons is greater than 12 inches, the lowest edge of the object shall be between 0 and 27 inches or 80 inches or more above the ground (see Figure 12A-3.05). For objects mounted on posts closer than 12 inches, follow the requirements for horizontal clearance defined above.

Figure 12A-3.05: Height Restriction for Signs Mounted Between Posts



Pedestrian Facilities During Construction

A. Introduction

When projects impact pedestrians, it is important for the engineer to develop a temporary traffic control plan for pedestrians, including those with disabilities. For Iowa DOT projects, see Iowa DOT Design Manual Section 9A-5 for temporary traffic control plans. The applicable guidelines for the temporary traffic control plan are the July 26, 2011 "Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way" (PROWAG) and the Manual on Uniform Traffic Control Devices (MUTCD).

According to PROWAG, when a pedestrian circulation path is temporarily closed for construction or maintenance activities, an alternate pedestrian access route complying with sections 6D.01, 6D.02, and 6G.05 of the MUTCD shall be provided (R205). However, MUTCD (Section 6D.01) also requires knowledgeable persons to conduct appropriate evaluations or use engineering judgment in determining temporary traffic controls for pedestrian circulation paths. This section includes guidance on conducting the evaluation when an alternate pedestrian access route may not be practical.

B. Evaluating Pedestrian Needs

The initial design activity should be to determine the level of the accessibility of the current pedestrian circulation path within the area of the project and the adjacent areas. The impact to the pedestrian circulation path, including transit stops, from the construction or maintenance activity needs to be determined. Develop pedestrian accommodations to provide the best accessibility practical through all stages of work. Consider obtaining local input through a public meeting or contact with residents or public officials to see where additional accessibility needs should be addressed (e.g. senior centers, medical facilities, schools, public facilities, etc.).

Whenever possible, the work should be done in such a manner that does not create a need to detour pedestrians from existing routes. Pedestrians rarely observe detours and the cost of providing accessibility and detectability might outweigh the cost of maintaining a continuous route through the construction zone (MUTCD 6D-01). All methods should be given consideration, including providing alternate means of traversing the construction zone. If pedestrians are to be directed through the construction zone, safety as well as accessibility must be addressed. If a pedestrian detour is developed, it should replicate the accessibility of the existing route.

C. Facility Options

To address the impacts to the pedestrian circulation path, including transit stops, consider the following:

- Develop a temporary traffic control plan to guide the pedestrians through the construction zone.
- Close the pedestrian circulation path through the construction zone.
- Close the pedestrian circulation path through the construction zone; develop a detour route consistent with the accessibility features present in the pedestrian circulation path being closed.
- Provide alternate means for pedestrians to traverse the construction zone, such as free accessible shuttles or other forms of assistance.

D. Barricades, Channelizing Devices, and Signs

Pedestrian barricades and channelizing devices shall comply with sections 6F.63, 6F.68, and 6F.71 of the MUTCD.

1. **Barricades:** Barricades are used for pedestrian circulation path closures. See Iowa DOT Specifications Section 2528.

2. **Channelizing Devices:** The designer should consider the safety of pedestrians and vehicles when choosing channelizing devices.

   a. **Type A:** Type A devices are redirective barriers designed for highway applications. These devices are suitable when pedestrians are routed into the travel way and allow for the most protection for pedestrians from vehicular intrusion.

   b. **Type B:** Type B devices are crashworthy but do not redirect vehicles. These devices are designed to minimize risks associated with flying debris.

   c. **Type C:** Type C devices include any device that meets ADA requirements for channelizing pedestrians and may not be crashworthy. These devices are for locations where vehicular intrusions are unlikely (e.g. closed roads, when there is a separation between pedestrians and vehicular traffic, or where vehicular traffic is at low speeds).

3. **Signs:** See Iowa DOT Standard Road Plan TC-601 and TC-602.

E. Temporary Pedestrian Facilities

Temporary pedestrian facilities should comply with the other sections within this chapter to the extent practical. It is strongly recommended that detour routes be on paved surfaces.

Temporary pedestrian facility surfaces must be firm, stable, and slip resistant. Granular surfacing for short term, temporary pedestrian facilities is acceptable. The granular surfacing material should be well graded, such as Class A road stone (Iowa DOT Specifications Section 4196, Gradation No. 8) or special backfill (Iowa DOT Specifications Section 4196, Gradation No. 30). Maintenance of the temporary pedestrian facility surface to meet the firm, stable, slip resistant, and minimum width is required at all times. The temporary pedestrian facility surface must be removed and a permanent pedestrian facility must be replaced prior to the end of the construction season.

F. Utility Construction

If the pedestrian circulation path is disturbed during utility construction, the requirements of this section and Section 12A-2 shall apply.

Bicycle and Pedestrian Facilities

A. Introduction

There are four major categories for bicycle and pedestrian facilities: sidewalks, shared use paths, on-street, and trails. Sidewalks are an integral component of the transportation system, usually used only by pedestrians. For information on designing sidewalks, see Section 12A-1 and Section 12A-2. Shared use paths are also an integral component of the transportation system and use the sidewalk standards, but must also be designed for bicycle usage. Shared use paths are generally separate from the street, but in limited instances it may be necessary to utilize an on-street facility.

The word "trail" has conflicting definitions in ADA, AASHTO, program funding, and common usage. Projects developed around the state and those let through the Iowa DOT are generally shared use paths as defined by the Access Board, not trails. Facilities with a transportation purpose cannot use the trail guidelines published by the Access Board, even though they are commonly referred to as trails. The trail information from the Access Board only applies in parks and other limited locations; therefore, they are not covered in this manual.

B. Definitions

The following definitions are from the "AASHTO Guide for the Development of Bicycle Facilities" (or *AASHTO Bike Guide*).

Bicycle Boulevard: A street segment, or series of contiguous street segments, that has been modified to accommodate through bicycle traffic and minimize through motor traffic.

Bicycle Facilities: A general term denoting improvements and provisions to accommodate or encourage bicycling, including parking and storage facilities, and shared roadways not specifically defined for bicycle use.

Bicycle Lane or Bike Lane: A portion of roadway that has been designated for preferential or exclusive use by bicyclists by pavement markings and, if used, signs. It is intended for one-way travel, usually in the same direction as the adjacent traffic lane, unless designed as a contra-flow lane.

Bicycle Route: A roadway or bikeway designated by the jurisdiction having authority, either with a unique route designation or with BIKE ROUTE signs, along which bicycle guide signs may provide directional and distance information. Signs that provide directional, distance, and destination information for bicyclists do not necessarily establish a bicycle route.

Bikeway: A generic term for any road, street, path, or way that in some matter is specifically designated for bicycle travel, regardless of whether such facilities are designated for the exclusive use of bicycles or are to be shared with other transportation modes.

Independent Right-of-Way: A general term denoting right-of-way outside the boundary of a conventional highway.

Roundabout: A type of circular intersection that provides yield control to all entering vehicles and features channelized approaches and geometry to encourage reduced travel speeds through the circular roadway.

Rumble Strips: A textured or grooved pavement treatment designed to create noise and vibration to alert motorists of a need to change their path or speed. Longitudinal rumble strips are sometimes used on or along shoulders or center lines of highways to alert motorists who stray from the appropriate traveled way. Transverse rumble strips are placed on the roadway surface in the travel lane, perpendicular to the direction of travel.

Shared Lane: A lane of a traveled way that is open to both bicycle and motor vehicle travel.

Shared Lane Marking: A pavement marking or symbol that indicates an appropriate bicycle positioning in a shared lane.

Shared Use Path: (From U.S. Department of Transportation, Federal Highway Administration) The term “shared use path” means a multi-use trail or other path, physically separated from motorized vehicular traffic by an open space or barrier, either within a highway right-of-way or within an independent right-of-way, and usable for transportation purposes. Shared use paths may be used by pedestrians, bicyclists, skaters, equestrians, and other nonmotorized users.

Traveled Way: The portion of the roadway intended for the movement of vehicles, exclusive of shoulders and any bike lane immediately inside of the shoulder.

C. Design Process

Comprehensive systematic design is necessary to ensure a useful shared use path or on-street bicycle facility is provided for the public. To do this, the following items need to be addressed.

1. Identification of need of shared use path(s) and/or on-street bicycle system.

2. Determine objective of shared use path(s) and/or on-street bicycle facility.

3. Develop shared use path(s) and/or on-street bicycle facility potential use.

4. Route(s) evaluation, location, and selection:
    - Adequate access
    - Directness and convenience
    - Continuity with shared use path network
    - Attractiveness of route
    - Safety and security
    - Delays along route
    - Cost of improvements
    - Shared use of facility
    - Maintenance
    - Conflicts with other vehicles
    - Adequacy of street use
        - Grades and geometrics
        - Surface obstructions and conditions
        - Traffic volumes and speeds
        - Truck and bus traffic
        - Parking
        - Intersection conditions

    - Signing and pavement markings
    - Sight distances
    - Clearance (vertical and horizontal)
    - Bridge and railroad crossings

5. Choosing an appropriate facility type. (Refer to *AASHTO Bike Guide* Exhibit 2.3 for more information in selecting a facility type).
    - Shared lanes
    - Paved shoulders
    - Bike lanes
    - Bike boulevards
    - Shared use paths

Shared Use Path Design

A. Accessible Shared Use Path Design

1. **General:** Applicable portions from the following draft documents were used to develop this section.

   a. **AASHTO Bike Guide:** The fourth edition (2012) of the AASHTO "Guide for the Development of Bicycle Facilities" (or *AASHTO Bike Guide*). References made to the *AASHTO Bike Guide* within this section are shown in parentheses, e.g. (AASHTO 5.2.1).

   b. **AGODA:** The June 20, 2007 Proposed Architectural Barriers Act "Accessibility Guidelines for Outdoor Developed Areas" (AGODA). This document is primarily used for shared use paths designed as bicycle facilities.

   c. **PROWAG:** The July 26, 2011 "Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way," also known as the Public Right-of-Way Accessibility Guidelines or PROWAG. This document is primarily used for shared use paths designed as sidewalks.

2. **Documenting Exceptions:** If the project cannot fully meet the minimum requirements included within this section, a document should be developed to describe why the minimum requirements cannot be met. It is recommended that this document be retained in the project file. For local agency projects administered through Iowa DOT, a certification with supporting documentation shall be submitted to the Iowa DOT administering office. The certification shall be as prescribed by the Iowa DOT and signed by a registered professional engineer or landscape architect licensed in the State of Iowa. For Iowa DOT projects, contact the Office of Design, Methods Section.

B. Shared Use Path Categories

1. **Type 1:** A shared use path adjacent or in close proximity to the roadway and functions similar to a sidewalk. In rural cross-sections, these paths would be at the top of the foreslope. These paths are generally used for transportation purposes.

2. **Type 2:** A shared use path similar to Type 3, except they serve as a transportation route to facilities that fulfill a basic life need, provide access to a program or service, or provide a safe route for non-drivers.

3. **Type 3:** A shared use path in independent right-of-way or not in close proximity to the roadway. Although Type 3 paths may fulfill a transportation function, these paths primarily serve a recreation and fitness benefit.

One shared use path project may have different combinations of Type 1, Type 2, and/or Type 3 segments, based on location and function. If Federal or State funding is being used on a project, the funding application should identify where Type 1, Type 2, or Type 3 segments will be used.

C. Shared Use Path Design Elements

The following considerations should be used as a guide when designing shared use paths.

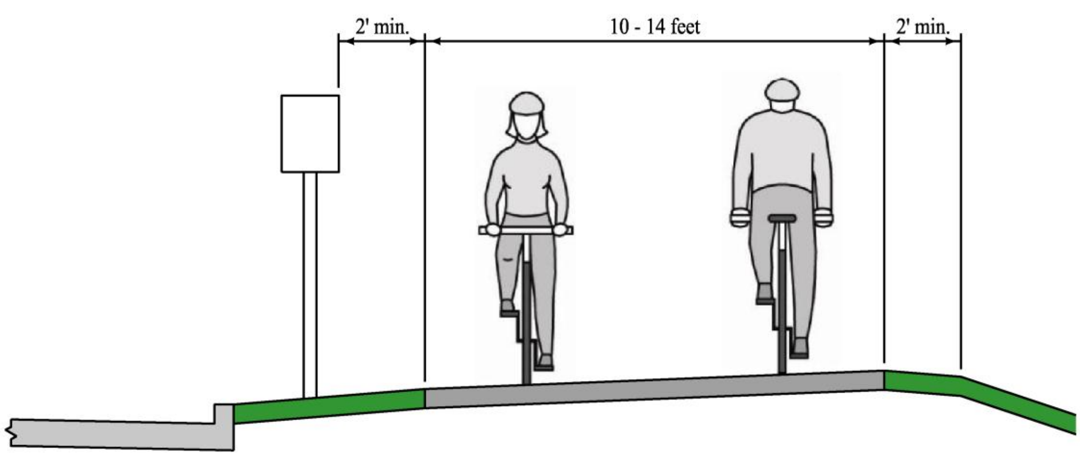
1. **Width:** A bicyclist requires a minimum of 4 feet and a preferred 5 feet of essential operating space based upon their profile. The typical path width is 10 feet to accommodate two-way traffic. Consider wider paths (11 to 14 feet) when at minimum one of the following is anticipated:
   - User volume exceeding 300 users within the peak hour.
   - Curves where more operating space should be provided.
   - Large maintenance vehicles.
   - There is a need for a bicyclist to pass another path user while maintaining sufficient space for another user approaching from the opposing direction. 11 feet is the minimum width for three lanes of traffic.

   Path width can be reduced to 8 feet where the following conditions prevail:
   - Bicycle traffic is expected to be low.
   - Pedestrian use is generally not expected.
   - Horizontal and vertical alignments provide well-designed passing and resting opportunities.
   - The path will not be regularly subjected to maintenance vehicle loading conditions.
   - A physical constraint exists for a short duration such as a utility structure, fence, etc.

   Path widths between 8 and 5 feet should be avoided; paths less than 5 feet do not meet ADA requirements.

   If segregation of pedestrians and bicycle traffic is desirable, a minimum 15 foot width should be provided. This includes 10 feet for two-way bicycle traffic and 5 feet for two-way pedestrian traffic. (AASHTO 5.2.1).

Figure 12B-2.01: Typical Cross-Section of Two-Way Shared Use Path on Independent Right-of-Way



Source: Adapted from *AASHTO Bike Guide* Exhibit 5.1

2. **Minimum Surface Thickness:** For Iowa DOT projects, contact the Pavement Design Section in the Office of Design for a pavement determination. For local agency projects administered through Iowa DOT, Iowa DOT will accept the thickness design as determined by the engineer.

For local projects, the pavement depth for both PCC and HMA pavements should have a minimum of 4 inches and a recommended thickness of 5 inches; if pavement thickness is proposed to be less than 4 inches, a pavement determination should be completed and documented.

3. **Cross Slope:** Shared use paths must have the capabilities to serve people with disabilities.
   a. **Type 1 and Type 2:** Cross slopes shall not exceed the requirements in Section 12A-2.
   b. **Type 3:** A 1.5% cross slope is recommended, but cross slopes should be a minimum of 1% and shall not exceed 5%. Cross slopes greater than 2% should be sloped to the inside of the horizontal curve regardless of drainage conditions. On unpaved paths, cross slopes may increase up to 5% due to the need of draining water off the path. On rare bicycle only facilities, the path does not need to meet accessibility guidelines and the cross slope can be between 5% and 8%. Cross slope transition should be comfortable for the user; therefore, a minimum transition length of 5 feet for each 1% change in cross slope should be used.

4. **Separation of Roadway and Path:** A separation should be provided between a two-way shared use path and the adjacent roadway to demonstrate to both the bicyclist and the vehicle driver that each facility is independent of the other. This is particularly important at night. If the separation from the face of the curb or the edge of the traveled way to the near edge of the path is less than 5 feet, a barrier or railing is recommended. The barriers or railings need not be of the size and strength to redirect errant motorists unless a crashworthy barrier is needed due to high speeds and clear zone requirements. Barriers at other locations serving only as a separation should be the height of standard guardrail.

   If needed, barriers and railings should be used, but since they can create considerable concerns in urban areas due to aesthetics, visibility, and maintenance problems, it may be necessary to initiate the documenting exceptions process (Section 12B-2, A, 2). The separation between the face of the curb and the path should be maximized, but with the presence of the curb, some landscaping area, and street lighting, the overall objectives of the separation can be satisfied.

5. **Lateral and Vertical Clearance:** Perhaps the most critical factor in developing safe and comfortable shared use path facilities is the provision of adequate clearance to a wide variety of potential obstructions that may be found along a prospective route. Guidelines for lateral and vertical clearance are particularly important in view of the wide range of riding proficiency that is found among riders. Clearance consideration must include:

   a. **Lateral Clearances to Fixed and Movable Obstructions:** A 2 foot minimum graded area with a 6:1 maximum cross slope (i.e., shoulder area) should be provided for clearance from lateral obstructions such as trees, poles, and bridge abutments measured from the edge of the pathway. The MUTCD requires a 2 foot minimum clearance to the sign face of post-mounted signs.

      If a barrier or rail is necessary, a minimum of 1 foot lateral offset from the edge of the path is desirable. Barriers terminating within 2 feet of the edge of the path should be marked with object markers. It is undesirable to place the pathway in a narrow corridor between 2 fences for long distances.

      A designer may want to consider that a typical ambulance width (including mirrors) is 11 feet.

      When minimum clearance cannot be achieved, refer to Section 12A-3 for protruding object requirements; refer to the *AASHTO Bike Guide* for mitigation measures, such as pavement markings, delineation, and signing.

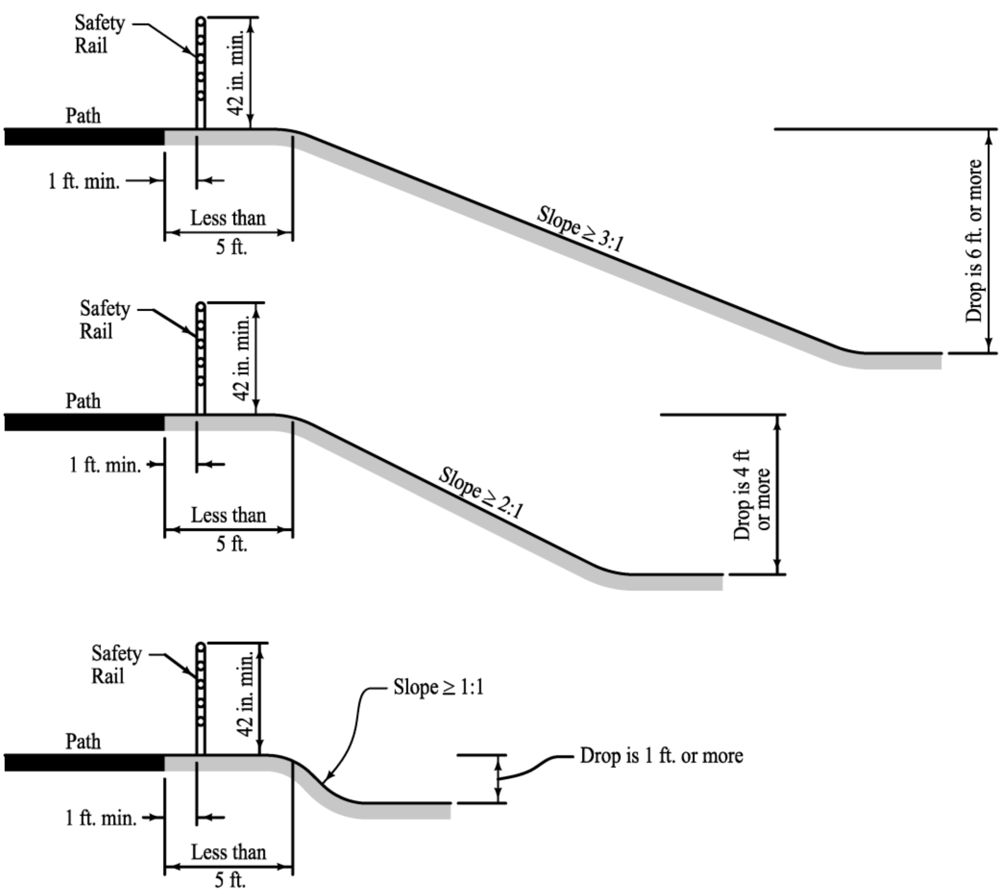
b. **Vertical Clearances to Overhead Obstructions:** The minimum vertical clearance is 10 feet. In some situations, such as tunnels and bridge underpasses, the vertical clearance should be greater than 10 feet in order to accommodate maintenance and emergency vehicles. In constrained areas, AASHTO allows the vertical clearance to obstructions to be a minimum of 8 feet. (AASHTO 5.2.1).

Refer to Section 12A-3 for legal requirements in low clearance situations.

6. **Shoulder Width and Slope:** The minimum graded shoulder width is 2 feet. The maximum shoulder area cross slope is 6:1.

7. **Safety Rail:** Safety rail should be a minimum of 42 inches in height. Provide safety rails at the outside of a structure. On steep fill embankment as described below, provide a safety rail or widen the shoulder area to 5 feet. (AASHTO 5.2.1)
   - Slopes 3:1 or steeper with a drop of 6 feet or greater.
   - Slopes 3:1 or steeper adjacent to a parallel body of water or other substantial obstacle.
   - Slopes 2:1 or steeper with a drop of 4 feet or greater.
   - Slopes 1:1 or steeper with a drop of 1 foot or greater.

Figure 12B-2.02: Safety Rail between Path and Adjacent Slope



See Iowa DOT Design Manual Section 11C-10 for guidance on safety rails.

Source: Adapted from *AASHTO Bike Guide* Exhibit 5.3

8. **Design Speed and Alignments:**

a. **Type 1:** Grades shall meet the requirements of Section 12A-2.

b. **Type 2:** Grades shall be less than or equal to 5% and all other Type 3 requirements should be met.

c. **Type 3:** There is no single design speed that is recommended for all paths. In general, a minimum design speed of 18 mph should be used, unless posted for slower speeds or in areas of steeper decline, in which case the design speed should be adjusted according to Table 12B-2.01. (AASHTO 5.2.4)

Table 12B-2.01: Minimum Design Speed and Horizontal Alignment

Terrain	Design Speed (mph)	Minimum Radius[1] (Horizontal Curve) (feet)
Grades less than 2%	18	60
Grades less than or equal to 5%	25	115
Grades 6% and more	30	166

[1] Based on 20 degree maximum lean angle

Source: *AASHTO Bike Guide* 5.2.4

The minimum radius of curvature negotiable by a bicycle can be calculated using the lean angle of the bicyclist or the superelevation and coefficient of friction of the shared use path. The minimum radii of curvature for a paved path are shown in Table 12B-2.02 based on lean angle of the cyclists.

Table 12B-2.02: Minimum Radii for Lean Angle of Cyclists

Design Speed (mph)	Minimum Radius (feet)
12	27
14	36
16	47
18	60
20	74
25	115
30	166

Source: *AASHTO Bike Guide* Exhibit 5.6

The minimum radii of curvature for a paved path based on superelevation should be calculated per the equations shown in the *AASHTO Bike Guide*. (AASHTO 5.2.2, 5.2.5, 5.2.6, and 5.2.8).

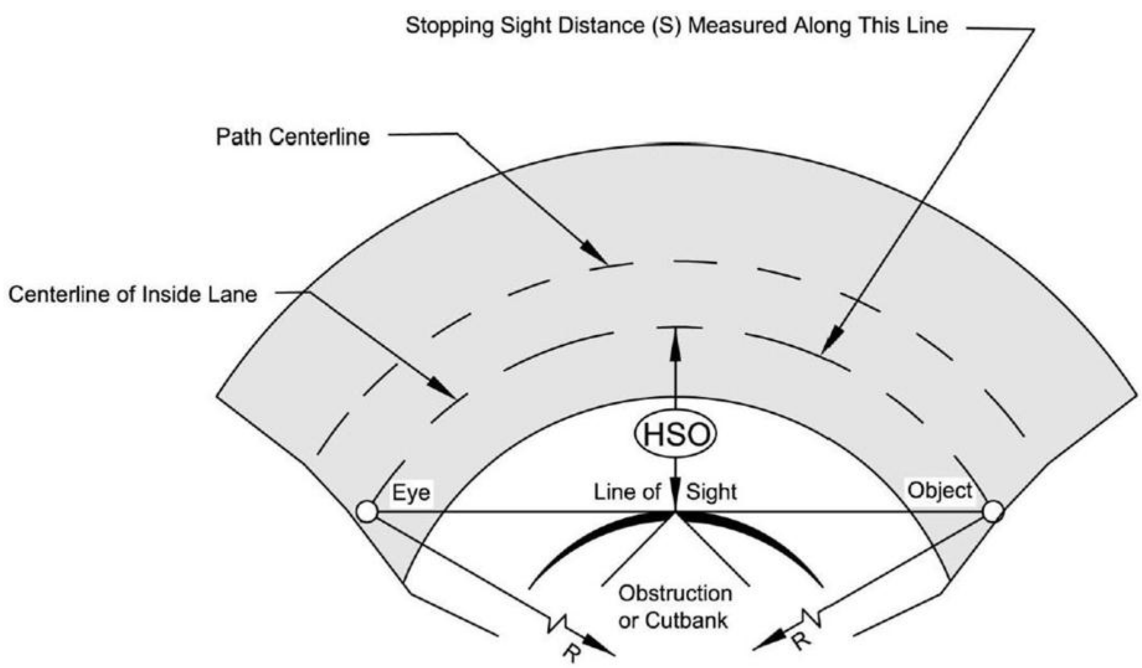
Table 12B-2.03 and Figure 12B-2.03 should be used to determine the minimum clearance necessary to avoid line-of-sight obstructions for horizontal curves. The lateral clearance (horizontal sight line offset or HSO) can be obtained from Table 12B-2.03, given the stopping sight distance from Equation 12B-2.01 and the proposed horizontal radius of curvature. Lateral clearances on horizontal curves should be calculated based on the sum of the stopping sight distances for both users traveling in opposite directions around the curve because bicyclists have a tendency to ride near the middle of narrow paths.

Table 12B-2.03: Minimum Lateral Clearance (Horizontal Sightline Offset or HSO) for Horizontal Curve

R	S= Stopping Sight Distance (ft)														
(ft)	20	40	60	80	100	120	140	160	180	200	220	240	260	280	300
25	2.0	7.6	15.9												
50	1.0	3.9	8.7	15.2	23.0	31.9	41.5								
75	0.7	2.7	5.9	10.4	16.1	22.8	30.4	38.8	47.8	57.4	67.2				
95	0.5	2.1	4.7	8.3	12.9	18.3	24.7	31.8	39.5	48.0	56.9	66.3	75.9	85.8	
125	0.4	1.6	3.6	6.3	9.9	14.1	19.1	24.7	31.0	37.9	45.4	53.3	61.7	70.6	79.7
155	0.3	1.3	2.9	5.1	8.0	11.5	15.5	20.2	25.4	31.2	37.4	44.2	51.4	59.1	67.1
175	0.3	1.1	2.6	4.6	7.1	10.2	13.8	18.0	22.6	27.8	33.5	39.6	46.1	53.1	60.5
200	0.3	1.0	2.2	4.0	6.2	8.9	12.1	15.8	19.9	24.5	29.5	34.9	40.8	47.0	53.7
225	0.2	0.9	2.0	3.5	5.5	8.0	10.8	14.1	17.8	21.9	26.4	31.3	36.5	42.2	48.2
250	0.2	0.8	1.8	3.2	5.0	7.2	9.7	12.7	16.0	19.7	23.8	28.3	33.1	38.2	43.7
275	0.2	0.7	1.6	2.9	4.5	6.5	8.9	11.6	14.6	18.0	21.7	25.8	30.2	34.9	39.9
300	0.2	0.7	1.5	2.7	4.2	6.0	8.1	10.6	13.4	16.5	19.9	23.7	27.7	32.1	36.7
350	0.1	0.6	1.3	2.3	3.6	5.1	7.0	9.1	11.5	14.2	17.1	20.4	23.9	27.6	31.7
390	0.1	0.5	1.2	2.1	3.2	4.6	6.3	8.2	10.3	12.8	15.4	18.3	21.5	24.9	28.5
500	0.1	0.4	0.9	1.6	2.5	3.6	4.9	6.4	8.1	10.0	12.1	14.3	16.8	19.5	22.3
565		0.4	0.8	1.4	2.2	3.2	4.3	5.7	7.2	8.8	10.7	12.7	14.9	17.3	19.8
600		0.3	0.8	1.3	2.1	3.0	4.1	5.3	6.7	8.3	10.1	12.0	14.0	16.3	18.7
700		0.3	0.6	1.1	1.8	2.6	3.5	4.6	5.8	7.1	8.6	10.3	12.0	14.0	16.0
800		0.3	0.6	1.0	1.6	2.2	3.1	4.0	5.1	6.2	7.6	9.0	10.5	12.2	14.0
900		0.2	0.5	0.9	1.4	2.0	2.7	3.6	4.5	5.6	6.7	8.0	9.4	10.9	12.5
1000		0.2	0.5	0.8	1.3	1.8	2.4	3.2	4.0	5.0	6.0	7.2	8.4	9.8	11.2

Source: *AASHTO Bike Guide* Exhibit 5.10

Figure 12B-2.03: Components for Determining Horizontal Sight Distance



Source: *AASHTO Bike Guide* Exhibit 5.9

For vertical alignment, use the preferred maximum segment length shown in Table 12B-2.04 whenever possible. Using the acceptable and allowed criteria should only be done when the engineer considers the ability of the users. For example, long rural segments would generally serve more physically capable users who have selected the path and could navigate the steeper grades over longer lengths.

Table 12B-2.04: Vertical Alignment

Grade Range	Maximum Segment Length (feet)		
	Preferred	***Acceptable***[1]	***Allowed***[2]
< 5%	Any length	Any Length	Any Length
≥ 5% and < 8.33%	--	50	200
≥ 8.33% and < 10%	--	30	30
≥ 10% and < 12.50%	--	--	10

[1] Derived from AGODA Section 1016 (Outdoor Recreation Access Routes)

[2] Derived from AGODA Section 1017 (Trails)

The minimum length of vertical curve needed to provide minimum stopping sight distance at various speeds on crest vertical curves is presented in Table 12B-2.05. The eye height of the typical adult bicyclist is assumed to be 4.5 feet. For stopping sight distance calculations the object height is assumed to be 0 inches. (AASHTO 5.2.7). Equation 12B-2.01 can also be used to determine the minimum length of crest vertical curve necessary to provide adequate sight distance.

Equation 12B-2.01

$$S > L \qquad L = 2S - \frac{200(\sqrt{h_1}+\sqrt{h_2})^2}{A}$$

$$L > S \qquad L = \frac{AS^2}{100(\sqrt{2h_1}+\sqrt{2h_2})^2}$$

where:

L= Minimum length of vertical curve (ft)
A = Algebraic grade difference (percent)
S = Stopping sight distance (ft)
h_1 = Eye height (4.5 feet for a typical bicyclist)
h_2 = Object height (0 ft)

Table 12B-2.05: Minimum Length of Crest Vertical Curve Based on Stopping Sight Distance

A	S=Stopping Sight Distance (ft)														
(%)	20	40	60	80	100	120	140	160	180	200	220	240	260	280	300
2												30	70	110	150
3								20	60	100	140	180	220	260	300
4						15	55	95	135	175	215	256	300	348	400
5					20	60	100	140	180	222	269	320	376	436	500
6				10	50	90	130	170	210	267	323	384	451	523	600
7				31	71	111	151	191	231	311	376	448	526	610	700
8			8	48	88	128	168	208	248	356	430	512	601	697	800
9			20	60	100	140	180	220	260	400	484	576	676	784	900
10			30	70	110	150	190	230	270	444	538	640	751	871	1000
11			38	78	118	158	198	238	278	489	592	704	826	958	1100
12		5	45	85	125	165	205	245	285	533	645	768	901	1045	1200
13		11	51	91	131	171	211	251	291	578	699	832	976	1132	1300
14		16	56	96	136	176	216	256	296	622	753	896	1052	1220	1400
15		20	60	100	140	180	220	260	300	667	807	960	1127	1307	1500
16		24	64	104	144	184	224	264	304	711	860	1024	1202	1394	1600
17		27	67	107	147	187	227	267	307	756	914	1088	1277	1481	1700
18		30	70	110	150	190	230	270	310	800	968	1152	1352	1568	1800
19		33	73	113	153	193	233	273	313	844	1022	1216	1427	1655	1900
20		35	75	115	155	195	235	275	315	889	1076	1280	1502	1742	2000
21		37	77	117	157	197	237	277	317	933	1129	1344	1577	1829	2100
22		39	79	119	159	199	239	279	319	978	1183	1408	1652	1916	2200
23		41	81	121	161	201	241	281	321	1022	1237	1472	1728	2004	2300
24	3	43	83	123	163	203	243	283	323	1067	1291	1536	1803	2091	2400
25	4	44	84	124	164	204	244	284	324	1111	1344	1600	1878	2178	2500

The line between the shaded and un-shaded portions of the table shows when the stopping sight distance is equal to the length of the crest vertical curve.

Source: *AASHTO Bike Guide* Exhibit 5.8

9. **Stopping Sight Distance:** Shared use paths must be designed with adequate stopping sight distance along the entire path to provide users with the opportunity to see and react to unexpected conditions. The distance needed to bring a path user to a fully controlled stop is a function of the user's perception and braking reaction time, the initial speed, the coefficient of friction between the wheels and the pavement, the braking ability of the user's equipment, and the grade. Minimum stopping sight distances can be determined using Equation 12B-2.02. Stopping sight distance must be provided along the entire length of the pathway and should be checked at all horizontal and vertical curves. (AASHTO 5.2.8).

$$S = \frac{V^2}{30(f \pm G)} + 3.67V$$

Equation 12B-2.02

where:

S = Stopping sight distance (ft)
V = Velocity (mph)
f = Coefficient of friction (use 0.16 for a typical bicycle)
G = Grade (ft/ft) (rise/run)

10. **Accessibility Requirements:** For construction of curb ramps and placement of detectable warnings, see Section 12A-2 to ensure ADA compliance.

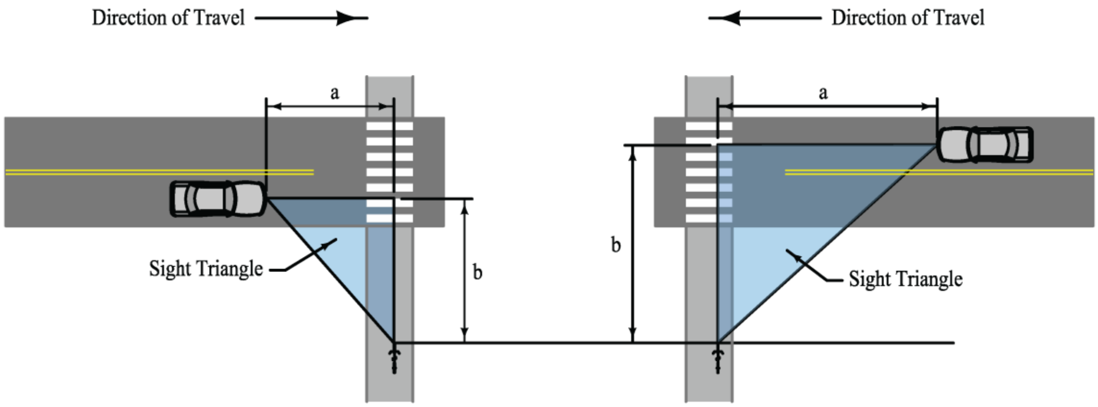
D. Intersection Sight Distance

1. **General:** Intersection sight distance is a fundamental component in the selection of appropriate control at a midblock path-roadway intersection. The least restrictive control that is effective should be used. The line of sight is considered to be 2.3 feet above the path surface.

   Roadway approach sight distance and departure sight triangles should be calculated using motor vehicles, which will control the design criteria. (AASHTO 5.3).

2. **Approach Sight Distance:** Pathway approach sight distance should be determined by the fastest path user, typically the adult bicyclist. If yield control is to be used for either the roadway approach or the path approach, available sight distance adequate for a traveler on the yield controlled approach to slow, stop, and avoid a traveler on the other approach is required. The roadway leg (a) of the sight triangle is based on the ability of a bicyclist to reach and cross the roadway if they do not see a conflict (see Figure 12B-2.04). Similarly, the path leg (b) of the sight triangle is based on the ability of a motorist to reach and cross the junction if they do not see a conflict (see Figure 12B-2.04). If sufficient sight distance is unable to be provided by the yield sight triangle described above, more restrictive control should be implemented.

Figure 12B-2.04: Yield Sight Triangles



Source: Adapted from *AASHTO Bike Guide* Exhibit 5.15

$$a = 1.47_{Road}\left(\frac{S}{147V_{Path}} + \frac{w + L_a}{1.47V_{Path}}\right)$$

Equation 12B-2.03
Length of Roadway Leg of Sight Triangle

$$b = V_{Path}\left(\frac{1.47V_e - 1.47V_b}{a_i} + \frac{w + L_a}{0.88V_{Road}}\right)$$

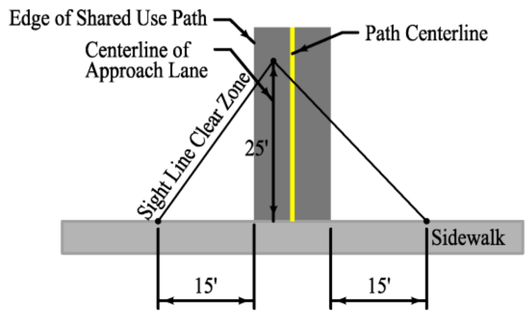
Equation 12B-2.04
Length of Path Leg of Sight Triangle

where:

a = Length of leg of sight triangle along the roadway approach (ft)
b = Length of leg of sight triangle along the path approach (ft)
w = Width of the intersection to be crossed (ft)
L_a = Design vehicle length
For Equation 12B-2.03: Typical bicycle length = 6 ft
For Equation 12B-2.04: Design vehicle length (ft)
V_{Path} = Design speed of the path (mph)
V_{Road} = Design speed of the road (mph)
S = Stopping sight distance for the path user traveling at design speed
V_e = Speed at which the motorist would enter the intersection after decelerating (mph) (assumed 0.60 x road design speed)
V_b = Speed at which braking by the motorist begins (mph) (same as road design speed)
a_i = motorist deceleration rate (ft/s^2) on intersection approach when braking to a stop is not initiated (assume -5.0 ft/s^2)

3. **Path-Sidewalk Intersection:** At an intersection of a shared use path and a sidewalk, a clear sight triangle extending at minimum 15 feet along the sidewalk must be provided. Refer to Figure 12B-2.05. If two shared use paths intersect, the same process for the roadway-path intersection should be used.

Figure 12B-2.05: Minimum Path-Sidewalk Sight Triangle



Source: Adapted from *AASHTO Bike Guide* Exhibit 5.16

E. Surface

It is important to construct and maintain a smooth riding surface on shared use paths. Shared use path pavements should be machine placed. Surface texture is needed but care must be exercised not to cause operational problems with too little or too much texture. Broom finish or burlap drag concrete surfaces are preferred over trowel finishes. Joints shall be sawed, not hand tooled.

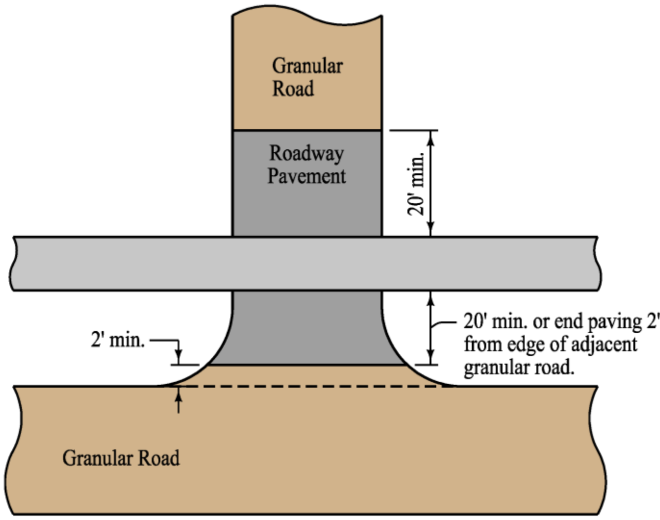
1. **Type 1 and Type 2:** Type 1 and Type 2 shared use paths shall be paved.

2. **Type 3:** Hard, all-weather pavement surfaces are preferred to unpaved surfaces due to the higher service quality and lower maintenance. Type 3 shared use paths should be paved; however, a granular surface may be allowed. If a granular surface is used, it must be maintained to be firm, stable, and slip resistant.

F. Crossings at Unpaved Surfaces

When crossing an unpaved roadway, alley, or driveway, a minimum of 20 feet in addition to the path width should be paved on each side of the path to reduce the amount of gravel tracked onto the path. If edge of parallel unpaved roadway is less than 20 feet from the closest edge of the path, only pave to within 2 foot of edge of the parallel unpaved roadway. The thickness of the path and adjacent roadway paving should be designed to accommodate vehicular traffic and meet the requirements of the agency responsible for the roadway.

Figure 12B-2.06: Crossing at Unpaved Surface



G. At-grade Railroad Crossing

Whenever it is necessary to cross railroad tracks with a bicycle, special care must be taken. The crossing should be at least as wide as the approaches of the shared use path. Whenever possible, the crossing should be straight and between 90 and 60 degrees to the rails. The greater the crossing angle deviates from being perpendicular, the greater the chance that a bicyclist's front wheel may be trapped in the flangeway causing a loss of control. (AASHTO 4.12).

H. Drainage

Drainage structures underneath paths should typically be designed to the same design year storm as the roadway drainage structures. When a Type 3 shared use path is built on a berm, consider the drainage needs of that path. For shared use paths constructed on slopes, drainage design should take into account control of the runoff from the slope. For higher flows it may be necessary to develop parallel ditches and culverts under the path. Drainage designs should also provide for low flows and seepage from the slope. Due to the potential for accidents from buildup of algae from low flows and side hill seepage, the need for subdrains or other treatments on the high side of the path should be evaluated.

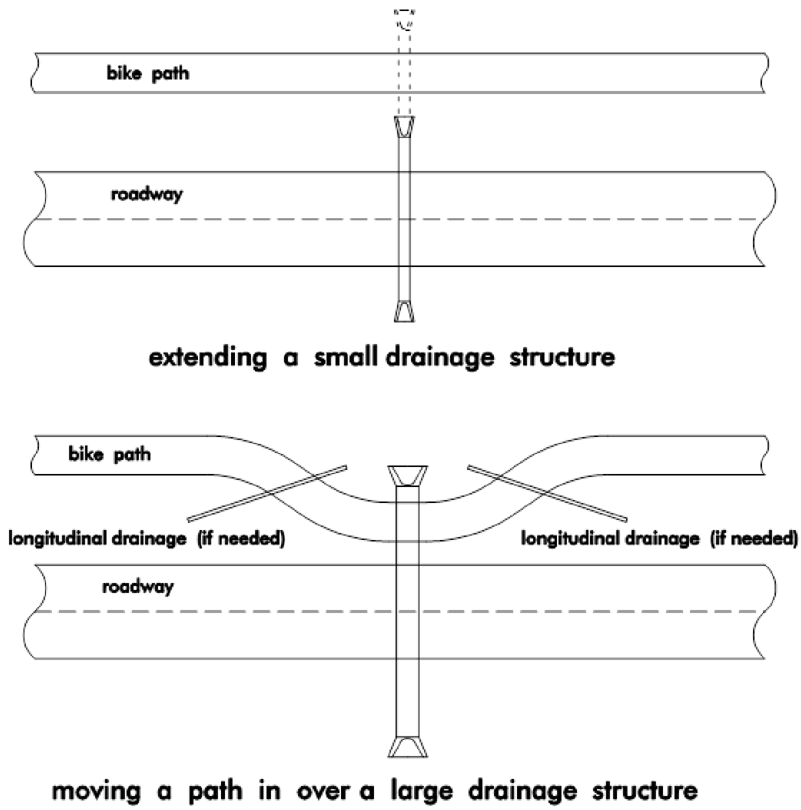
1. **Urban Areas:** The minimum recommended pavement cross slope of 1% usually provides enough slope for proper drainage. Sloping in one direction, usually toward the street, instead of crowning is preferred and usually simplifies the drainage and surface construction. However, care must be exercised not to trap water on the high side of the shared use path, particularly in curved areas. (AASHTO 5.2.11).

2. **Rural Areas:** The best way to accomplish drainage underneath a shared use path is by extending smaller structures under the path or moving the path closer to the roadway to cross larger structures, see Figure 12B-2.07.

   For paths placed on the backslope, smaller drainage structures (normally pipes less than 60 inches and box culverts less than 5 feet by 4 feet) should be extended through the path. For larger culverts, the path should be moved in to cross the structure and then moved back out to the backslope. If this is done, longitudinal drainage will have to be provided where the path crosses the ditch. Depending upon how close the path comes to culvert openings, safety railing may be needed on the culverts.

For paths on the foreslope, culverts should be extended as necessary.

Figure 12B-2.07: Accommodating Drainage Structures



I. Structure Design

The minimum width for a shared use path on a new roadway bridge, widened roadway bridge, or separate pedestrian structure is 10 feet. Through conversations with the Iowa Bicycle Coalition, this was determined to be adequate width in most situations. If heavy use is anticipated, such as near a school, a 12 or 14 foot wide path should be used. If a separate shared use path structure is to be constructed, it should have a 5% maximum running grade.

If widening a bridge or building a new structure is beyond the scope of a project, it may be possible to use an existing sidewalk as a path. The path should be separated from vehicular traffic with a barrier. Signage may be necessary instructing cyclists to dismount before crossing the bridge. For Iowa DOT administered projects, the designer should contact the Office of Design and the Office of Traffic and Safety for further assistance if considering a narrowed path across a bridge.

J. Pavement Markings

Ladder or zebra pavement markings per MUTCD are recommended at crosswalks. Other pavement markings are not required, except as mitigation strategies. (AASHTO 5.4).

K. Signing

All signs should be retroreflective and conform to the color, legend, and shape requirements described in the MUTCD. In addition, guide signing, such as to indicate directions, destinations, distances, route numbers, and names of crossing streets should be used. In general, uniform application of traffic control devices, as described in the MUTCD, should be used and will tend to encourage proper bicyclist behavior. (AASHTO 5.4).

L. Lighting

Fixed-source lighting reduces conflicts along shared use paths and at intersections. In addition, lighting allows the bicyclist to see the shared use path direction, surface conditions, and obstacles. Lighting for paths is important and may be considered where heavy nighttime riding is expected (e.g., paths serving college students or commuters) and at roadway intersections. Lighting should be considered through underpasses or tunnels and when nighttime security could be a problem. Where special security problems exist, higher illumination levels may be considered. Light standards (poles) should meet the recommended horizontal and vertical clearances. (AASHTO 5.2.12).

On-Street Bicycle Facilities

A. General

Cyclists have similar access and mobility needs as other transportation users. However, cyclists must use their own strength and energy to propel the bicycle, thus a bicyclist is generally slower than other vehicles that are operating on the roadway. Additionally, cyclists are more vulnerable to injury during a crash and are of any age group. With these factors in mind, it is imperative that designing bicycle facilities is done with great care.

The 2012 draft of the “AASHTO Guide for the Development of Bicycle Facilities” (or *AASHTO Bike Guide*) was used as a reference for developing this section. References made to the *AASHTO Bike Guide* within this section are shown in parentheses, e.g. (AASHTO 4.2).

B. Elements of Design

Since cyclists usually have a higher eye height and are slower than the adjacent traffic, the roadway design elements for motor vehicles usually meet or exceed the minimum design elements required for cyclists.

Surface conditions affect cyclists more significantly than motor vehicles. Therefore, when establishing bicycle lanes and routes, it is important that the roadway surface is in good condition and is free of potholes, bumps, cracks, loose gravel, etc. If the roadway is not in good bicycle riding condition, it should be repaired either with resurfacing or reconstruction. Chip-sealed surfaces prove to create difficult riding conditions. (AASHTO 4.2).

C. Facilities

Except where prohibited, bicycles may be operated on all roadways. The following are the different types of bicycle facilities that are located on the roadway along with their design criteria.

1. **Shared Lanes:** Shared lanes already exist on local neighborhoods and city streets. However, these lanes can include design features that will make the lanes more bicycle friendly. This includes good pavement quality, adequate sight distance, lower speeds, bicycle-compatible drainage grates, bridge expansion joints, railroad crossings, etc. (AASHTO 4.3).

   a. **Major Roads (Wide Curb/Outside Lanes):** Lane widths should be 13 to 15 feet wide with 14 foot lanes as preferred. Lane widths of 14 feet and greater allow motorists to pass cyclists without encroaching into adjacent lanes; however, it is important to note that 15 foot lanes should be used only on appropriate sections with steep grades or sections where drainage grates, raised delineators, or on-street parking effectively reduces the usable width. The gutter should not be included in the measurement as usable width. Lanes 15 feet or wider could encourage faster vehicular movements or even two vehicles operating side by side in one lane. (AASHTO 4.3.1).

b. **Marked Shared Lanes:** In areas that need to provide enhanced guidance for cyclists, shared lanes may be marked with pavement marking symbols. This marking should be provided in locations where there are insufficient widths to provide bicycle lanes or shared use paths. This pavement marking not only lets the cyclists know where to be located within the lane but also the direction of travel.

   Shared lane markings are not appropriate for paved shoulders or bicycle lanes, and should not be used on roadways that have a speed limit above 35 mph. Markings should be placed immediately after an intersection and spaced not greater than 250 foot intervals. Refer to both the MUTCD and AASHTO 4.4.

c. **Signs for Shared Roadways:** Along with pavement markings, signage is a very useful tool to communicate and inform both motorists and cyclists about shared roadways. It is important to note, that signs shall be used only when needed in order to prevent confusion, reduce clutter, and improve visibility. Refer to both the MUTCD and AASHTO 4.3.2.

Figure 12B-3.01: Share the Road Sign Assembly

Source: *AASHTO Bike Guide* Exhibit 4.1

2. **Paved Shoulders:** For higher speed and higher traffic roadways, adding or improving a paved shoulder can greatly improve cyclist accommodations on roadways. This will not only benefit the cyclists and motorists by giving the cyclists a place to ride that is located outside of the travel lane, but it also can extend the service life of roads by reducing edge deterioration.

   It is important to note that paved shoulders should not be confused with bicycle lanes, as bicycle lanes are travel lanes and paved shoulders are not. Paved shoulders should have a minimum width of 4 feet wide with a preferred width of 5 feet. Also, they should be at least 5 feet in locations of guardrails, curbs, or other roadside barriers. Additionally, the width may be increased in areas where the speeds exceed 50 mph, areas of heavy truck traffic, or locations with static obstruction exist at the right side of the roadway.

   It is preferred to have paved shoulders on both sides of a two-way roadway; however, in constrained locations and where pavement widths are limited, it may be preferable to provide a wider shoulder on one side of the roadway and a narrower shoulder on the other. This may be beneficial in uphill roadway sections to provide slow-moving cyclists additional maneuvering space and sections with vertical or horizontal curves that limit sight distance over crests and on the inside of horizontal curves.

In locations where unpaved driveways or roadways meet a paved shoulder, it is recommended to pave at least 10 feet of the driveway and 20 feet or to the right-of-way line, whichever is less, of the unpaved public road. This will help minimize loose gravel from spilling onto the travel way and affecting the cyclists. Additionally, raised pavement markers should not be used, unless they are beveled or have tapered edges.

Rumble strips may be used on paved shoulders that include the bicycle traffic; however, the minimum clear path should be 4 feet from the rumble strip to the outside edge of paved shoulder or 5 feet to the adjacent curb or other obstacle. Gaps at a minimum of 12 feet and a recommended distance of 40 to 60 feet for the rumble strips should also be provided in order to allow room for cyclists to leave or enter the shoulder without crossing the rumble strip. (AASHTO 4.5). Rumble strips should have the following design:

- Width: 5 inches
- Depth: 0.375 inches
- Spacing: 11 to 12 inches (may be reduced to 6 inches)

3. **Bicycle Lanes:** Bicycle lanes are a portion of the roadway that is designated for bicycle traffic. They are one-way facilities that typically carry bicycle traffic in the same direction as the adjacent motor vehicle traffic. They are appropriate and preferred on corridors located in both urban and suburban areas; however, they may be used on rural roadways. Paved shoulders can be designated as bicycle lanes by installing bicycle lane symbol markings, yet marked shoulders will still need to meet the criteria listed herein.

Bicycle lanes should have a smooth surface with utility and grate covers flush with the surface of the lane. Additionally, bicycle lanes should be free of ponding water, washouts, debris accumulation, and other potential hazards. (AASHTO 4.6).

a. **Two-way Streets:** It is recommended that bicycle lanes are provided on both sides of two-way streets as bicycle lanes on only one side may encourage wrong-way use. The exceptions are in cases of long downhill grades where bicyclists’ speeds are similar to typical motor vehicle speeds. In this case, shared lane markings may be used in the downhill direction and a bicycle lane in the uphill direction.

b. **One-way Streets:** On one-way streets, the bicycle lane should be on the right-hand side of the roadway. A bicycle lane may be placed on the left side of the roadway if there are a significant number of left turn lanes, or if left-sided bicycle lanes will reduce conflicts with bus traffic, on-street parking, and/or heavy right-turn movements, etc.

Bicycle lanes should also be provided on both streets of a one-way couplet as to provide a more complete network and discourage wrong-way riding. If width constraints are in effect, shared lane markings should be considered.

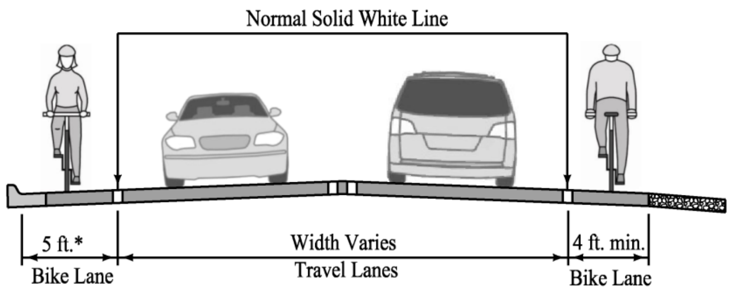
In some designated one-way streets, it may be preferred to provide bicyclists a contra-flow bicycle lane using markings and separated by a double yellow centerline. This design should be used where there are few intersecting driveways, alleys, and streets on the side of the street with the contra-flow lane. (AASHTO 4.6.3).

c. **Lane Widths:** The preferred operating width for bicycle lanes is 5 feet; however, 4 feet is the minimum in locations where there is an absence of on-street parking and a curb and gutter. In some instances, wider lanes may be more desirable. These instances are:

- In locations with narrow parking lanes and high turnover. A wider bicycle lane of 6 to 7 feet will allow cyclists to ride out of the area of opening vehicle doors.
- In areas with high bicycle use. A bicycle lane width of 6 to 8 feet will allow cyclist to pass each other or ride side-by-side.
- In high-speed and high-volume roadways and/or high heavy vehicle traffic. A wider lane will provide an additional separation between cyclists and motorist, thus increasing safety and comfort of the cyclists.

With wider bicycle lanes, appropriate signage and markings shall be used to delineate the bicycle lanes from the vehicle lanes.

Figure 12B-3.02: Typical Bicycle Lane Cross-sections - Parking Prohibited

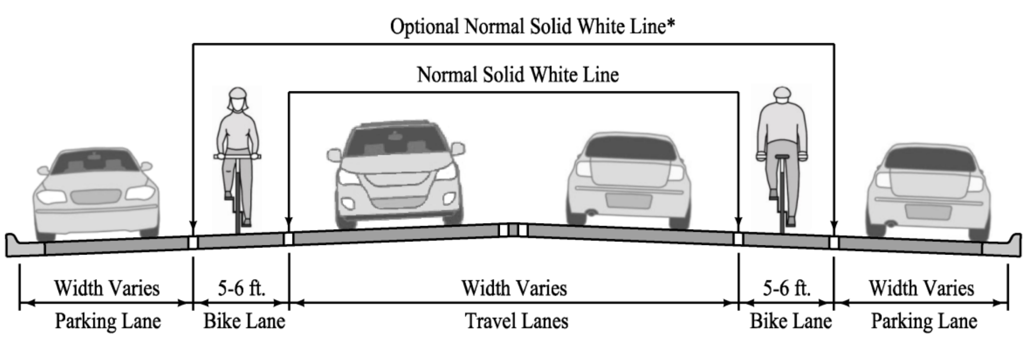


* On extremely constrained, low-speed roadways with curbs but no gutter, where the preferred bike lane width cannot be achieved despite narrowing all other travel lanes to their minimum widths, a 4 foot wide bike lane can be used.

Source: Adapted from *AASHTO Bike Guide* Exhibit 4.13

d. Bicycle Lanes and On-street Parking: With on-street parking facilities, bicycle lanes shall be located between the vehicle travel lane and the parking spot. For parallel on-street parking, the recommended width of a marked parking lane is 8 feet with a minimum of 7 feet. When the parking lane is not marked, the recommended width of the shared bicycle and parking lane is 13 feet with a 12 foot minimum. Any on-street diagonal parking that is adjacent to bicycle lanes shall be back-in parking as to prevent accidents due to poor visibility of bicyclists. (AASHTO 4.6.5).

Figure 12B-3.03: Typical Bicycle Lane Cross-sections - On-street Parking

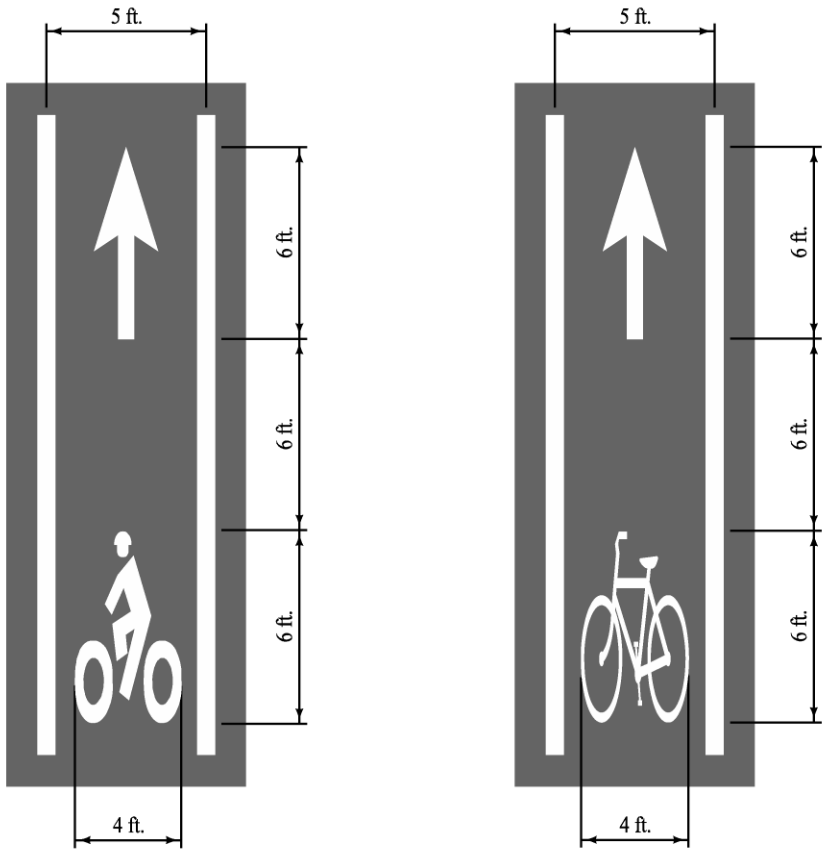


* The optional normal (4 to 6 inch) solid white line may be helpful even when no stalls are marked (because parking is light), to make the presence of a bicycle lane more evident. Parking stall markings may also be used.

Source: Adapted from *AASHTO Bike Guide* Exhibit 4.13

e. Signs and Markings: Bicycle lanes are designated for preferential use by bicyclists with a normal white line (4 to 6 inches wide) and one of the two standard bicycle lane symbols, which may be supplemented with a directional arrow marking. Pavement signs and non-raised pavement markings should be used instead of curbs, posts, raised pavement markings, or barriers. Raised devices are hazardous to cyclists and make it more difficult for cyclists to maintain riding in the bicycle lane. Refer to both the MUTCD and AASHTO 4.7.

Figure 12B-3.04: Bicycle Lane Symbol Markings

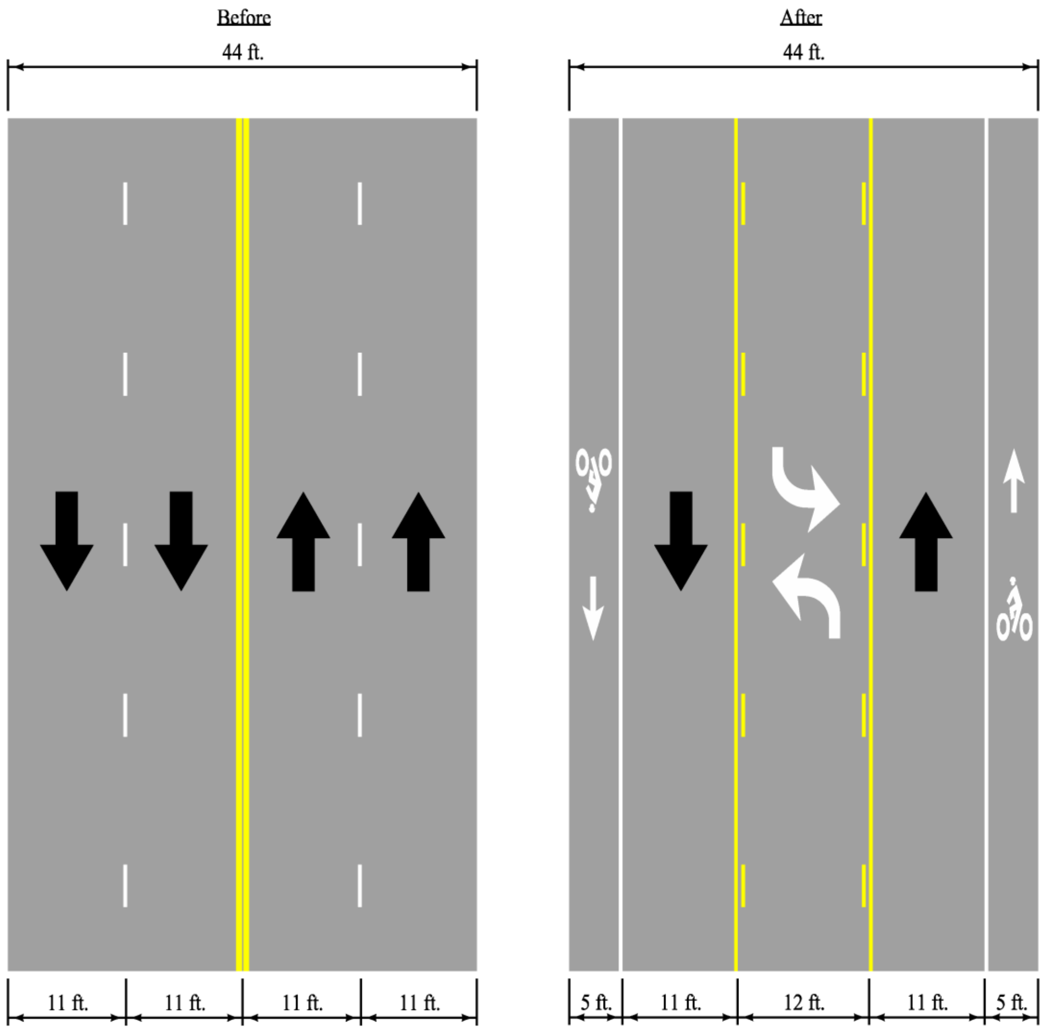


Source: Adapted from *AASHTO Bike Guide* Exhibit 4.17

f. Intersection Design: Most conflicts between motor vehicles and bicyclist occur at intersections and driveways. Due to the vulnerability of cyclists as well as the low visibility the cyclists have in relationship to the motorists, good intersection bicycle lane design and intersection pavement marking design is crucial to the success of an intersection that incorporates bicycle lanes. Refer to both the MUTCD and AASHTO 4.8 for additional information pertaining to intersection pavement marking and bicycle lane design.

4. Retrofitting Bicycle Facilities on Existing Roadways: Existing streets and highways may be retrofitted to improve bicycle accommodations by either reconfiguring the travel lanes to accommodate bicycle lanes or by widening the roadway to accommodate bicycle lanes or paved shoulders. These retrofits are best accomplished as either a reconstruction project or a repaving project as these projects will eliminate traces of old pavement markings. (AASHTO 4.9).

Figure 12B-3.05: Example of Road Diet



* Dimensions are illustrative

Source: Adapted from *AASHTO Bike Guide* Exhibit 4.23

5. **Bicycle Boulevards:** A bicycle boulevard is described as a local street or a series of contiguous street segments that have been modified to function as a through street for cyclists while discouraging through vehicle traffic. To be effective, bicycle boulevards should be long enough to provide continuity over a distance of between 2 and 5 miles.

   Due to the low traffic volumes and speeds, local streets naturally create a bicycle-friendly environment in which the cyclists share the roadway with the vehicles. However, many local streets are not continuous enough for long bicycle routes. Therefore, in order to create a bicycle boulevard, some short sections of paths or segments may need to be constructed between local streets in order to create the continuous route.

Some design elements that are involved in the design of bicycle boulevards are:

- Traffic diverters at key intersections that allow bicycle through traffic but reduce or deny vehicle traffic
- Two-way stop-controlled intersection that give the bicycle boulevard priority
- Neighborhood traffic circles or mini-roundabouts
- Traffic-calming features
- Wayfinding signs to guide bicyclists
- Shared lane markings where appropriate
- Bicycle-sensitive traffic signals at busy intersections
- Median refuges large enough for bicycles
- Curb extensions on crossed thoroughfare with on-street parking

It is important to note that before the design of a bicycle boulevards, an investigation of the proposed boulevards should be performed since many of the design elements listed may already be in use. (AASHTO 4.10).

D. Bicycle Guide Signs

Guide signs are an important element to all bicycle facilities as they help cyclists navigate to their destination. There are many guidelines and standards that go along with the type and placement of guide signs. See both the MUTCD and AASHTO 4.11.

E. Railroad Crossings for Bicycles

Where roadways or shared use paths cross railroad tracks on a diagonal, the designer should take care in the design of the crossing as to prevent steering difficulties for the cyclists. This includes:

- Increasing the skew angle between the tracks and the bike path to 60 degrees or greater so bicyclists can avoid catching their wheels in the flange of the tracks. This can be accomplished with reverse curves or with a widened shoulder.
- Creating a smooth crossing surface that will last over time and not be slippery when wet.
- Minimizing flange openings as much as possible. Under special rail conditions, rubber fillers products may be used. Contact the railroad company for approval prior to the design and installation of the fillers.

See both the MUTCD and AASHTO 4.12.1.

F. Obstruction Markings for Bicycle Lanes

The design of bicycle facilities should avoid obstruction and barriers as much as possible. However, in rare circumstance in which an obstruction or barrier cannot be avoided, signs, reflectors, and markings should be utilized to alert they cyclists. (AASHTO 4.12.2).

G. Traffic Signals for Bicycles

Traffic signals have traditionally been designed based off the operating characteristics of motor vehicles. However, at intersections with medium to high bicycle usage that incorporates shared lanes or bicycle lanes, traffic signal designers should include the characteristics of bicyclists to their traffic signals. The signal parameters that could be modified to accommodate bicyclists when appropriate are minimum green interval, all-red interval, and extension time. This information can be found in AASHTO 4.12.3 and 4.12.4 as well as the latest edition of the "Highway Capacity Manual."

H. Bridges and Viaducts for Bicycles

Two considerations should be taken into account before the design of bicycle accommodations with bridges - the length of the bridge and the design of the approach roadway. If the bridge approach does not include bicycle accommodations, the bridge can still facilitate use by bicyclists by including a wide shoulder or bicycle lanes and include paved shoulder, shared lanes, or shared use path as part of the bridge project. Additionally, if the bridge is continuous and spans over a 1/2 mile in length with speed of excess of 45 mph, a concrete barrier separated shared use path on both sides of the bridge should be considered. By allowing paths on both sides of the bridge, wrong-way travel of the cyclists will be deterred. (AASHTO 4.12.5).

I. Traffic Calming and Management of Bicycles

There are many things that a designer can do to reduce the traffic speed of cyclists and to manage bicycles effectively. These things include narrowing streets to create a sense of enclosure; adding vertical deflections such as speed humps, speed tables, speed cushions, and raised sidewalks; adding curb extension or chokers; adding chicanes; installing traffic circles; and incorporating multi-way stops. (AASHTO 4.12.6 and 4.12.7).

J. Intake Grates and Manhole Castings for Bicycle Travel

It is important to have intake grate openings run perpendicular to the direction of travel as this will prevent bicycle wheels from dropping into the gaps and causing crashes. SUDAS Specifications Figure/Iowa DOT Standard Road Plan SW-603, Type R and Type S, are intake grates that are appropriate for use on bicycle routes. Where it is not immediately feasible to replace existing grates, metal straps can be welded across slots perpendicular to the direction of travel at a maximum longitudinal spacing of 4 inches. Additionally, open-throat intakes can be used instead of grate intakes in order to eliminate the grate all together. The presence of the depressed throat of the intake should be taken into account.

Surface grates and manhole castings should be flush with the roadway surface. In the case of overlays, the grates and castings should be raised to within 1/4 inch of the new surface. If this is not possible or practical, the pavement must taper into drainage inlets so it does not have an abrupt edge at the inlet. Take care in the design of the taper of the pavement around inlets and castings so to avoid “birdbaths” or low spots that are not drainable in the pavement. (AASHTO 4.12.8).

K. Bicycles at Interchanges

When designing bicycle facilities at interchanges, it is important to consider both safety and convenience for the cyclists. This is best achieved by designing right-angle intersection or single lane roundabouts at the intersection between the local route and the ramps. These designs promote low speeds, minimize conflict areas, and increase visibility. Additionally, stop signs or signals are encouraged for motorists turning from the off ramp to the local route rather than allowing a free-flowing movement as this will increase the safety of the cyclists.

At complex interchanges that include high-speeds and free-flowing motor vehicle movements, a well signed and clearly directed grade-separated crossings may be necessary. These grade-separated facilities should still include good visibility, be convenient, and consist of adequate lighting. (AASHTO 4.12.9).

L. Bicycles at Roundabouts

In designing roundabouts for bicycle usage, single lane roundabouts are safer and easier to navigate for cyclists. Multi-lane roundabouts include too many conflict points due to bicycle weaving/changing lanes and motorist cutting off cyclists when exiting the roundabout.

In instances of bicycle lanes approaching a roundabout, the bicycle lane should be terminated at least 100 feet from the edge of the entry curve of the roundabout and prior to the crosswalk. Also, prior to the roundabout and after the termination of the bicycle lane, a tapering of the bicycle lane to the travel lane should be provided. This is done to achieve the appropriate entry width for the roundabout and the taper should be 7:1 for a 20 mph design speed or 40 feet for a 5 to 6 foot bicycle lane. Additionally, the bicycle lane line should be dotted 50 to 200 feet in advance of the taper to encourage cyclists to merge into traffic.

In rare circumstances, bicyclists should be given the option to merge with traffic prior to the roundabout or exit onto the adjacent sidewalk via a ramp. These instances include multi-lane roundabouts, high design speed roundabouts, and/or complex roundabouts. However, in some jurisdictions, cyclists riding on sidewalks may be prohibited. In designing bicycle ramps prior to a roundabout, the following criteria should be followed:

- Place bicycle ramps at the end of the full width bicycle lane and just before the taper of the bicycle lane.
- Where no bicycle lane is present on the approach to the roundabout, a bicycle ramp should be placed at least 50 feet prior to the crosswalk at the roundabout.
- Bicycle ramps should be placed at a 35 to 45 degree angle to the roadway.
- If the ramp is placed outside of the sidewalk, it can have up to a 20% slope; if the ramp is placed within the sidewalk, it should be designed in a manner to prevent a tripping hazard.
- If the ramp is placed outside the sidewalk, a detectable warning device should be placed at the top of the ramp; if the ramp is placed within the sidewalk, the detectable warning device should be placed at the bottom of the ramp.
- Bike ramps should be placed relatively far from the marked crosswalk as to prevent pedestrians from mistaking the ramp as a crosswalk.

Bike ramps at the exits of roundabouts should be built with the similar geometry and placement as the ramps that are designed at roundabout entries. Bike ramps at the exits of roundabouts should be placed at least 50 feet beyond the crosswalk of the roundabout. Refer to AASHTO 4.12.10 and the FHWA Roundabout Guide.

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CHAPTER 13
Traffic Signals

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General Information

A. Introduction

The purpose of this chapter is to supplement SUDAS Specifications Section 8010 and to provide general guidance for traffic signal designs on roadways within Iowa. The information is provided as an overview for traffic signals design consideration.

B. Scope

There is no legal requirement to use the information within this chapter by local agencies. This document refers to a number of other resources available for the designer to be considered when designing a traffic control signal. The document loosely follows the format of the MUTCD, as published by The U.S. DOT, FHWA and as adopted or modified by the Iowa DOT. However, no attempt is made to re-print the content of the MUTCD herein. A variety of other technical resources are also noted for consideration by the designer.

By MUTCD definition, a traffic control signal is “any highway traffic signal by which traffic is alternately directed to stop and permitted to proceed” with highway traffic signal being defined as “a power-operated traffic control device by which traffic is warned or directed to take some specific action. These devices do not include power-operated signs, illuminated pavement markers, barricade warning lights, or steady-burning electric lamps.” From an application standpoint traffic control signals are used to assign vehicular or pedestrian right-of-way.

The design for traffic control signals shall be in conformance with the current edition of the MUTCD as adopted or modified by the Iowa DOT. The following should be used as design standards as applicable to a project (all accessed October 2012):

- MUTCD Part 4 Highway Traffic Signals
- Jurisdiction Design Standards and Construction Standards
- Iowa DOT and FHWA regarding the design of traffic control signals
- Institute of Transportation Engineers - “Manual of Traffic Signal Design,” “Traffic Engineering Handbook,” “Traffic Signal Timing Manual,” “Manual of Traffic Engineering Studies” Robertson, H.D, Editor, J.E. Hummer, and D.C. Nelson. Institute of Transportation Engineers,Washington, DC, 1994 and “Traffic Control Devices Handbook.”
- Other standard references such as the National Electrical Code by the National Fire Protection Association (NFPA), and the National Electrical Manufacturers Association (NEMA) Standards Publications.

Other resources to consider and that are referenced within this document include:

- Mn/DOT Traffic Engineering Manual
- Mn/DOT Signal Design Manual
- Mn/DOT Signal and Lighting Certification Manual
- Mn/DOT Signals 101 Course Presentation
- Mn/DOT Signal Justification Reports
- Missouri DOT Traffic Control Devices
- Arizona DOT Traffic Engineering Policies, Guidelines, and Procedures

C. Definitions

A resource for traffic signal definitions can be found within MUTCD Section 4A.02 "Definitions Relating to Highway Traffic Signals."

Traffic Control Signal Needs Studies

A. General

The MUTCD states that "A traffic control signal should not be installed unless an engineering study indicates that installing a traffic control signal will improve the overall safety and/or operation of the intersection." The first question that must be answered is whether a traffic control signal is justified or is the most effective treatment option. It is the responsibility of the Engineer or agency to make this determination with serious consideration given to the following MUTCD Section 4B:

Section 4B.01 General
Section 4B.02 Basis of Installation or Removal of Traffic Control Signals
Section 4B.03 Advantages and Disadvantages of Traffic Control Signals
Section 4B.04 Alternatives to Traffic Control Signals
Section 4B.05 Adequate Roadway Capacity

B. Data Collection

The engineering study should be based upon a complete collection of site and traffic data (vehicle, pedestrian, etc) pertaining to the candidate location. Section 9-4.01 of the Mn/DOT Traffic Engineering Manual notes the studies which will be helpful in assessing and demonstrating the need for a signal as follows:

- Volume studies, including approach volumes, turning movements, and peak hour detail counts
- Pedestrian counts, including any unusual numbers of children, handicapped, and elderly
- Traffic gap studies
- Speed studies
- Crash studies
- Intersection delay studies

Procedures for completing various traffic studies are found in the ITE Manual of Traffic Engineering Studies.

MUTCD Section 4C.01 provides a detailed description of engineering study data which may be needed to conduct a warrant analysis. These include:

1. The number of vehicles entering the intersection in each hour from each approach during 12 hours of an average day. It is desirable that the hours selected contain the greatest percentage of the 24 hour traffic volume.

2. Vehicular volumes for each traffic movement from each approach, classified by vehicle type (heavy trucks, passenger cars and light trucks, public-transit vehicles, and, in some locations, bicycles), during each 15 minute period of the 2 hours in the morning and 2 hours in the afternoon during which total traffic entering the intersection is greatest.

3. Pedestrian volume counts on each crosswalk during the same periods as the vehicular counts in Item B above and during hours of highest pedestrian volume. Where young, elderly, and/or

persons with physical or visual disabilities need special consideration, the pedestrians and their crossing times may be classified by general observation.

4. Information about nearby facilities and activity centers that serve the young, elderly, and/or persons with disabilities, including requests from persons with disabilities for accessible crossing improvements at the location under study. These persons might not be adequately reflected in the pedestrian volume count if the absence of a signal restrains their mobility.

5. The posted or statutory speed limit or the 85th-percentile speed on the uncontrolled approaches to the location.

6. A condition diagram showing details of the physical layout, including such features as intersection geometrics, channelization, grades, sight-distance restrictions, transit stops and routes, parking conditions, pavement markings, roadway lighting, driveways, nearby railroad crossings, distance to nearest traffic control signals, utility poles and fixtures, and adjacent land use.

7. A collision diagram showing crash experience by type, location, direction of movement, severity, weather, time of day, date, and day of week for at least 1 year.

The following data, which are desirable for a more precise understanding of the operation of the intersection, may be obtained during the periods specified in item 2 of the preceding paragraph:

1. Vehicle-hours of stopped time delay determined separately for each approach.

2. The number and distribution of acceptable gaps in vehicular traffic on the major street for entrance from the minor street.

3. The posted or statutory speed limit or the 85th-percentile speed on controlled approaches at a point near to the intersection but unaffected by the control.

4. Pedestrian delay time for at least two 30 minute peak pedestrian delay periods of an average weekday or like periods of a Saturday or Sunday.

5. Queue length on stop-controlled approaches.

It is critical to present the above information in an organized fashion. Mn/DOT makes use of a Signal Justification Report, which contains the following information:

1. Intersection Location: Trunk highway cross-street name and county road numbers, municipality, and county. A map should be included that identifies the site.

2. Type of Work: Type of signal or beacon proposed, whether temporary or permanent.

3. Character of Site: Function and importance of roads, number of lanes, existing and proposed geometrics, channelization, grades, presence or absence of parking, bus stops and routes, posted speed limit, 85^{th} percentile speed if markedly different, and sight distance restrictions.

4. Land Use: Present land use at the intersection, presence of any special traffic generators, proposed or likely future development.

5. Traffic Control: Existing traffic control, present and planned adjacent signals, and proposed or existing coordinated systems.

6. Actual Traffic Volumes at the Intersection: Volumes must include at least 16 hours of counts on all approaches, turning movement counts for at least a.m. and p.m. peak hours. Unusual numbers of heavy vehicles and unusual percentages of turning movements must be noted. Volumes shall have been counted within two years of the date of submission of the report.

7. Iowa DOT generated or approved volume estimates for a proposed intersection, such as found in an official TAM or SPAR report, and for which warrant estimation methods are acceptable.

8. Pedestrian counts, particularly if the intersection is a school crossing or is used by large numbers of elderly or handicapped pedestrians.

9. Crash Data: Number and general types of crashes which have occurred for a minimum of 12 months before the date of the report. If Warrant 7 for crash experience is addressed, a collision diagram must be included, showing crashes by type, location in the intersection, directions of movement, severity, date, time of day, weather, light, and roadway conditions.

10. Any special site conditions adding to the Engineer's judgment that signals are necessary.

The above information can be presented in either checklist or narrative form, so long as it is clearly and logically presented. Volumes can be presented in graph or tabular form.

Mn/DOT’s Section 9-4.02.04 signal justification also provides a section on “Signal Removal Justification Criteria.”

C. Warrants

MUTCD Section 4C.01 “Studies and Factors for Justifying Traffic Control Signals” states, “An engineering study of traffic conditions, pedestrian characteristics, and physical characteristics of the location shall be performed to determine whether installation of a traffic control signal is justified at a particular location.

The investigation of the need for a traffic control signal shall include an analysis of the applicable factors contained in the following traffic signal warrants and other factors related to existing operation and safety at the study location:

Section 4C.01 Studies and Factors for Justifying Traffic Control Signals
Section 4C.02 Warrant 1, Eight-Hour Vehicular Volume
Section 4C.03 Warrant 2, Four-Hour Vehicular Volume
Section 4C.04 Warrant 3, Peak Hour
Section 4C.05 Warrant 4, Pedestrian Volume
Section 4C.06 Warrant 5, School Crossing
Section 4C.07 Warrant 6, Coordinated Signal System
Section 4C.08 Warrant 7, Crash Experience
Section 4C.09 Warrant 8, Roadway Network

The satisfaction of a traffic signal warrant or warrants shall not in itself require the installation of a traffic control signal.”

Accompanying MUTCD figures and tables for the above warrants include:

Table 4C-1 Warrant 1, Eight-Hour Vehicular Volume
Figure 4C-1 Warrant 2, Four-Hour Vehicular Volume
Figure 4C-2 Warrant 2, Four-Hour Vehicular Volume (70% Factor)

Figure 4C-3 Warrant 3, Peak Hour
Figure 4C-4 Warrant 3, Peak Hour (70% Factor)

Mn/DOT's Traffic Signal Design Manual Section 9-4.02 provides additional guidance for the following:

- Section 9-4.02.02 Warrants for Flashing Beacons at Intersections
- Section 9-4.02.03 Advance Warning Flashers Consideration

Features

A. Traffic Control Signal Features

The MUTCD Chapter 4D Traffic Control Signal Features establishes traffic signal uniformity and serves as a critical resource for checking each traffic signal design. The features of traffic control signals of interest to road users are the location, design, and meaning of the signal indications. Uniformity in the design features that affect the traffic to be controlled, as set forth in the MUTCD, is especially important for reasonably safe and efficient traffic operations. This chapter includes the following sections:

Section 4D.01 General
Section 4D.02 Responsibility for Operation and Maintenance
Section 4D.03 Provisions for Pedestrians
Section 4D.04 Meaning of Vehicular Signal Indications
Section 4D.05 Application of Steady Signal Indications
Section 4D.06 Application of Steady Signal Indications for Left Turns
Section 4D.07 Application of Steady Signal Indications for Right Turns
Section 4D.08 Prohibited Steady Signal Indications
Section 4D.09 Unexpected Conflicts During Green or Yellow Intervals
Section 4D.10 Yellow Change and Red Clearance Intervals
Section 4D.11 Application of Flashing Signal Indications
Section 4D.12 Flashing Operation of Traffic Control Signals
Section 4D.13 Preemption and Priority Control of Traffic Control Signals
Section 4D.14 Coordination of Traffic Control Signals
Section 4D.15 Size, Number, and Location of Signal Faces by Approach
Section 4D.16 Number and Arrangement of Signal Sections in Vehicular Traffic Control Signal Faces
Section 4D.17 Visibility, Shielding, and Positioning of Signal Faces
Section 4D.18 Design, Illumination, and Color of Signal Sections
Section 4D.19 Lateral Placement of Signal Supports and Cabinets
Section 4D.20 Temporary Traffic Control Signals
Section 4D.21 Traffic Signal Signs, Auxiliary

Accompanying MUTCD figures and tables for signal features include:

Table 4D-1 Minimum Sight Distance
Figure 4D-1 Maximum Mounting Height of Signal Faces Located Between 40 Feet and 53 Feet from Stop Line
Figure 4D-2 Horizontal Location of Signal Faces
Figure 4D-3 Typical Arrangements of Signal Lenses in Signal Faces

B. Pedestrian Control Features

The MUTCD Chapter 4E Pedestrian Control Features establishes pedestrian control uniformity and serves as a critical resource for checking each traffic signal design. Pedestrian signal heads provide special types of traffic signal indications exclusively intended for controlling pedestrian traffic. These signal indications consist of the illuminated symbols of a WALKING PERSON (symbolizing WALK) and an UPRAISED HAND (symbolizing DONT WALK). This Chapter includes the following sections:

Section 4E.01 Pedestrian Signal Heads
Section 4E.02 Meaning of Pedestrian Signal Head Indications
Section 4E.03 Application of Pedestrian Signal Heads
Section 4E.04 Size, Design, and Illumination of Pedestrian Signal Head Indications
Section 4E.05 Location and Height of Pedestrian Signal Heads
Section 4E.06 Accessible Pedestrian Signals
Section 4E.07 Countdown Pedestrian Signals
Section 4E.08 Pedestrian Detectors
Section 4E.09 Accessible Pedestrian Signal Detectors
Section 4E.10 Pedestrian Intervals and Signal Phases

Accompanying MUTCD figures and tables for pedestrian control features include:

Figure 4E-1 Typical Pedestrian Signal Indications
Figure 4E-2 Recommended Pushbutton Locations for Accessible Pedestrian Signals

C. Agency Specific Information

Agencies often have design requirements that differ or are in addition to those found in the MUTCD. Therefore, one of the first steps in the traffic signal design process is to learn the design requirements by meeting with agency staff, studying agency specific design manuals, and/or studying the MUTCD. Field observations of existing traffic signals within an agency's jurisdiction can also provide insight to specific design requirements.

Determining agency specific design requirements prior to design can be challenging. It can be difficult to ask all the right questions, give all the necessary answers, and not overlook any details. More challenges can arise when staff is less experienced or a new working relationship is being established. Most design requirements that are overlooked will be caught during the design process or review process. However, taking steps to prevent design requirements from being overlooked will accelerate the design process and minimize costs by eliminating or reducing change orders. The following are some examples of design requirements that can vary between agencies.

- The 2003 edition of the MUTCD requires a maximum distance of 180 feet from the stop line to the 12 inch signal faces unless a near side supplemental signal face is used. The previous version required a maximum distance of 150 feet and some agencies continue to follow the old requirement.
- Some agencies center mast arm mounted signal heads over the lane line and others center them over the center of the lane.
- Certain agencies elect to install supplemental signal heads on the vertical shaft of the mast arm pole and others elect not to.
- Doghouse style five section heads are used for protected / permissive left turns by some agencies but not others.

- Protected / permissive left turn lane operation can vary. Some agencies configure left turn lane loop detectors to call the protected phase only when all loop detectors are covered by vehicles while other agencies always call the protected phase.
- Detector types, sizes, and layouts vary between agencies.
- The size and number of conduits, handholes, and wiring varies greatly among agencies.
- Some agencies share conduit between signal cable, street light power, and/or interconnect while others keep these cables in separate conduits.
- Some agencies choose to install emergency preemption.
- Signal wiring details vary among agencies.
- Some agencies use the “astro” type brackets to mount all signal heads and others do not use this on side of pole mounted heads. Bracketing and banding of all hardware (typically to the poles) varies greatly among agencies.
- Traffic signal cabinets, cabinet risers, and controller types and preferences vary greatly among agencies.
- Mounting heights for signal heads, street light luminaires, detection cameras, monitoring cameras, etc. vary greatly among agencies.

D. Preliminary Signal Design Discussion List

Signal designers should meet and confer to agree on preliminary signal design details. Having a list of the basic criteria to be discussed at a preliminary stage can be of significant benefit to both the engineer and agency. The following list is based on Mn/DOT’s Signal Design Manual “Pencil Sketch” review list.

1. General nature of the signal project - new installation, minor or major revisions.

2. Phasing of the intersection, relation of proposed phasing to the traffic volumes and turning movements; use of protected-permissive left-turn phasing rather than protected-only; use of overlaps.

3. Determine design standards based on who will operate the system.

4. Use of four and five section heads and non standard bracketing.

5. Head type (LED, optically programmed, etc.).

6. Appropriateness of poles and pedestals for the site.

7. Placement of signal standards to ensure legal placement of all vehicle and pedestrian signal indications.

8. Placement of pedestrian pushbuttons relative to signal standards and in place sidewalks and crosswalks.

9. Need for emergency vehicle pre-emption (EVP) and police door with auto/flash switch, manual/stop time switch, and on/off power switch for signal heads only, including placement of components.

10. Detector placement and functions. See the Signal Design Manual for loop detector placement diagrams.

11. Placement and type of handholes.

12. Design of equipment pad.

13. Type of service equipment.

14. Discuss needs for combined pad with lighting and/or TMC.

15. Need for intersection geometric improvements.

16. For revised systems, the wording of the signal pole notes for the revision.

17. Need for AWF's, supplemental heads, etc.

18. House moving route needs (Mn/DOT uses a mast-arm mount that can swivel).

19. Painting of signal.

20. Luminaires metered or unmetered.

21. Source of power (to determine cabinet location).

22. Interconnect (determine need and type, location of master).

E. Additional Information

The MUTCD Chapter 4E Pedestrian Control Features establishes pedestrian control uniformity and serves as a critical resource for checking each traffic signal design. Pedestrian signal heads provide:

Chapter 4F Traffic Control Signals for Emergency Vehicle Access
Chapter 4G Traffic Control Signals for One-Lane, Two-Way Facilities
Chapter 4H Traffic Control Signals for Freeway Entrance Ramps
Chapter 4I Traffic Control Signals for Movable Bridges
Chapter 4J Lane-Use Control Signals
Chapter 4K Flashing Beacons
Chapter 4L In-Roadway Lights

Design Considerations

In addition to basic MUTCD requirements, the safe and efficient operation of a signalized intersection requires careful attention and balance of a number of design parameters. This section provides some reference resources for the traffic signal designer in consideration of these features.

A. Geometrics

The geometrics of an intersection are a critical consideration given the potential impact on intersection safety and performance. Geometrics directly impact sight distance, vehicle separation, operations, and capacity. As a result, intersection geometrics should always be considered whether dealing with existing, reconstructed, or new signalized intersections.

References are made to Signalized Intersections: Informational Guide, FHWA-HRT-04-091, August 2004, which provides a single, comprehensive document with methods for evaluating the safety and operations of signalized intersections and tools to remedy deficiencies. The treatments in this guide range from low-cost measures such as improvements to signal timing and signage, to high-cost measures such as intersection reconstruction or grade separation. While some treatments apply only to higher volume intersections, much of this guide is applicable to signalized intersections of all volume levels.

1. **Basic Geometric Considerations:** The geometric design section of the Signalized Intersections: Informational Guide provides the following comments:

   Geometric design of a signalized intersection involves the functional layout of travel lanes, curb ramps, crosswalks, bike lanes, and transit stops in both the horizontal and vertical dimensions. Geometric design has a profound influence on roadway safety; it shapes road user expectations and defines how to proceed through an intersection where many conflicts exist.

   In addition to safety, geometric design influences the operational performance for all road users. Minimizing impedances, eliminating the need for lane changes and merge maneuvers, and minimizing the required distance to traverse an intersection all help improve the operational efficiency of an intersection.

   The needs of all possible road users must be considered to achieve optimal safety and operational levels at an intersection. At times, design objectives may conflict between road user groups; the practitioner must carefully examine the needs of each user, identify the tradeoffs associated with each element of geometric design, and make decisions with all road user groups in mind.

   The Geometric Design section addresses the following design topics to be considered when designing traffic signal controlled intersections:
   - 3.1 Channelization
   - 3.2 Number of Intersection Legs
   - 3.3 Intersection Angle
   - 3.4 Horizontal and Vertical Alignment
   - 3.5 Corner Radius and Curb Ramp Design
   - 3.6 Sight Distance

- 3.7 Pedestrian Facilities
- 3.8 Bicycle Facilities

2. Additional Sight-distance Considerations:

a. Sight distance is a safety requirement that impacts intersection geometrics as fundamental as horizontal and vertical alignments. It is a design requirement that is discussed in detail as it relates to the visibility of traffic signal indications in the MUTCD. In addition to the sight distance requirements of the MUTCD, the AASHTO "Policy on Geometric Design of Highways and Streets 2001" states that drivers of the first stopped vehicles on all approaches should have adequate sight distance to view one another. It also states that left turning vehicles should have adequate sight distance to select gaps in oncoming traffic and complete turning maneuvers. This requires consideration of offset left turn lanes to provide adequate left turn sight distance. If right turns are allowed on a red signal indication, the appropriate departure sight triangle should be provided. Finally, the policy states that the appropriate departure sight triangles should be provided for left and right turning vehicles on the minor approach for two-way flashing operations. Two-way flashing operations are flashing yellow for the major street and flashing red for the minor street. See Chapter 9 - Intersections in the AASHTO "Policy on Geometric Design of Highways and Streets 2001" for additional sight distance information.

b. One sight distance issue that deserves additional consideration is the sight triangle and the sight obstructions found within it. Certain obstructions are obvious like structures near the street. Other obstructions are not always obvious or are installed after the traffic signal is designed and constructed. These obstructions seem to blend into the background. They are obstructions like entrance monuments, special street name signs, business signs, and landscape vegetation that may not be a problem initially but become a problem as the plants reach maturity. Finally, be aware of the signal cabinet size and location including the height of the footing or cabinet riser so it does not become a sight obstruction.

c. Sight distance requirements are less restrictive at signalized intersections as drivers are required by law to obey the signal indications; however, there are instances when drivers do not obey traffic signals. A traffic signal should be designed to exceed minimum sight distance requirements when possible. Drivers are taught to drive defensively and providing additional sight distance will only aid drivers in collision avoidance.

3. Turn Lanes:

a. Traffic volumes, turning movement counts, and crash history are used to complete intersection capacity and accident analyses. The results of the analyses determine the need for turn lanes, the number of turn lanes, and the length of the turn lanes. The turn lane information is used to properly design the geometrics of signalized intersection approaches.

b. Turn lane capacity issues often create safety problems. Left or right turning vehicle queues blocking through traffic create increased potential for rear-end accidents. Sideswipe potential also increases as traffic attempts to maneuver out of defacto turn lanes or around left turn queues blocking through lanes. High volumes of turning vehicles combined with high volumes of opposing vehicles significantly reduce the number and size of available gaps needed to complete turning maneuvers increasing the potential for right angle collisions. As a result, properly designed turn lanes improve safety as well as capacity.

c. Determining turn lane design details when upgrading existing signalized intersections in largely developed areas is relatively straight forward. Capacity problems are recognized through evidence obtained from capacity analyses, visual inspections, and/or citizen comments. Capacity analyses and visual inspections of peak hour traffic often reveal long queues that do not clear after multiple signal cycles. Heavy turning volumes and a lack of turn lanes on multilane facilities often result in shared lanes acting as defacto turn lanes. If turn lanes exist, traffic volumes may exceed the capacity of the turn lanes resulting in vehicle queues spilling out of the turn lanes and into the through lanes.

d. Determining turn lane design details when constructing new signalized intersections in undeveloped or under developed areas experiencing significant growth is a challenge. In many cases, there is no visual evidence of existing capacity or safety problems. The challenge is judging future traffic patterns and the extent of the traffic growth over a given time period, usually twenty years, with no guarantees as to the type, extent, and rate of development. Judgment is improved with information and the information is obtained from capacity analyses that examine existing and proposed development, existing traffic volume data, and future traffic volume data derived from land use maps and the ITE Trip Generation Manuals. This information combined with traffic growth rates obtained from developed areas with similar land use characteristics and engineering judgment are used to arrive at an intersection design that will support existing traffic volumes as well as future growth.

e. Past experience has helped to formulate several design guidelines used to initially determine the number of lanes needed at an intersection. These guidelines are planning level guidelines and should be confirmed with the results of the operational analysis methods discussed in the Operations section of this chapter. The guidelines can be found in Chapter 10 of the Highway Capacity Manual 2000 (HCM 2000) and are summarized as follows:
   1) Exclusive Left Turn Lanes:
      - A single exclusive left turn lane should be considered when the minimum left turn volume is 100 veh/hr.
      - Dual exclusive left turn lanes should be considered when the minimum left turn volume is 300 veh/hr.
   2) Exclusive Right Turn Lanes:
      - An exclusive right turn lane should be considered when the right turn volume exceeds 300 veh/hr and the adjacent mainline volume exceeds 300 veh/hr/ln.
   3) Number of Lanes:
      - Enough lanes should be provided to prevent the total volume of the approach from exceeding 450 veh/h/ln.

f. Past experience has also helped to formulate several design guidelines used to initially determine turn lane lengths needed at intersections. Like the guidelines used to determine the number of lanes, the guidelines used to determine turn lane lengths are planning level guidelines and should be confirmed with the results of an operational analysis. Also remember that the lengths discussed here are the actual storage lengths and do not include taper lengths. Taper requirements are discussed in several sources including Chapter 5 - Roadway Design, the Iowa DOT Design Manual, and the AASHTO Policy on Geometric Design of Highways and Streets. The guidelines are as follows:
   - Enough storage length should be provided to equal one foot for each vehicle per hour (vph) turning during the peak hour in the horizon year. For example, 250 vph turning during the peak hour in the horizon year would require a 250 foot turn lane.

- Storage length can also be computed using the following equation:

  Storage Length = (h / s) (v + g) (p)

  h = horizon year peak hour volume (vph)

  s = number of signal cycles per hour
  A signal cycle is typically 60 to 120 seconds. Engineering judgment is used to select the cycle length or lengths to use in the equation.

  v = average vehicle length
  The average vehicle length often used is 20 feet.

  g = average gap between vehicles
  The average vehicle gap often used is 5 feet.

  p = probability factor
  The probability factor is based on the Poisson distribution and associated with the probability that enough length is provided to store all vehicles.

Probability Factor (p)	Probability of Storing All Vehicles
1.50	0.90
1.75	0.95
1.85	0.98
2.00	> 0.98

  A paper written by the Transportation Research Institute at Oregon State University suggests modifying the average vehicle length plus gap (v + g) based on the percentage of trucks using the turn lane. The paper suggests modifying v + g as follows:

Percent Trucks	v + g
< 2%	25’
5 %	27’
10 %	29’

  The initial storage length for dual left turn lanes can be found by dividing the storage length found from one of the two methods discussed above by 1.8.

  Example:

  h = 250 vph
  s = 100 s/cycle
  3600 s/hr / 100 s/cycle = 36 cycles/hr
  5% trucks
  v + g = 27’
  p = 1.85 (95% probability)

  Single lane storage length = (250 / 36) (27) (1.75)
  Single lane storage length = 328’: Say 325’

Determining turn lane length also requires some additional considerations. One consideration is the length of the queues in the through lanes. If the turn lanes are not long enough, through lane queues may prevent turning vehicles from entering the turn lanes leaving the turn lanes nearly empty until the through lane queues begin clearing. This issue could be addressed with lagging lefts but lagging lefts require additional considerations to prevent left turn traps and an operational analysis to determine optimal signal phasing and timing. If through lane queues block the turn lanes, the turn lanes could be lengthened beyond the through lane queues. However, the additional length needed may not be practical.

Another consideration is maximum turn lane length. Once a turn lane becomes too long, the signal cycle cannot serve all the traffic waiting in the turn lane reducing, if not eliminating, the benefits of the extra length. At this point, it may be more practical to add turn lanes or look at other solutions to relieve congestion. When is a turn lane too long? It is difficult to point to an exact number but in the neighborhood of 350 to 400 feet. An operational analysis will provide better evidence regarding the maximum length.

The final consideration that can impact the length of a turn lane is visibility. A turn lane that starts just beyond the crest of a vertical curve may not be visible until a vehicle is at the start of the lane. It may be practical to extend the turn lane to increase its visibility giving drivers more time to react to the lane.

g. Lane balance should be considered when addressing lane geometrics. Left turn lanes should be opposing or offset to one another. If dual left turn lanes are required on one approach, dual left turn lanes or a wide median should be installed on the opposing approach to promote lane balance. Through lanes should be located so they align with one another as the intersection is traversed. Creating a lane shift through an intersection creates driver confusion.

4. Agency Geometric Considerations: The Mn/DOT Traffic Engineering Manual (Section 9-6.00 Traffic Signal Design) provides a good identification of major issues for design consideration and serves as an example of agency specific criteria. Since this is a PDF document, Sections 9-6.02 through 9-6.05 are provided below:

Intersection geometry is an important element of traffic signal design. The design of traffic signal system hardware and operation of the traffic signal system should be preceded by a thorough evaluation and, if necessary, geometric improvement of the existing intersection. Mn/DOT Section 9-6.03 notes the following geometric elements should be considered:

a. Pavement width should be adequate for anticipated traffic movements and future capacity requirements. Highway capacity analysis should be performed to get a better understanding of the capacity of the intersection.

b. If appropriate islands should be designed and constructed so that the driver has adequate reaction distance to them and they are large enough to install a standard signal foundation. Existing shoulders should always be carried through the intersection; this will usually provide enough reaction distance to the island. However, turning radii should be checked to ensure enough setback for comfortable turns.

c. Turn lanes must provide adequate storage in order to prevent turning traffic from interfering with other traffic movements and thus causing capacity breakdown.

d. When a median width is more than 30 feet between opposing through lanes, special signal design considerations are necessary (See MN MUTCD, Section 4H). Extremely wide medians confuse drivers on the crossing street, prevent them from being comfortable with opposing traffic, and cause them to lose track of their path. Wide medians also cause capacity restrictions because more time is needed for vehicle movements and clearances through the intersection.

e. Sidewalks should be constructed as close to the center of the corner as possible. Pedestrian crosswalks should be in line with the sidewalk and as close to the intersection as practical.

f. Alignment changes within the intersection should be avoided. Vehicles approaching the intersection should be directed through the intersection. Vertical alignments approaching signals must allow for proper signal visibility.

g. Driveways within an intersection should be signalized and accommodated by the intersection geometrics. Whenever feasible, the driveways should be located or relocated outside the limits of the intersection.

h. The size of corner radii is an important consideration. Excessively large corner radii may obscure intersection limits and create a hazard for bicycles and pedestrians, while very small radii may create a hazard for motorists. Corner radii at signalized intersections should not be less than 20 feet nor more than 60 feet. A turning radius guide for 58 foot vehicles should be used to determine proper corner radii. At intersections where bus routes are located, corner radii should be analyzed giving due consideration to bus maneuvers.

i. It may be necessary to relocate utilities such as manholes, catch basins, fire hydrants, overhead power and telephone lines and power poles, to obtain adequate geometrics for signalization. The existence of these utilities must not get in the way of adequate geometrics.

j. Pedestrian curb ramps should be considered in accordance with Chapter 12 - Sidewalks and Bicycle Facilities if sidewalks are present.

k. Handhole spacing should be based on the following factors:
   - Location of junction points within the signal system
   - Physical features, such as driveways, utilities, etc.
   - Cable pull length based on size of cable and diameter of conduit

B. Operational Characteristics

The behavior of the traffic at an intersection is another highly important element of signal design. Mn/DOT Section 9-6.03 notes the following elements should be considered:

1. Existing 15 minute vehicle volumes, by vehicle class, and pedestrian volumes, are the most basic operational consideration. Data used should represent intersection operation in peak periods. Saturated approaches should have an upstream count taken to determine the demand volume rather than the service volume at the intersection.

2. Intersection capacity should be determined based on the Highway Capacity Manual and other sources.

3. The vehicle approach posted speeds should be determined for the location of advance detection.

4. Adjacent land uses should be evaluated to identify activities which may conflict with intersection operation. Items that should be considered include entrances, advertising devices, and areas of high pedestrian activity (schools, manufacturing plants, shopping centers, etc.).

5. Crashes within the intersection should be studied to determine causes and possible design solutions.

6. Pedestrian volumes and school-crossing activities should be studied to determine pedestrian routes and necessary design treatments. Pedestrian movements in and around signals should be routed into the intersection crosswalks in front of vehicles stopped for the signal. Provide pedestrian refuges in medians 6 feet and wider.

C. System (Arterial) Considerations

In many cases, an individual traffic control signal must be considered as part of a system, either as one of a series of signals along a linear route, or as one signal in a grid network. Mn/DOT Section 9-6.04 notes the following elements should be considered.

System considerations in signal design should include but are not limited to the following:

1. Adjacent signals should be interconnected whenever they are less than one-half mile apart, when the travel time between adjacent signals is less than the cycle length at each signal, or when platoons leaving one intersection remain intact to the next signal.

2. Properly spaced signalized intersections greatly simplify coordination in planning new signals. Minimum spacing of one-quarter mile is recommended. Irregular signal spacing reduces the overall operational efficiency of the mainline movements and greatly complicates signal coordination.

3. Whenever possible, platoons should be kept intact to allow easier mainline coordination and minimize cross-street delay.

4. New street or roadway construction should anticipate the need for future signals and the need for handholes and conduit, particularly under the roadway.

5. Pretimed controllers are used in built-up urban environments, particularly central business districts. The streets are not excessively wide and the traffic patterns are quite predictable. In this environment, a signal cycle should contain pedestrian movements. Actuated controllers are used in suburban and rural environments. In the rural environment, the actuated controller tends to reduce the number of stops and does not cut off platoons of vehicles. In the suburban environment, the arterial streets tend to be very wide, and the volumes are usually quite high on these arterials. There are not usually many pedestrians crossing such an arterial, so an actuated controller tends to operate much more efficiently, as it is not necessary to time pedestrian intervals except when an actual demand exists.

6. Splits and offsets should be carefully estimated to determine their impact on arterial flow. A split is the relative percentage of green time allocated to each of the various phases at a single intersection. An offset is the travel time between signals, usually expressed in percent of cycle length.

7. Minimum pedestrian walk and clearance timings should be anticipated when designing coordinated signal systems.

D. Signal Design Elements

Mn/DOT Section 9-6.05 notes the following elements should be considered:

1. The most efficient operation of a signal system is attained with the fewest phases that are enough to move traffic without hazardous conflicts. Procedures exist to determine the optimum number of phases for an intersection.

2. The primary consideration in signal head placement is clear visibility. Drivers approaching an intersection shall be given a clear and unmistakable indication of their right-of-way assignment. The number and placement of signal faces shall conform to the requirements of the MUTCD. Overheads should be located as near as practicable to the line of the driver's normal view. When an overhead is to control two lanes, it should be installed over the lane line dividing the two lanes. An overhead should be used over each lane when speeds are above 40 mph. The size of lenses shall be as stated in the MUTCD. See the signal head placement charts in the Signal Design Manual. In general, vehicle signal faces should be placed and aimed to have maximum effectiveness for an approaching driver located a distance from the stop line equal to the distance traveled while reacting to the signal and bringing the vehicle to a stop at an average approach speed. Visors, shields, or visual delimiting should be used to help in directing the signal indication to the approaching traffic, and to reduce sun phantom resulting from external light entering a signal lens.

3. Vehicle detectors should be placed according to the detector spacing chart and the loop placement diagrams.

4. At locations where pedestrians are expected, provisions must be made to control pedestrian activity in and around the signalized intersection. At locations where pedestrians are expected, pedestrian indications shall be provided if minimum pedestrian crossing time exceeds minimum vehicular green time, or if any of the conditions set out in section 4E.3 of the MN MUTCD are met. Pedestrian push buttons should be installed at locations with pedestrian activity where it is not operationally efficient to provide pedestrian timing on every cycle. Pedestrian signal indications shall be mounted, positioned, and aimed so as to be in the line of pedestrians' vision, and to provide maximum visibility at the beginning of the controlled crossing.

5. If it is determined to prohibit pedestrian movement across any approach, that prohibition must be clearly visible to pedestrians by use of Standard Sign R9-3a on each side of the prohibited crosswalk. See part 4 of the MN MUTCD for further information.

6. Street lighting should normally be installed with traffic signals and flashing beacons. The luminaires are generally 250-watt high-pressure sodium vapor luminaires, mounted in the far-right quadrants of the major street. Larger intersections may require additional luminaires. Forty foot mounting heights provide even light distribution. Street lights installed on Type A signal mast-arm poles should be mounted at approximately 350 degrees clockwise from the mast arm in order to provide frontal illumination of any signs mounted on the mast arm.

Signal design must take into account the existing adjacent lighting systems and the equipment available to provide access to the luminaires for relamping and maintenance. The presence of overhead power lines must also be taken into account. These must be designed around or moved.

E. Traffic Signal Operations

The Mn/DOT Traffic Engineering Manual provides an exceptional discussion on basic traffic signal operations and design considerations. These are not reprinted within this document but these references are noted below.

- Mn/DOT Traffic Signal Timing and Coordination Manual
- Traffic Engineering Manual
  - Chapter 2. Traffic Signal Phasing and Operations
  - Chapter 3. Head Placement Charts
  - Chapter 4. Detection
- Mn/DOT Signal & Lighting Certification Manual

F. Pedestrian Considerations

1. Geometrics:

a. Geometrics have a significant impact on pedestrian operations and safety at signalized intersections as alluded to in the previous section. Intersection skew, number of lanes, lane width, medians, islands, and curb returns all impact the distance pedestrians must travel to cross an intersection. As the distance to traverse an intersection approach increases, so does the signal timing that must be allocated to the pedestrian clearance interval. Long pedestrian clearance intervals have a negative impact on traffic capacity and operations. A pedestrian actuation will disrupt traffic signal coordination and require several cycles to bring a corridor back into coordination. However, large pedestrian volumes may dictate signal timing resulting in less than optimal conditions for vehicles. A traffic engineer must balance the priorities of vehicles and pedestrians with no calculations or answers that clearly define a solution but do provide guidance.

b. Right turns present challenges for pedestrians. A driver of a vehicle turning right on red will be looking left for a gap in traffic. A pedestrian approaching from the right may have a walk indication. If the driver sees a gap but does not look back to the right, the pedestrian may not be seen by the driver resulting in a collision. As a result, a traffic engineer must decide whether to allow right turns on red.

c. Right turn lanes can present additional challenges for pedestrians, especially if the returns are large and channelize traffic with an island. The islands can channelize right turning vehicles away from the traffic signal indications creating difficulties signalizing the right turn movement. Using a stop sign instead of a supplemental signal indication for the channelized right turning movement is not an option. It creates a confusing message when all movements on the approach see green indications, including right turning vehicles, until they are partially through the turning maneuver and see a stop sign. Some agencies assign the right turning vehicles a yield sign but it creates an issue protecting pedestrians. If a pedestrian push button is used at the back-of-curb and pedestrians must cross a right turn lane controlled by a yield sign, it may give pedestrians a false sense of security when crossing in front of right turning vehicles. Drivers of right turning vehicles see a yield sign and look left, away from the pedestrians stepping off the curb, for a gap in traffic. In fact, drivers of right turning vehicles would be looking even farther left due to the channelization and orientation of the vehicles making it even more difficult for drivers to see pedestrians approaching from the right. Consequently, pedestrian volume and safety are important considerations when considering and designing right turn lanes.

d. The final geometric consideration as it relates to pedestrians is the pedestrian refuge. Right turn islands and medians often double as pedestrian refuges. If islands and medians are intended to be used as pedestrian refuges, they must be large enough to hold pedestrians and be ADA compliant. A traffic engineer must consider the likelihood that pedestrians will stop and get stranded in an island or median. On large approaches, it may be intended that pedestrians only cross a portion of the approach and stop in a median or island. As a result, a traffic engineer must decide whether to install supplemental push buttons in the right turn island or median. If islands and medians are not intended to function as pedestrian refuges, they must be located so they do not obstruct the path of pedestrians.

2. **Visibility:** Visibility is important to the safe operation of the pedestrian indications. Pedestrian indications as well as the push buttons should be easily located by pedestrians. Consider where vehicles, especially large trucks, may stop so they do not obstruct the view of the pedestrian indications. This will require careful location of median noses, stop bars, crosswalks, and the pedestrian heads. Finally, make sure there are no obstructions in the returns that may prevent drivers and pedestrians from seeing one another such as the signal cabinet or vegetation.

3. **Special Considerations:** Circumstances often arise that require special considerations. For example, children may have difficulty understanding the meaning of pedestrian indications. Count down pedestrian heads may be easier for children to understand; therefore, have increased value in school zones. Count down pedestrian heads may also have added value on wide approaches. The flashing numbers can attract a person’s eye and the numbers tell a pedestrian how much time they have to cross which has added value on very wide approaches. There may be a particular area within a city that has a high concentration of visually impaired. In this case, audible pedestrian indications may have added benefit. In many cases, some extra thought and minimal dollars can change a design from adequate to desirable.

4. **Americans with Disabilities Act:** The Americans with Disabilities Act (ADA) addresses several design requirements relating to pedestrians. ADA addresses design requirements for items such as sidewalk ramps, truncated domes, and pedestrian push buttons. These topics are addressed in detail in Chapter 12 - Sidewalks and Bicycle Facilities and other design manuals such as the MUTCD and the AASHTO Policy on Geometric Design of Highways and Streets.

   a. Accessible Pedestrian Signals (APS): Each traffic signal project location should be evaluated to determine the need for accessible pedestrian signals, especially if the project location presents difficulties for individuals with visual disabilities. An engineering study should be completed that determines the needs for pedestrians with visual disabilities to safely cross the street. The study should consider the following factors:
   - Potential demand for accessible pedestrian signals
   - Requests for accessible pedestrian signals by individuals with visual disabilities
   - Traffic volumes when pedestrians are present, including low volumes or high right turn on red volumes
   - The complexity of the signal phasing, such as split phasing, protected turn phases, leading pedestrian intervals, and exclusive pedestrian phases
   - The complexity of the intersection geometry

   One tool that is available for evaluation of the need for APS and also prioritizing the order for installing APS equipment on crosswalks can be found at www.apsguide.org developed by the National Cooperative Highway Research Program (NCHRP).

If APS are warranted, it is necessary to provide information to the pedestrian in non-visual formats. This will include audible tones and vibrotactile surfaces. Pedestrian push buttons should have locator tones for the visually impaired individual to be able to access the signal. Consistency throughout the pedestrian system is very important. Contact the Jurisdictional Engineer regarding the standards and equipment types that should be incorporated into the design of the accessible pedestrian signal system. New tones such as clicks, ticks, and other electronic sounds have replaced the cuckoos and chirping tones of past systems.

b. APS Design Elements: Refer to MUTCD Sections 4E.08 through 4E.13 and the following information.

1) **Push Button Stations:** An APS push button station is a weather-tight housing with a 2 inch diameter push button, a speaker, and a pedestrian sign. Braille signing, raised print or a tactile map of the crosswalk may also be provided. The push button has a vibrotactile arrow pointing in the direction of the crossing.
2) **Location of Pedestrian Push Buttons:** Push buttons should be located adjacent to the sidewalk, between 1.5 and 6 feet from the edge of curb, shoulder, or pavement and no more than 5 feet from the outside crosswalk line. Where physical constraints make the 6 feet maximum impractical, push buttons should be located no more than 10 feet from the edge of curb, shoulder, or pavement. Where two push buttons are provided on the same corner of the intersection, they should be separated by at least 10 feet. If the 10 feet separation is not feasible, audible speech walk messages are required. Supplemental push button poles or posts will typically be needed to meet the above criteria. Push buttons should be mounted at a height of approximately 3.5 feet, but no more than 4 feet above the adjacent sidewalk. The push button should be located so pedestrians using the audible or vibrotactile indication can align themselves and prepare for the crossing while waiting close to the push button station and the crossing departure point.
3) **Locator Tone:** APS push buttons have a locator tone to allow visually impaired individuals to access the signal. The locator tone should be audible 6 to 12 feet from the push button. The locator tone is active during the pedestrian clearance and “DON’T WALK” intervals.
4) **Walk Indications:**
   - In addition to visual indications, APS include audible and vibrotactile walk indications. When at least 10 feet separation is provided between pedestrian push button stations, the audible walk indication is a percussive tone. If 10 feet separation is not provided, speech messages are required. The speech message should name the street to be crossed and indicate that the walk sign is on. For example: “Main. Walk sign is on to cross Main.” Other audible messages may be developed, including counting down the pedestrian clearance time, depending on the needs of the particular crosswalk or intersection. Designations such as “Street” or “Avenue” should not be used unless necessary to avoid ambiguity at a particular location. If the traffic signal rests in WALK, the tone/message should be limited to 7 seconds and be repeated with each actuation.
   - The vibrotactile walk indication is provided by a high visual contrast tactile arrow on the push button that vibrates during the walk interval. The vibrotactile indication is particularly useful to individuals who have both visual and hearing impairments. The pedestrian must be able to stand with a hand on the device while being aligned and waiting to begin the crossing. The arrow should be aligned parallel to the direction of travel on the associated crosswalk.

c. APS System Options:

- Products currently in the marketplace involve use of 2-wire or 4-wire systems, indicating the number of wires between the push button station and the control unit (CU). The 2-wire system uses a central CU mounted in the controller cabinet, and may provide Ethernet connectivity. Advantages of this system include minimal field wiring required on retrofit applications and central control of multiple crossings.
- The 4-wire system requires a separate CU mounted in the applicable pedestrian signal head for each push button station. In addition to the typical two wires between the push button and the controller cabinet, a 4-wire cable must be provided between the push button station and the CU. This system may be more cost effective for installations with only one or two crossings.

d. APS Compliant Equipment: The following equipment currently meets 2009 MUTCD and 2011 proposed public right-of-way accessibility guidelines (PROWAG) for accessible pedestrian signals. Other compliant equipment may also be available.

- Advisor Guide and Advisor Advanced Pedestrian Stations (AGPS and AAPS) manufactured by Campbell Company.
- EZ Communicator Navigator APS manufactured by Polara.

e. Location of Pedestrian Push Buttons: It is common to see a narrow grass strip between the sidewalk and pole used to mount the push buttons or to only see sidewalk on one side of a pole containing multiple push buttons. It is difficult to impossible for a person in a wheelchair to reach the push button in cases like these since it often requires the person to struggle with one wheel in the grass and one on the sidewalk. As a result, sidewalks must be paved up to the pole used to mount the push buttons and be at a reasonable slope. There should also be sidewalk on each side of a pole that has a push button. The MUTCD requires a pedestrian push button mounting height of approximately 3.5 feet above the sidewalk; keep in mind that the 3.5 feet is above the grade where the pedestrian would be when accessing the button. Often times pole foundation elevations end up above grade and installing a push button based on the foundation elevation and not the ground elevation where the pedestrian accesses the button results in a mounting height that is too high. Finally, consider the proximity of the push buttons to the street. If the poles used to mount the push buttons are too far from the street, pedestrians will not use the push buttons. Consider installing supplemental poles closer to the street for mounting the push buttons.

G. Driver and Pedestrian Expectations

Other traffic signal design considerations involve driver and pedestrian expectancy. A traffic engineer must look beyond the traffic signal being designed and consider the characteristics of the corridor and the attributes of the existing traffic signals along the corridor. For example, left turn phasing should be applied consistently and not switch between protected only and protected/ permissive without legitimate reasons. If pedestrian signal heads are used, they should be used consistently and not sporadically where one intersection uses the heads and the next intersection relies on vehicular signal heads to guide pedestrians. Traffic signal head style, placement, and orientation should be consistent along a corridor as well as sign type, size, and location. Intersections should not randomly switch between doghouse and vertical five section heads, center of lane and lane line placement, or vertical and horizontal signal head orientation. Consistently applied design criteria improve driver and pedestrian expectations which typically promote safety and operations. However, circumstances exist that may, at times, require changes to design criteria to increase vehicle and pedestrian safety and operations.

H. Future Development and Improvements

One of the biggest traffic signal design challenges is designing a traffic signal in an area that is under developed or being redeveloped. Under these circumstances, much of the data needed for design is either unknown or unstable. Land uses are often modified and business prospects continually change often having significant impacts on existing and future traffic volumes. In addition, the rate at which traffic volumes will increase is difficult to determine. In such cases, the traffic signal designer must work closely with adjacent area land use planning agencies to work towards reasonable expectations for future travel demands and overall operations. Future phases can be accommodated for within the design to significantly reduce the need to replace foundation locations, adjust mast-arm lengths, or add additional functionality to the traffic signal. These simple steps can build credibility with the public and add considerable efficiency to the traffic signal design and overall engineering process.

Specifications Information

This section provides design information that complements and is organized similar to SUDAS Specifications Section 8010, which includes:

Part 1 - General

Part 1 provides direction on general items such as submittals; substitutions; delivery, storage, and handling; scheduling and conflicts; and measurement and payment.

Part 2 - Products

Part 2 describes the products to be provided and is arranged as follows:
- 2.01 Underground
- 2.02 Detection
- 2.03 Communications
- 2.04 Cabinet and Controller
- 2.05 Poles, Heads, and Signs

Part 3 - Execution

Part 3 describes how these products should be installed and matches the arrangement described in Part 2, with the following additions:
- 3.06 Temporary Traffic Signal
- 3.07 Surface Restoration
- 3.08 Testing
- 3.09 Documentation

The information below provides selective guidance on the specifications.

A. Part 1 - General

1. **Submittals:** There are several key submittals required of the contractor following award of the project. These are described below.

   a. **Schedule of Unit Prices:**
      1) **Document:** Prepared by the traffic signal designer and included within the contract documents (generally attached to the back of the traffic signal specifications).
      2) **Purpose:** Contracting authority approval of the unit pricing for all major traffic signal items. Establish unit pricing for change order work if needed. Used to estimate partial payments.
      3) **Includes:** Identification of major traffic signal items along with an estimate of quantity and units of measurement. Two additional blank columns are provided (unit price, and unit extension).
      4) **Contractor Action:** Within 30 days after award, the contractor is required to submit a completed schedule of unit prices to the contracting authority for engineer approval.

5) **Engineer Action:** Review the schedule in a timely manner. Check the appropriateness of each unit price, the accuracy of each unit extension calculation, and ensure that the grand total for all unit extensions matches the lump sum bid item for traffic signalization. Upon acceptance, sign and date the document and provide a copy to the contractor.

b. Material and Equipment List:

1) **Document:** Prepared by the traffic signal designer and included within the contract documents (generally attached to the back of the traffic signal specifications).
2) **Purpose:** Contracting authority approval of the make and model numbers for all major traffic signal items.
3) **Includes:** Identification of major traffic signal items along with an estimate of quantity and units of measurement. Two additional blank columns are provided (manufacturers name and each items model number).
4) **Contractor Action:** Within 30 days after award, the contractor is required to submit a completed list of materials and equipment to the contracting authority for engineer approval.
5) **Engineer Action:** Review the schedule in a timely manner. Check the appropriateness of each identified manufacturer and model number. Upon acceptance, sign and date the schedule and provide a copy to the contractor.

c. Contractor Certification:

1) **Document:** Prepared by the contractor on company letterhead.
2) **Purpose:** Contracting authority approval of key project personnel.
3) **Includes:** Name, contact information, and certification of the Level II International Municipal Signal Association (IMSA) Certified Traffic Signal Technician(s) working on the project.
4) **Contractor Action:** Within 30 days after award, the contractor is required to submit the contractor certification to the contracting authority for engineer approval.
5) **Engineer Action:** Review the appropriateness of the information and on acceptance, sign and date the document, and provide a copy to the contractor.

d. Shop Drawings:

1) **Document:** Prepared by the traffic signal pole supplier for the contractor.
2) **Purpose:** Contracting authority approval of traffic signal poles, supports, and related hardware.
3) **Includes:** Shop drawing information detailing each traffic signal pole, accompanying parts, and necessary hardware.
4) **Contractor Action:** Within 30 days after award, submit shop drawings to the contracting authority for engineer approval.
5) **Engineer Action:** Review the shop drawings in a timely manner. Check the appropriateness of each detail. Upon acceptance, sign and date the shop drawings and provide a copy to the contractor.

e. Catalog Cuts:

1) **Document:** Prepared by the traffic signal equipment supplier for the contractor.
2) **Purpose:** Contracting authority approval of all items within the equipment and materials list as well as for supporting components.
3) **Includes:** Catalog cut information detailing the make, model number, manufacturer, and specific details for all traffic signal equipment.
4) **Contractor Action:** Within 30 days after award, submit catalog cuts to the contracting authority for engineer approval.

5) **Engineer Action:** Review the catalog cuts in a timely manner. Check the appropriateness of each item. Upon acceptance, sign and date the catalog cut documents and provide a copy to the contractor.

2. **Substitutions:** Comply with SUDAS Specifications Division 1 - General Provisions and Covenants.

3. **Delivery, Storage, and Handling:** Comply with SUDAS Specifications Division 1 - General Provisions and Covenants.

4. **Scheduling and Conflicts:** Comply with SUDAS Specifications Division 1 - General Provisions and Covenants.

5. **Special Requirements:** Comply with the current edition of the MUTCD as adopted by the Iowa DOT.

6. **Measurement and Payment:** Traffic signal work is typically bid as a lump sum item of which no measurements are made. However, partial payments to the contractor are established through measuring installed quantities and applying these quantities to the appropriate approved unit price (see Schedule of Unit Prices above).

B. Part 2 - Products

1. **Underground:**

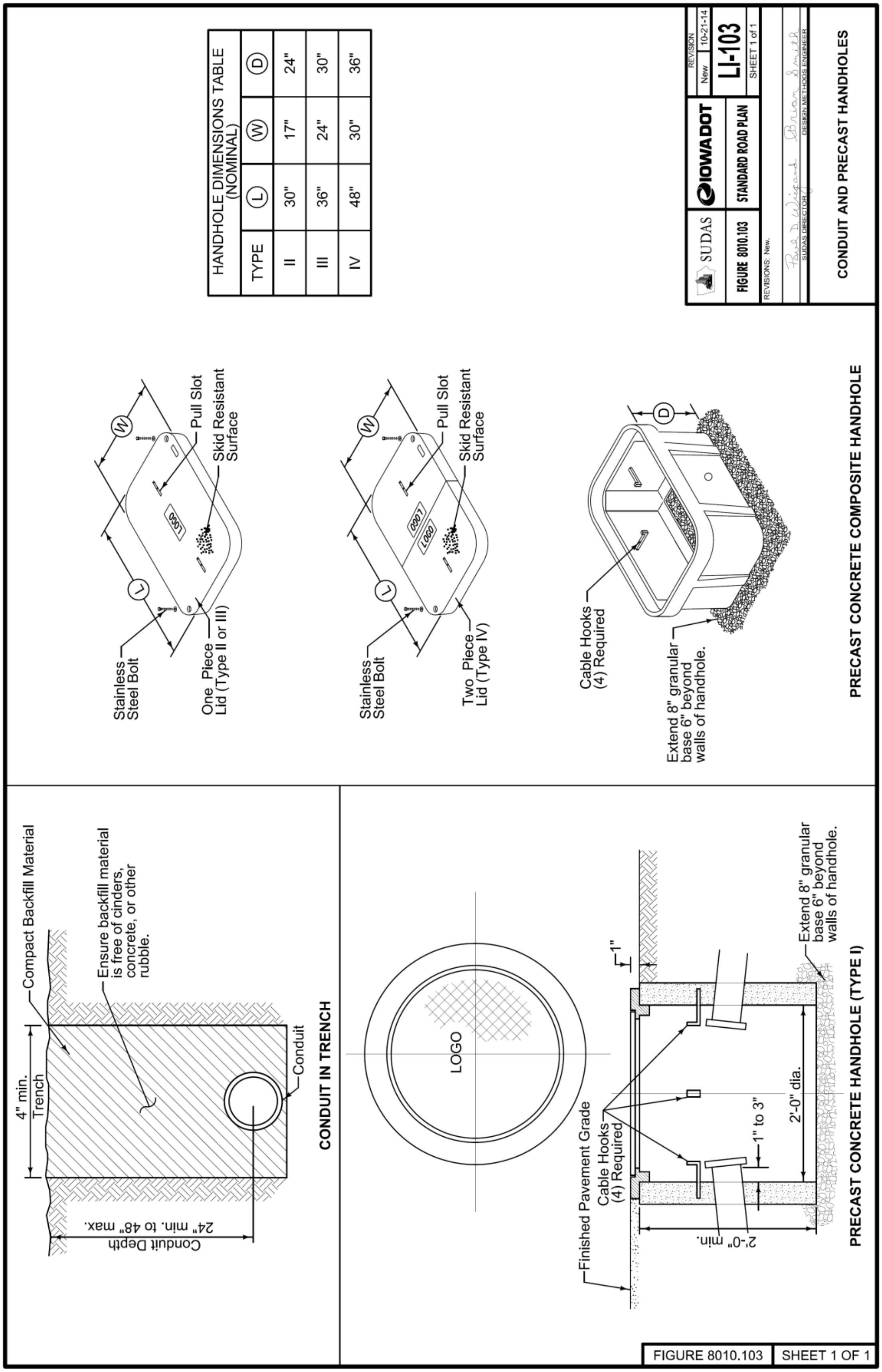
a. **Handhole:** Handholes are a critical component to traffic signal design. The standard precast concrete handhole shown in Figure 13E-1.01 is typically used at all locations except where fiber optic cables are used and adjacent to the controller cabinet.

Composite handholes can come in all shapes and sizes (see Quazite example table) and must be specified by the Engineer. These are typically made of a polymer concrete. Polymer concrete is made from selectively-graded aggregates in combination with a polymer resin system. When combined through a process of mixing, molding and curing, an extremely powerful cross-linked bond is formed. Precast polymer concrete is reinforced with fiberglass for strength and rigidity.

The designer should ensure that the contract documents clearly distinguish between handhole types, sizes, and desired locations. Handholes are typically uniquely numbered on the contract documents.

An online resource can be found through Chapter 12 - Handholes, Pulling Vaults, and Junction Boxes from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for considering handhole features and functions along with execution issues such as installation, inspection, and key points to remember.

Figure 13E-1.01: Conduit and Handholes
(SUDAS Specifications Figure 8010.103)



b. Conduit: The SUDAS Specifications allow both steel and PVC plastic conduit. Steel conduit is typically used on all service risers and plastic PVC or HDPE is used at all other locations. A typical signal installation will use a variety of conduit sizes. When connecting HDPE conduit to PVC conduit, the designer should work with the Contractor to clarify the method or materials to be used.

A conduit check list from Mn/DOT Signal Design Documents, Checklists, and Worksheets is noted below: The designer should ensure the following:

- Conduit size and cables listed.
- Correct symbol for in-place conduit.
- Correct symbol for proposed conduit.
- Check for conflict with in place underground utilities.
- Conduit fill less than 40% (Check).
- 3 inch RSC minimum size conduit under all public traveled roadways.
- Spare 4 inches of conduit out of controller cabinet for future use, threaded and capped.
- Conduit runs for interconnect should be as straight as possible.
- No PVC above ground (for example: bridge crossings and wood pole systems).
- All conduits except those within pads shall drain.
- Primary power shall be in a separate conduit run and separate hand holes.
- Size of bends and elbows in conduit in accordance with National Electrical Code or UL guidelines.
- If conduit is suspended under a bridge, does the distance between supports conform to code, is a hanger detail given in plan, and are expansion fittings called for?
- Conduit placed under in-place pavement does not need to be labeled (bored or pushed).

An online resource can be found through Chapter 11 - Conduits and Fittings from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for considering conduit installation and features.

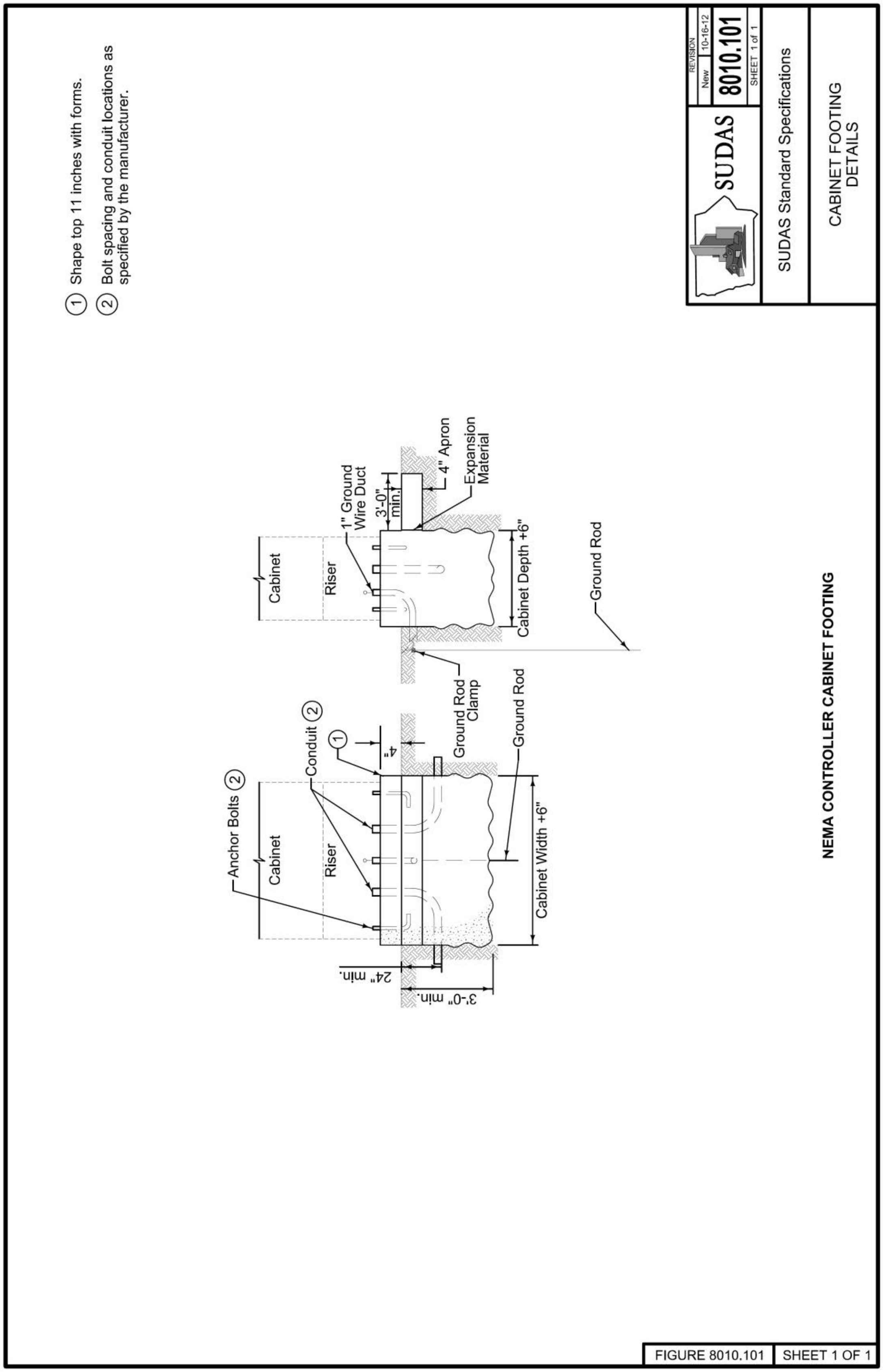
c. Wiring and Cable: Signalized intersections require a variety of standard wires and cables; however, the number, size, and quantity of extra conductors pulled can vary by agency. The designer should include sufficient details to ensure the clear identification of cable runs by conduit. The inspector should make sure all wires are terminated neatly and in an organized fashion. With the exception of detector lead-in wires, no splices are allowed within handholes. All plan terminology should be consistent for example:

- Cable symbols correct (3/C #12, 2/C #14, 3/C #20 all different, for example).
- Ped indications on different phases shall have separate 3/C #12 cables.
- Separate 2/C #14 for each detector.
- Provide spares for future expansion of system, if necessary, and label them.

An online resource can be found through Chapter 15 - Wiring from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for labeling and training wires (very Mn/DOT specific though).

d. Foundations: Signalized intersections require footings or foundations for all poles, controller pads, and other service cabinets such as fiber optic hubs or electrical service panels. Controller footing details are included for NEMA controller cabinets as shown in Figure 13E-1.02. The designer should ensure that the plans reflect any desired future use spare conduit stubs out of the foundation.

Figure 13E-1.02: Cabinet Footing Details
(SUDAS Specifications Figure 8010.101)



Foundation size and depths vary according to pole style, mast-arm length, and pole loadings. The SUDAS Specifications provide figures for both pedestal poles and for mast-arm poles (Figure 13E-1.04). SUDAS standard mast arm pole foundation designs (Table 13E-1.01 and Figure 13E-1.04) are based on the following guidelines, parameters, and assumptions:

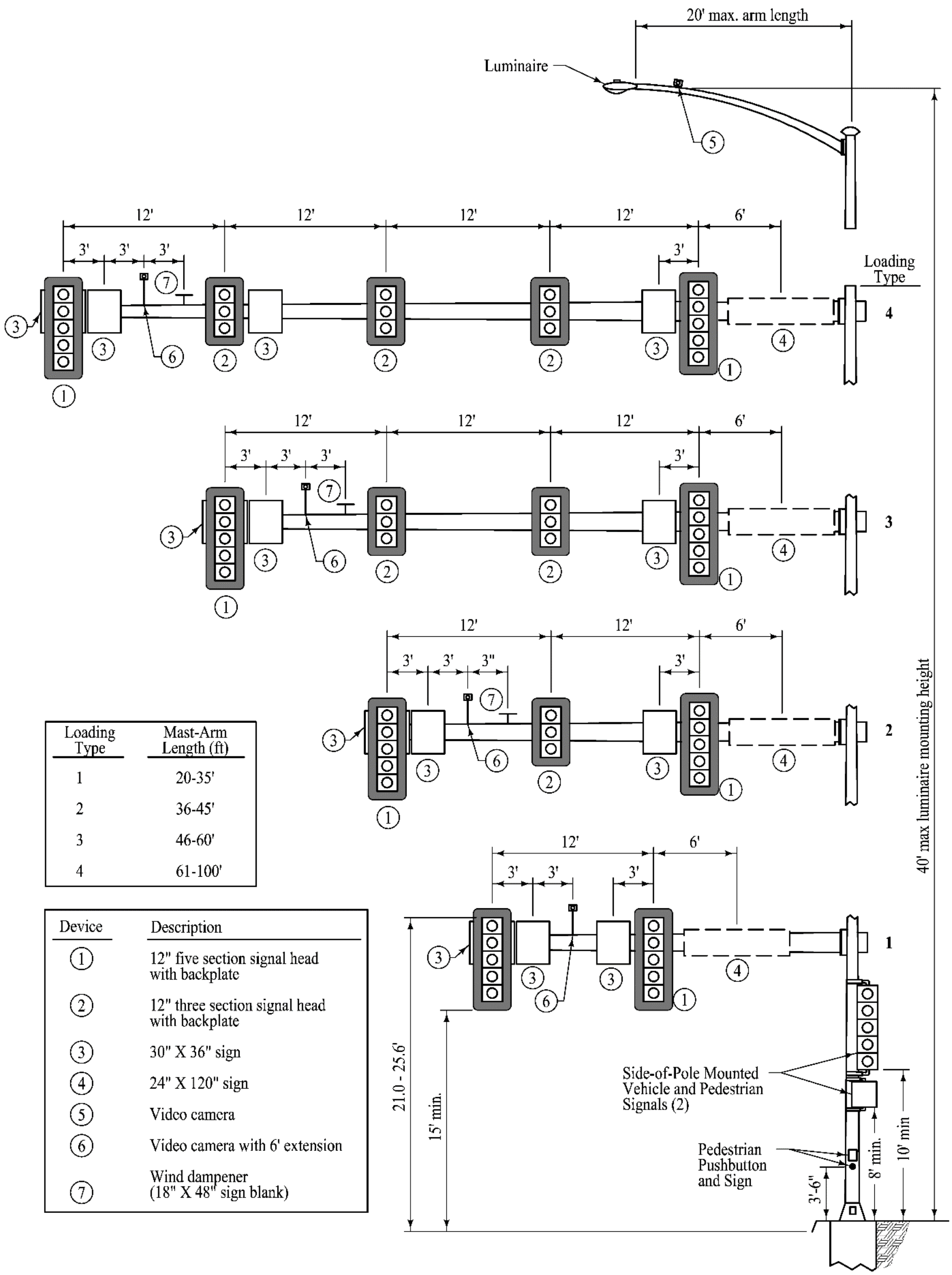
- Broms' method for lateral resistance (moment/shear design) per AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*, 6th Edition, 2013 (AASHTO LTS-6), with a safety factor of 2.86, which accounts for the possible under capacity of the soil strength (0.7) and the overload factor for the loadings (2.0).
- Alpha method for torsion design per FHWA-NHI-10-016 *Drilled Shafts: Construction Procedures and LRFD Design Methods*, May 2010, with a safety factor of 1.0.
- Disturbed soil due to frost: 2.5 feet for moment/shear design, 5.0 feet for torsion design. Broms' method as presented in AASHTO LTS-6 includes an additional 1.5 diameters of foundation length to be added to the minimum foundation length required. The maximum value of 1.5 diameters or 2.5 feet shall be used when determining the disturbed soil for moment/shear design.
- Groundwater is present for moment/shear and torsion designs.
- Pole loadings as shown in Figure 13E-1.03, with poles designed per AASHTO LTS-6 specifications. Basic wind speed equals 90 mph with a 50 year mean recurrence interval and gust effect factor of 1.14 for strength design. Use Category II for fatigue design. Apply only natural wind gust loads (i.e. do not apply galloping loads, vortex shedding loads, or truck-induced gust loads) for fatigue design. Install vibration mitigation devices on all traffic signal pole mast arms over 60 feet in length as shown in the figures.
- Cohesive soils along the length of the foundation with an average blow count (N60) greater than or equal to 8, which equates to an average unconfined compressive strength (Qu) greater than or equal to 2.0 kips per square foot.
- Reinforced concrete design per AASHTO LTS-6 specifications.

For pole loading conditions greater than shown in Figure 13E-1.03, granular soils, or lower strength soils, special foundation designs will be required. Soil boring testing should be performed prior to construction to verify soil types and strengths if non-typical soils are suspected.

Table 13E-1.01: Standard Mast Arm Pole Foundation Designs

Loading Type (Figure 13E-1.03)	Maximum Mast Arm Length (feet)
1	35
2	45
3	55
3	60
4	70
4	80
4	90
4	100

Figure 13E-1.03: Mast Arm Pole Loadings for Standard Foundation Designs



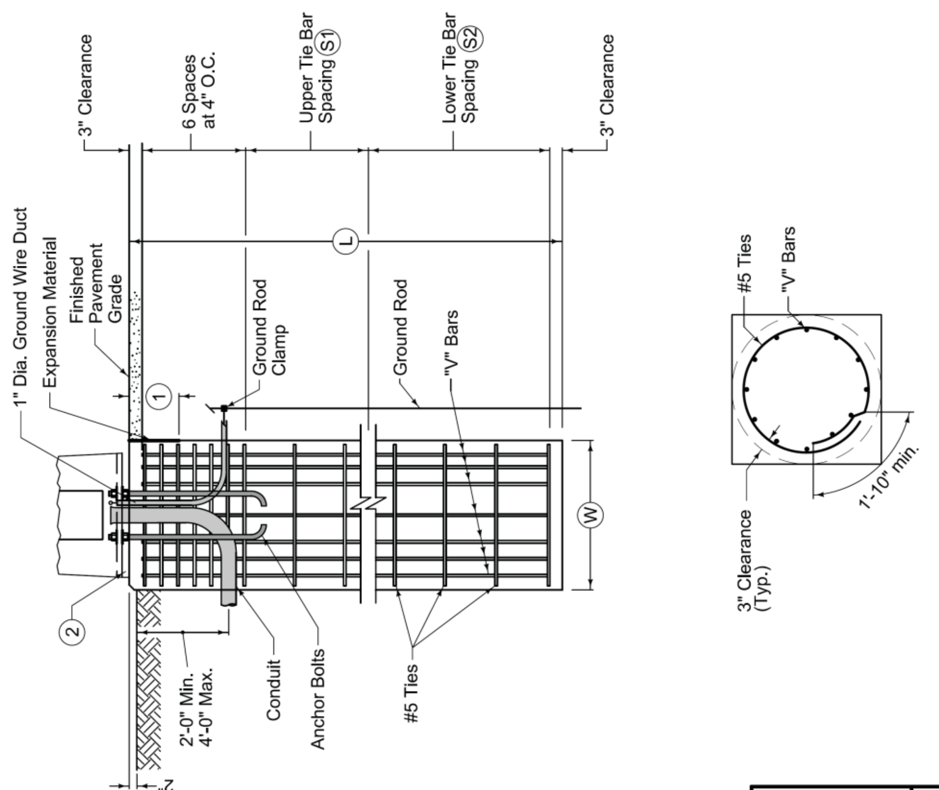
Loading Type	Mast-Arm Length (ft)
1	20-35'
2	36-45'
3	46-60'
4	61-100'

Device	Description
1	12" five section signal head with backplate
2	12" three section signal head with backplate
3	30" X 36" sign
4	24" X 120" sign
5	Video camera
6	Video camera with 6' extension
7	Wind dampener (18" X 48" sign blank)

Figure 13E-1.04: Pole Foundation Details
(SUDAS Specifications Figure 8010.102)

1. Shape top 11 inches with forms.
2. Install rodent guard or non-shrink grout with weep hole.

Max. Mast Arm Length	Foundation W	Foundation L	"V" Bars Count	"V" Bars Size	"V" Bars Length	Tie Bars Bar Count	Upper Spacing # Spaces	Upper Spacing S1	Lower Spacing # Spaces	Lower Spacing S2
35'-0"	3'-0"	12'-0"	12	8	11'-6"	17	9	12"	N/A	N/A
45'-0"	3'-0"	14'-0"	12	8	13'-6"	19	11	12"	N/A	N/A
55'-0"	3'-0"	16'-0"	12	8	15'-6"	25	12	8"	5	12"
60'-0"	3'-0"	18'-0"	13	8	17'-6"	28	15	8"	5	12"
70'-0"	3'-6"	18'-0"	12	10	17'-6"	28	15	8"	5	12"
80'-0"	3'-6"	21'-0"	14	10	20'-6"	40	22	6"	10	8"
90'-0"	4'-0"	22'-0"	16	10	21'-6"	42	24	6"	10	8"
100'-0"	4'-0"	24'-0"	18	10	23'-6"	47	32	6"	7	8"



SUDAS | IOWADOT | FIGURE 8010.102 | STANDARD ROAD PLAN | REVISION New 04-18-17 | TS-102 | SHEET 1 of 2

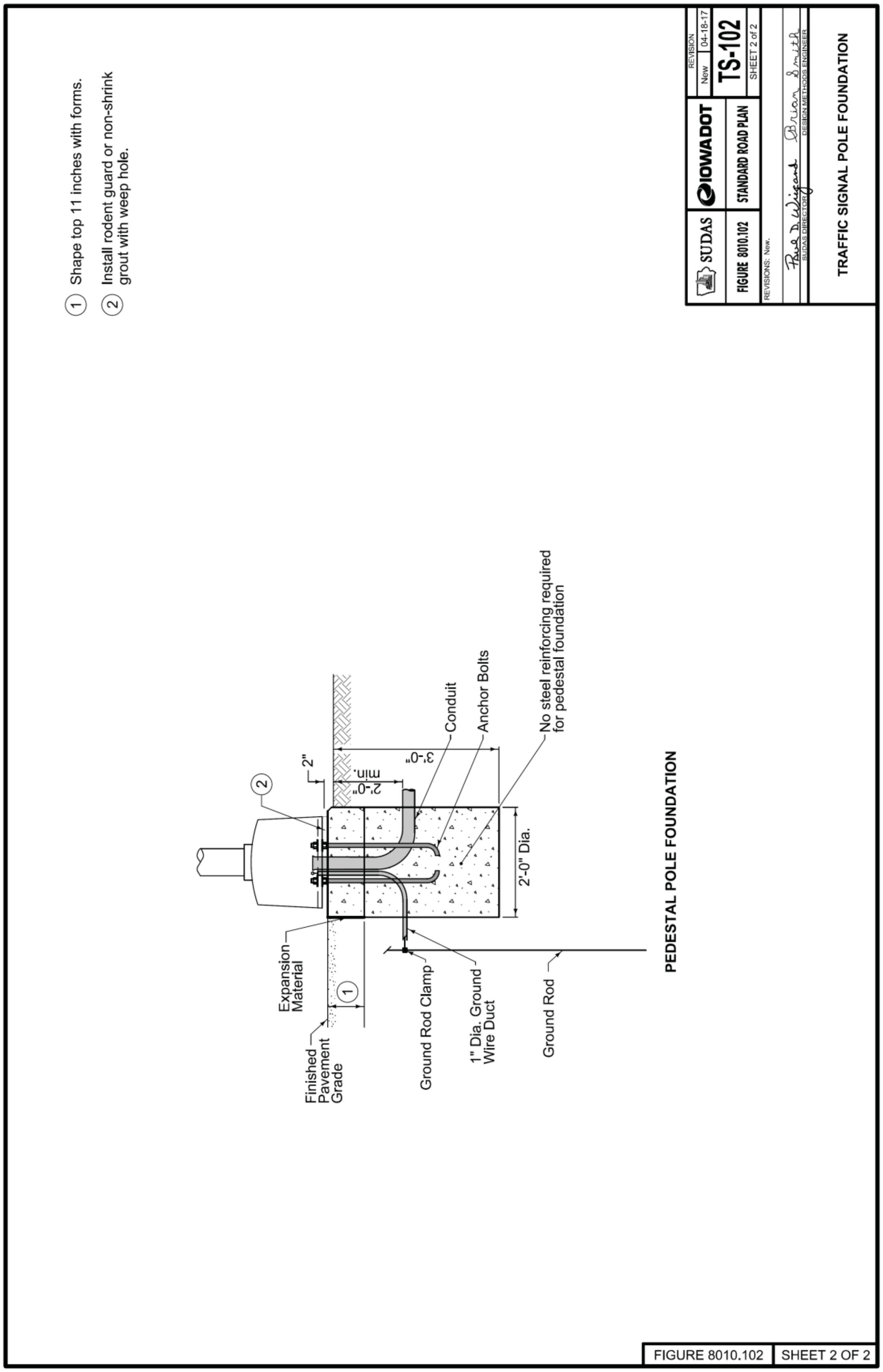
REVISIONS: New.

SUDAS DIRECTOR | DESIGN METHODS ENGINEER

TRAFFIC SIGNAL POLE FOUNDATION

FIGURE 8010.102 SHEET 1 OF 2

Figure 13E-1.04 (Continued): Pole Foundation Details
(SUDAS Specifications Figure 8010.102)



The designer should ensure that all foundations:

- Are located in compliance with applicable clear zone requirements
- Do not conflict with pedestrian walkways or ramps
- Are at the proper finish grade elevation

An online resource can be found through Chapter 10 - Foundations and Equipment Pads from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for foundation types and installation details.

e. Bonding and Grounding: All traffic signal installations must be bonded and grounded according to the National Electrical Code.

Bonding is defined in the Code Book as the permanent joining of metallic parts required to be electrically connected. In a traffic signal, the term is used to describe the electrical and mechanical connection of conduit, metal poles, cabinets, and service equipment.

Grounding is defined in the Code as a conducting connection, whether intentional or accidental, between an electrical circuit or equipment and the earth, or to some conductive body that serves in place of earth.

The designer should ensure that the contract documents include sufficient notation for the traffic signalized intersection to be properly bonded and grounded. This includes placing ground rods at each traffic signal pole and at the controller as well as through use of bonding and grounding jumpers within the handholes.

An online resource can be found through Chapter 13 - Grounding and Bonding from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for bonding and grounding details.

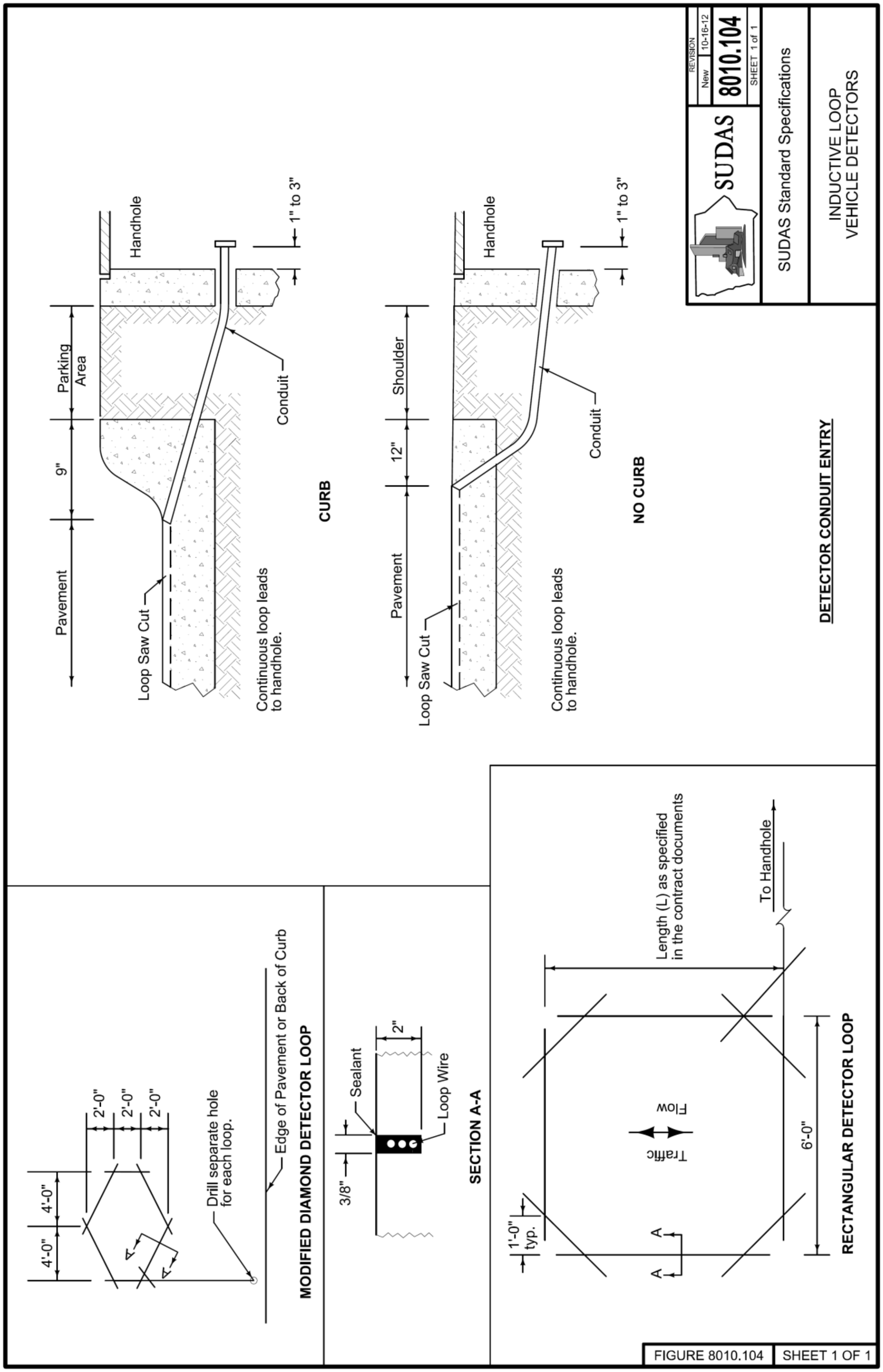
2. Detection: Detectors provide vehicle and pedestrian inputs to the traffic signal controller. Proper detector installation, operation, and maintenance is critical to the safe and efficient operation of any signalized intersection. An online resource to learn more about detection styles, modes, and typical layouts can be found within Chapter 9 - Traffic Signals from Mn/DOT's Traffic Engineering Manual. Since this document is a PDF, some of the information from this source is provided below.

Detector sizes and locations vary by agency and by location. SUDAS provides a standard drawing for a typical rectangular detector loop (Figure 13E-1.05).

An online resource can be found through Chapter 16 - Vehicle Detection from Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for installation and mounting details.

a. Inductive Loop Vehicle Detector: The most common type of vehicle detection device in use today is the inductive loop. This is a loop of wire imbedded in the pavement (saw cut in existing concrete or NMC loop in new concrete) carrying a small electrical current. When a large mass of ferrous metal passes over the loop, the magnetic field is disturbed and generates, or induces, a change in resonant frequency in the wire. This change in frequency is then recognized by the detector amplifier and signals the controller that a vehicle is present.

Figure 13E-1.05: Inductive Loop Vehicle Detectors
(SUDAS Specifications Figure 8010.104)



b. **Pedestrian Push Button Detector:** There are a number of ways to provide pedestrian actuation at a signalized intersection. The most common equipment used by far is the pedestrian pushbutton detector. Pressing the button provides a contact closure that actuates the call. There are plenty of examples of good and bad pedestrian pushbutton placement; however, part of the problem is getting the pedestrian to use the button. Specific information regarding pedestrian detectors can be found in the MUCTD Section 4E.08 Pedestrian Detectors.

   An online resource can be found through Chapter 19 - Accessible Pedestrian Signal Push Buttons from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for style, installation, and mounting details.

c. **Video Detection Camera System:** Vehicle detection by video cameras is a popular form of vehicle detection within Iowa. The rapid processing of video images provides the detection outputs to the controller. The designer should carefully consider the type of equipment necessary to provide video detection, the maintenance needs of this equipment, and the specific installation and mounting requirements necessary.

   Designers should consider relevant manufacturer recommendations and other online resources such as the Guidelines for Using Video Detection at Intersections and Interchanges by Bonneson at Texas Transportation Institute.

d. **Microwave Vehicle Detector:** Microwave detection is often used within Iowa during temporary signal control to provide simple, non-intrusive vehicle detection. A variety of styles and levels of sophistication exist in the market today.

3. **Communications:** The designer may be required to provide supplemental specifications for these items given the highly proprietary nature of this equipment and the needs of the contracting agency. Generic specifications have been provided in the SUDAS Specifications.

4. **Cabinet and Controller:** The designer may be required to provide supplemental specifications for the controller, cabinet, and emergency vehicle pre-emption system given the highly proprietary nature of this equipment. Generic specifications have been provided in the SUDAS Specifications. New information was added to the specifications regarding uninterruptable power supply battery back-up system. The designer should carefully consider the cabinet and mounting requirements of the battery back-up system.

   An online resource can be found through Chapter 22 - Traffic Signal Cabinets from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for style, installation, and mounting details.

5. **Poles, Heads, and Signs:**

   a. **Vehicle Traffic Signal Head Assembly:** Vehicle signal heads must comply with the following MUTCD sections:

      Section 4D.16 Number and Arrangement of Signal Sections in Vehicular Traffic Control Signal Faces
      Section 4D.17 Visibility, Shielding, and Positioning of Signal Faces
      Section 4D.18 Design, Illumination, and Color of Signal Sections

An online resource can be found through Chapter 18 - Signal Heads from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for style, installation, and mounting details.

b. Pedestrian Signal Head Assembly: Pedestrian vehicle signal heads must comply with the following MUTCD sections:

Section 4E.01 Pedestrian Signal Heads
Section 4E.02 Meaning of Pedestrian Signal Head Indications
Section 4E.03 Application of Pedestrian Signal Heads
Section 4E.04 Size, Design, and Illumination of Pedestrian Signal Head Indications
Section 4E.05 Location and Height of Pedestrian Signal Heads
Section 4E.06 Accessible Pedestrian Signals
Section 4E.07 Countdown Pedestrian Signals

c. Traffic Signal Poles and Mast Arms: Signalized intersections require poles and mast arms to achieve proper traffic signal and pedestrian head placement. Mast arm details and typical loadings are shown on Figure 13E-1.03; additional mast arm details are shown on Figure 13E-1.06. The designer should ensure that the plan locations comply with all clear zone, sight restriction, and pedestrian flow criteria. Vertical clearance to overhead utility lines is a constant issue that designers should take note of during pre-design field activities. Although the minimum height from the pavement to the bottom of the signal housing is 15 feet, the designer should consider the street classification and the volume of large trucks in establishing the signal height above the pavement. However, the top of the signal housing cannot exceed 25.6 feet above the pavement. If the project being designed has specific requirements relative to the elevation of the end of the mast arm in relation to the connecting point on the vertical pole, include those requirements in the special provisions of the contract documents.

An online resource can be found through Chapter 17 - Mast Arm Poles and Pedestals from Mn/DOT's Lighting and Signal Certification Field Guide, which provides the designer with a photographic resource for style, installation, and mounting details.

d. Traffic Signal Pedestal Poles: Pedestal poles provide alternate mounting heights for signal and pedestrian heads and are much easier to locate within a tight right-of-way. Pedestal pole details and typical head mounting information are shown in Figure13E-1.07.

e. Traffic Signs: The designer must ensure that all signs comply with Iowa DOT standards and the MUTCD.

Figure 13E-1.06: Mast Arm Pole Details
(SUDAS Specifications Figure 8010.105)

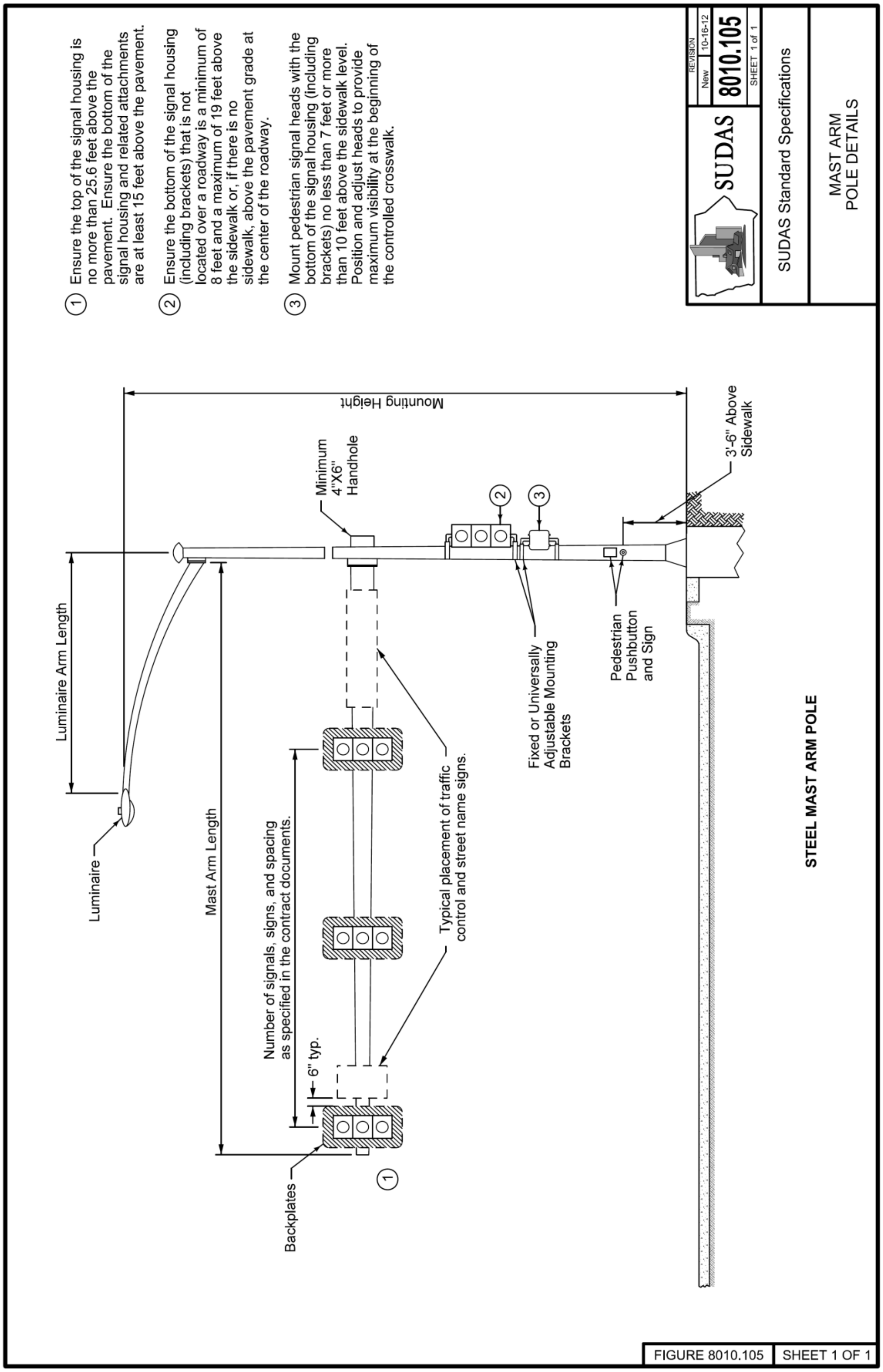
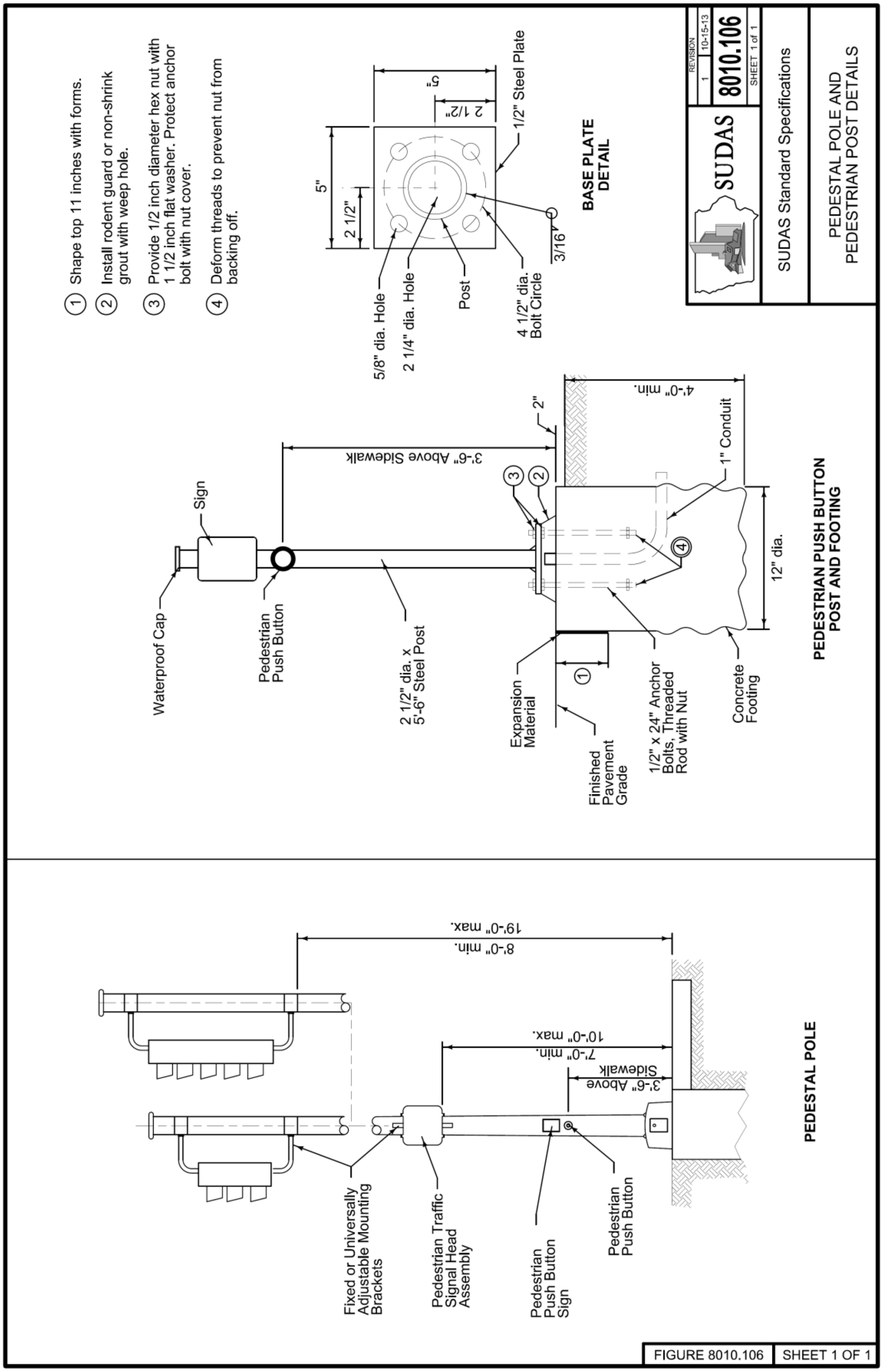


Figure 13E-1.07: Pedestal Pole and Pedestrian Post Details
(SUDAS Specifications Figure 8010.106)



C. Items Requiring Supplemental Specifications

A summary listing of items within SUDAS Specifications Section 8010 requiring supplemental specifications to be provided by the designer includes the following:

- Composite handhole and cover - specify materials and dimensions.
- Foundations - specify dimensions and any conduit stubs needed for future use.
- Communications - specify all traffic monitoring equipment along with any fiber optic equipment and materials.
- Cabinet, controller, and emergency vehicle preemption - specify all relevant equipment.
- Traffic signal poles and mast arms - specify specialty finish for pole if necessary.
- Traffic signs - specify sheeting, sign dimensions, and mounting requirements.

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CHAPTER 14
Trenchless Construction

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Chapter 14 - Trenchless Construction

14A General Information

14B New Construction

14C Rehabilitation

14C Rehabilitation (Continued)

14D References

General Information

A. Purpose

The purpose of this chapter is to discuss the various trenchless methods of construction and rehabilitation. This chapter does not describe each trenchless method in great detail; rather, it provides the designer with a general description of the various construction methods available and the applications and limitations of each method. The goal of this chapter is to educate the designer to recognize situations where a trenchless construction method may be preferred over open cut construction.

B. When to Consider Trenchless Construction

There are many situations where trenchless construction is preferable to open cut construction. The most common are for road, railroad, and river crossings. However, other situations may be suitable for trenchless construction, such as avoiding possible damage to adjacent structures, homes, and businesses caused by dewatering operations; installations in tight corridors; and minimizing disturbances in environmentally sensitive areas.

C. Cost Analysis of Trenchless vs. Open Cut Construction

Most trenchless construction methods have a higher dollar cost than that of their open cut counterparts. However, one needs to consider the benefits that trenchless construction provides and weigh all of the costs before deciding against using a trenchless technique. It is easy to determine the cost of the tangible work items that trenchless construction avoids, such as pavement removal and replacement, dewatering, surface restoration, right-of-way, or utility easement acquisition. However, the benefits of trenchless construction go well beyond these items and include avoiding public inconvenience and lost business revenue caused by a closed roadway; minimizing utility conflicts; reducing dust, erosion, vibration, tree removal, and other environmental impacts; eliminating danger to workers and the public posed by an open trench; and reducing the potential damage to adjacent structures caused by large scale dewatering operations. Unfortunately, it is difficult to assign a dollar value to these potential situations.

D. Definitions

Annular Space: Free space between the existing pipe and any lining.

Auger Boring: *(See also guided auger boring)* A technique for forming a bore from a drive pit to a reception pit, by means of a rotating cutting head. Spoil is removed back to the drive shaft by helically wound auger flights rotating in a steel casing. The equipment may have limited steering capability.

Back Reamer: A cutting head attached to the leading end of a drill string to enlarge the pilot bore during a pull-back operation to enable the carrier or sleeve or casing to be installed in.

Bent Sub: An offset section of drill stem close behind the drill head that allows steering corrections to be made by rotation of the drill string to orientate the cutting head. Frequently used in directional drilling.

Bentonite: *(See also drilling fluid)* A colloidal clay sold under various trade names that forms a slick slurry or gel when water is added. Also known as drillers mud.

Boring: (1) The dislodging or displacement of spoil by a rotating auger or drill string to produce a hole called a bore. (2) An earth-drilling process used for installing conduits or pipelines. (3) Obtaining soil samples for evaluation and testing.

Boring Pit: An excavation in the earth of specified length and width for placing the machine on line and grade.

Butt Fusion Weld: A method of joining polyethylene pipe where two pipe ends and rapidly brought together under pressure to form a homogeneous bond.

Carrier Pipe: The tube which carries the product being transported and which may go through casings at highway and railroad crossings. It may be made of steel, concrete, clay, plastic, ductile iron, or other materials. On occasion it may be bored direct under the highways and railroads.

Cased Bore: A bore in which a pipe, usually a steel sleeve, is inserted simultaneously with the boring operation. Usually associated with auger boring or pipe jacking.

Casing: A pipe used to line bore holes through which a pipe(s) called carrier pipes or ducts are installed. Usually not a Product Pipe.

Closed Face: The ability of a tunnel boring machine to close or seal the facial opening of the machine to prevent or slow the entrance of soils into the machine. Also may be the bulkheading of a hand dug tunnel to slow or stop the inflow of material.

Closed-circuit Television Inspection (CCTV): Inspection method utilizing a closed circuit television camera system with appropriate transport and lighting mechanisms to view the interior surface of sewer pipes and structures.

Creep: The dimensional change, with time, of a material under continuously applied stress after the initial elastic deformation.

Cured-in-place Pipe (CIPP): A lining system in which a thin flexible tube of polymer or glass fiber fabric is impregnated with thermoset resin and expanded by means of fluid pressure into position on the inner wall of a defective pipeline before curing the resin to harden the material. The uncured material may be installed by winch or inverted by water or air pressure, with or without the aid of a turning belt.

Deformed Reformed Pipe (DRP): A term used to describe some systems in which the liner is deformed to reduce its size during insertion, and then reverted to its original shape by the application of pressure and/or heat.

Directional Drilling: A steerable system for the installation of pipes, conduits, and cables in a shallow arc using a surface launched drilling rig. Traditionally, the term applies to large scale crossings in which a fluid-filled pilot bore is drilled using a fluid-driven motor at the end of a bend-sub, and is then enlarged with a back reamer to the size required for the product pipe. The required deviation during pilot boring is provided by the positioning of a bent sub. Tracking of the drill string is achieved by the use of a downhole survey tool.

Drill String: 1) The total length of drill rods/pipe, bit, swivel joint, etc. in a drill borehole. 2) System of rods used with cutting bit or compaction bit attached to the drive chuck.

Drilling Fluid/Mud: A mixture of water and usually bentonite and/or polymer continuously pumped to the Cutting Head to facilitate cutting, reduce required torque, facilitate the removal of cuttings, stabilize the borehole, cool the head, and lubricate the installation of the Product Pipe. In suitable soil conditions, water alone may be used.

Duckbill: Alternative name for the steering device attached to the front of a directional drilling string.

Elastic Modulus: A measure of the stress buildup associated with a given strain.

Face: Wall of the entrance pit into which the bore is made.

Flexural Modulus of Elasticity: Mathematically defined as the stress divided by the strain of the material; measure of the rigidity or stiffness of a material. A high flexural modulus indicates a stiffer material.

Flexural Strength: The strength of a material in bending expressed as the tensile stress of the outermost fibers at the instant of failure.

Fold and Form Pipe: A pipe rehabilitation method where a plastic pipe manufactured in a folded shape of reduced cross-sectional area is pulled into an existing conduit and subsequently expanded with pressure and heat. The reformed plastic pipe fits snugly to and takes the shape of the ID of the host pipe.

Guided Auger Boring: A term applied to auger boring systems, which are similar to microtunneling, but with the guidance mechanism actuator sited in the drive shaft (e.g. a hydraulic wrench that turns a steel casing with an asymmetric face at the cutting head). The term may also be applied to those auger boring systems with rudimentary articulation of the casing near the head activated by rods from the drive pit.

Impact Moling: Method of creating a bore using a pneumatic or hydraulic hammer within a casing, generally of torpedo shape. The term is usually associated with non-steered or limited steering devices without rigid attachment to the launch pit, relying upon the resistance of the ground for forward movement. During the operation the soil is displaced, not removed. An unsupported bore may be formed in suitable ground, or a pipe drawn in, or pushed in, behind the impact moling tool. Cables may also be drawn in.

Inversion: The process of turning a resin-saturated tube inside out by application of air or water pressure.

Jacking Frame: A structural component that houses the hydraulic cylinders used to propel the microtunneling machine and pipeline. The jacking frame serves to distribute the thrust load to the pipeline and the reaction load to the shaft wall or thrust wall.

Jacking Shield: A fabricated steel cylinder from within which the excavation is carried out either by hand or machine. Incorporated within the shield are facilities to allow it to be adjusted to control line and grade.

Launch Pit: Also known as Drive Pit, but more usually associated with "launching" an Impact Moling tool.

Liner Plate: A product used to line tunnels instead of casing, and comes in formed steel segments. When these segments are bolted together, they form a structural tube to protect the tunnel from collapsing. The segments are made so that they may be bolted together from inside the tunnel.

Microtunneling: A trenchless construction method for installing pipelines with the following features: (1) Remote controlled **-** The microtunneling boring machine (MTBM) is operated from a control panel, normally located on the surface. Personnel entry is not required for routine operation. (2) Guided **-** The guidance system usually references a laser beam projected onto a target in the MTBM. (3) Pipe jacked **-** The process of constructing a pipeline by consecutively pushing pipes and MTBM through the ground using a jacking system for thrust. (4) Continuously supported – Continuous pressure is provided to the face of the excavation to balance groundwater and earth pressures.

Mixed Face: A soil condition that presents two or more different types of material in the path of the bore.

Modulus of Elasticity (E): The stress required to produce strain, which may be a change of length (Young's modulus); a twist or shear (modulus or rigidity); or a change of volume (bulk modulus), expressed in dynes per square centimeter.

Open Cut: *(See also conventional trenching)* The method by which access is gained to the required level underground for the installation, maintenance or inspection of a pipe, conduit or cable. The excavation is then backfilled and the surface restored.

Open Face Shield: Shield in which manual excavation is carried out from within a steel tube at the front of a pipe jack.

Ovality: The degree of deviation from perfect circularity, or roundness, of the cross section of a pipe.

Pilot Bore: The action of creating the first (usually steerable) pass of any boring process which later requires back-reaming or similar enlarging. Most commonly applied to Guided Boring, Directional Drilling, and 2-pass microtunneling systems.

Pipe Bursting: A replacement method, also known as Pipe Cracking and Pipe Splitting. A technique for breaking the existing pipe by brittle fracture, using force from within, applied mechanically, with the remains being forced into the surrounding ground. At the same time, a new pipe, of the same or larger diameter, is drawn in behind the bursting tool. The pipe bursting device may be based on an Impact Moling tool to exert diverted forward thrust to the radial bursting effect required, or by a hydraulic device inserted into the pipe and expanded to exert direct radial force. Generally, a PVC or HDPE pipe is used.

Pipe Eating: A replacement technique, usually based on microtunneling, in which a defective pipe is excavated together with the surrounding soil as for a new installation. The microtunneling shield machine will usually need some crushing capability to perform effectively. The defective pipe may be filled with grout to improve steering performance. Alternatively, some systems employ a proboscis device to seal the pipe in front used of the shield to collect and divert the existing flow, thus allowing a sewer, for example, to remain "live."

Pipe Jacking: A system of directly installing pipes behind a shield machine by hydraulic jacking from a drive shaft such that the pipes form a continuous string in the ground.

Pipe Ramming: A non-steerable system of forming a bore by driving an open-ended steel casing using a percussion hammer from a Launch Pit. The soil may be removed from the casing by augering, jetting, or with compressed air.

Pipe Splitting: Replacement method for breaking an existing pipe by longitudinal slitting. At the same time, a new pipe of the same or larger diameter may be drawn in behind the splitting tool. See also Pipe Bursting.

Ramming: A percussion hammer is attached to an open end casing, which is driven through the ground. The spoil within the casing is removed to leave an open casing.

Reception/Exit Shaft/Pit: Excavation into which trenchless technology equipment is driven and recovered following the installation of the Product Pipe, conduit, or cable.

Resin Impregnation (Wet-out): A process used in cured-in-place pipe installation where a plastic coated fabric tube is uniformly saturated with a liquid thermosetting resin while air is removed from the coated tube by means of vacuum suction.

Resins: An organic polymer, solid or liquid; usually thermoplastic or thermosetting.

Shield: A steel cylinder at the face of a utility tunnel or casing, which may sometimes employ the use of a mechanical excavator and may be steerable, and provide hazard protection from the area covered.

Sliplining: (1) General term used to describe methods of lining with continuous pipes and lining with discrete pipes. (2) Insertion of a new pipe by pulling or pushing it into the existing pipe and grouting the annular space. The pipe used may be continuous or a string of discrete pipes. This latter is also referred to as Segmental Sliplining.

Slurry: A fluid, normally water, used in a closed loop system for the removal of spoil and for the balance of groundwater pressure during microtunneling.

Swab (Bull Plug): A steel plug that is pulled through a horizontal bore to remove the cuttings.

Thermoset: A material, such as epoxies, that will undergo or has undergone a chemical reaction by the action of heat, chemical catalyst, ultraviolet light, etc., leading to an infusible state.

Trenchless Technology: Techniques for utility line installation, replacement, rehabilitation, renovation, repair, inspection, location and leak detection, with minimum excavation from the ground surface.

Tunnel Boring Machine (TBM): (1) A full-face circular mechanized shield machine, usually of Man-Entry diameter, steerable and with a rotary cutting head. For pipe installation, it leads a string of jacked pipes. It may be controlled from within the shield or remotely. (2) (Mole, Tunneling Head) A mechanical excavator used in a tunnel to excavate the front face of the tunnel.

Tunneling: A construction method of excavating an opening beneath the ground without continuous disturbance of the ground surface and of large-enough diameter to allow individuals access and erection of a ground support system at the location of material excavation.

Upsizing: Any method that increases the cross sectional area of an existing pipeline by replacing with a larger diameter pipe.

Utility Tunneling: A process in which a temporary support liner is constructed as the tunnel is excavated. The liner typically consists of steel or concrete liner plates, steel ribs with wood lagging, or an all wood box culvert. Personnel are required inside the tunnel to perform the excavation and/or spoil removal.

Wing Cutters: Appendages on cutting heads that will open to increase the cutting diameter of the head when turned in a forward direction, and close when turned in a reverse direction. They are used to cut clearance for the casing pipe.

Definitions Source: North American Society for Trenchless Technology.

Commonly Used Trenchless Technologies

A. Techniques Common in the United States

Several trenchless technologies are commonly used in the United States. They are separated into two categories based on whether they are used most frequently for new construction or reconstruction.

New Construction:
- Auger Boring
- Compaction Boring
- Pipe Ramming
- Slurry Methods
- Horizontal Directional Drilling
- Pipe Jacking and utility tunneling
- Microtunneling

Reconstruction:
- Cured in place pipe
- Fold and formed pipe
- Sliplining
- Pipe Bursting

Each of these technologies is discussed in greater depth in Parts 14B and 14C.

B. Techniques Common in Iowa

A study was conducted by the Iowa Highway Research Board (IHRB) to determine which trenchless methods were the most prevalent within the State of Iowa. This study, included responses from both the public and private sector, determined which trenchless technologies are the most commonly used construction methods within the State of Iowa. The study indicated that horizontal directional drilling (HDD), auger boring, and pipe jacking are the most common new construction technologies used within Iowa. For rehabilitation work, cured-in-place pipe is the most commonly used technology in Iowa.

Planning a Bore

A. Bore Pit Locations

Careful consideration should be given to the location of the bore. Adequate room for launch and reception pits, if necessary, should be provided. Potential utility conflicts with the bore pit should be identified and addressed. Restricting the size of the bore pit or work area may affect the boring method that can be used. When unsure about the size of bore pit required for a particular technique, it is recommended that the designer contact a boring contractor for additional information.

B. Manhole Locations

When possible, manholes should be located at both ends of a long or difficult bore. This allows minor deviations in line or grade between the bore and the open cut section to be corrected. In addition, it provides access to both ends of the section of pipe for maintenance purposes.

C. Bore Lengths

The length of the bore specified needs to be carefully considered. Crossing under a 24 foot roadway requires a bore longer than 24 feet. Adequate length to protect roadway and foreslope from loss of support and sloughing during bore entry and exit is required. If possible, it is desirable to place the bore pit locations beyond the roadway foreslope. For roadways with an urban section, the bore pits should be located several feet away from the back of the curb to prevent undermining of the roadway.

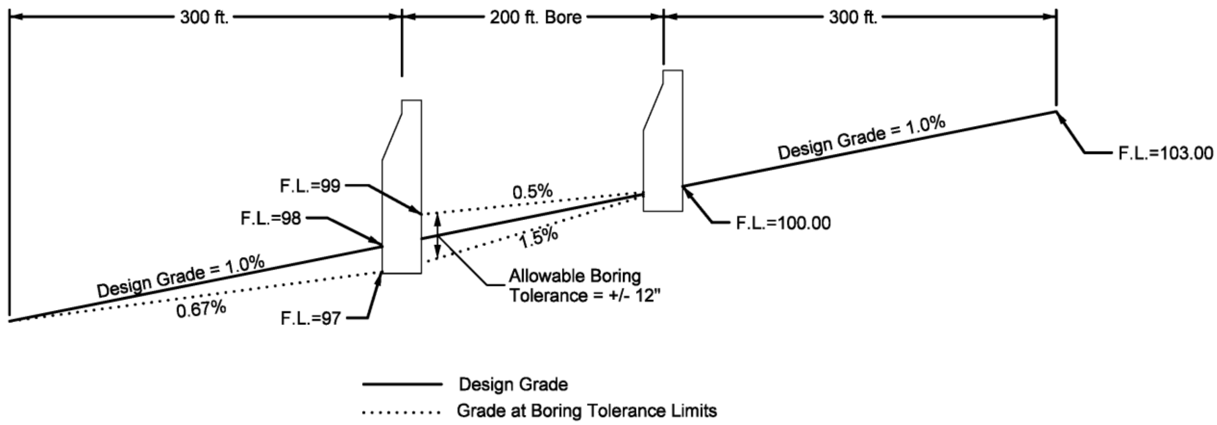
D. Acceptable Tolerances

The designer should recognize that as tolerance specifications tighten, the cost for boring will increase. Different trenchless methods have different tolerance limitations. Methods and machines that are able to meet tight tolerances tend to be more complex, and therefore, more costly. In addition, the contractor assumes a greater risk when agreeing to complete a bore with tight tolerance requirements.

Since every installation is unique, bore tolerances should be determined and specified on a case-by-case basis. Unless there are special circumstances, it would be unreasonable to require a water main or force main to meet the same tolerances as a gravity sewer line. In order to reduce costs, the designer should allow as much grade and alignment variation as possible while still meeting the operational requirements of the installation.

For example, assume a 12 inch sanitary sewer on a 1% grade is being bored for a length of 200 feet as shown in Figure 14B-1.01. Due to capacity/velocity limitations, the minimum allowable pipe slope is 0.5%. If the bore tolerances are set at ± 0.2% (common for gravity sewers) the project would likely require the use of a significantly oversized casing pipe with the auger boring technique to allow for adequate adjustment, or would need to be done by microtunneling in order to ensure compliance with the specifications. Increasing the allowable tolerances to ± 12 inches, would likely allow the steered auger boring method to be utilized, without an oversized casing pipe. This could result in significantly reduced boring costs, while still meeting the minimum grade requirements of the sewer line.

Figure 14B-1.01: Tolerance Considerations When Planning a Bore



For gravity sewers, which are laid at minimum grades, consideration should be given to providing additional slope through the length of the bore. While this may not always be possible, it helps reduce the potential for backfall in the pipe.

Often, the casing pipe may not meet the tolerances required in the specifications. However, the contractor normally has the ability to make grade corrections for the carrier pipe by using casing chocks. These chocks allow the position of the carrier pipe to be adjusted inside of the casing pipe as required to meet the specified grade. As mentioned above, for projects that require a high degree of accuracy, an oversized casing may be installed to allow additional maneuvering room inside the casing for the carrier pipe.

E. Information to Provide to Contractor

If soil borings were conducted, the soil boring log should be included with the specifications or at least be available upon request. The specifications should spell out in detail how unexpected circumstances will be handled. Will the contractor be entirely responsible if something goes wrong, or is there a risk allocation clause in place? In addition, the specifications should indicate what the tolerance requirements for the bore would be. Finally, the material requirements for the bore, including the casing pipe (see Section 9C-1), if required, should be indicated.

F. Risk Allocation

One of the factors that results in increased prices for tunneling and boring is the risk associated with the process. While soil borings and other information can provide a glimpse of the ground conditions that may be encountered, they do not provide the big picture. For example, a contractor may be nearing the end of a long bore when an unexpected large boulder or old foundation is struck. The only option may be to abandon the bore and begin again. Normally, the specifications place the costs associated with this upon the contractor. The boring contractors plan for these types of unexpected problems by increasing their bid prices to cover the costs associated with the additional work.

If the jurisdiction agrees to share in the costs that are associated with encountering differing site conditions, the contractor's risk is reduced, and they can lower their bid prices since they are no longer forced to "poke and hope."

By including a "differing site conditions" clause in the contract, the jurisdiction agrees to relieve the contractor from the burden of extraordinary costs required to complete its performance due to unexpected site conditions. This clause allows the contractor to negotiate an additional work order when the site conditions encountered are different than those reasonably expected.

While the actual clause used in the contract documents may vary, the following is commonly used text found in federal contracts 48 C.F.R § 52.236-2. The intent of this language is to provide for an equitable adjustment, as well as the procedures necessary for a contractor to make a claim for differing site conditions:

1. The Contractor shall promptly, and before the conditions are disturbed, give a written notice to the Jurisdiction of (a) subsurface or latent physical conditions at the site which differ materially from those indicated in this contract, or (b) unknown physical conditions at the site, of an unusual nature, which differ materially from those ordinarily encountered and generally recognized as inhering in work of the character provided for in the contract.

2. The Jurisdiction shall investigate the site conditions promptly after receiving the notice. If the conditions do materially so differ and cause an increase or decrease in the Contractor's cost of, or the time required for, performing any part of the work under this contract, whether or not changed as a result of the conditions, an equitable adjustment shall be made under this clause and the contract modified in writing accordingly.

3. No request by the Contractor for an equitable adjustment to the contract under this clause shall be allowed, unless the Contractor has given the written notice required; provided, that the time prescribed in (1) above for giving written notice may be extended by the Jurisdiction.

4. No request by the Contractor for an equitable adjustment to the contract for differing site conditions shall be allowed if made after final payment under this contract.

G. Drawing on Contractor’s Expertise

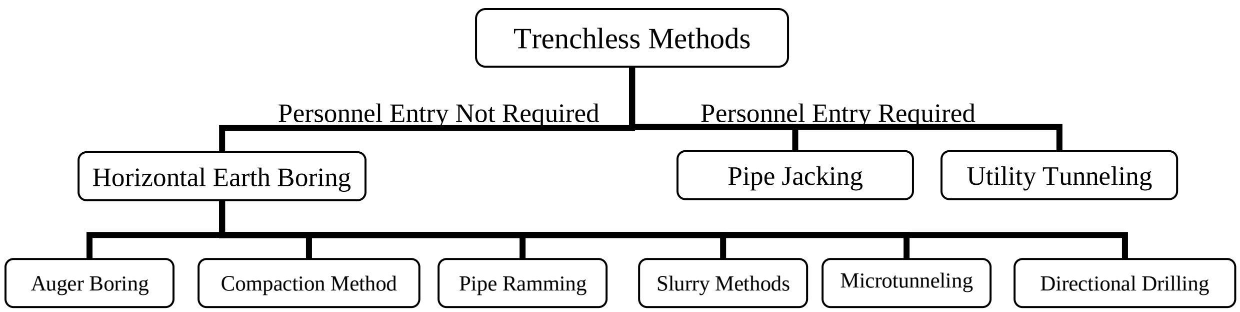
Once a tentative design has been laid out for a bore or tunnel, it may be prudent to discuss the proposed installation with an experienced tunneling and boring contractor. Most contractors are willing to discuss the practicality of a proposed installation, provide insight to potential problems, give a range of expected costs associated with the installation, and provide recommendations on how to reduce the overall cost of the project.

Methods of New Construction

A. Trenchless Methods for New Construction

The trenchless construction methods available for new facilities are divided into two main classes: Horizontal Earth Boring, which is performed without workers being inside the borehole, and Pipe Jacking / Utility Tunneling, which require workers inside the borehole during the excavation and casing processes. The chart below illustrates the various methods available.

Figure 14B-2.01: Classification of Trenchless Construction Methods



B. Expected Service Life for New Construction

The expected service life for a new pipe or other type of product installed by boring or tunneling is generally the same as a similar material installed by open cut methods. Due to the possibility of over-excavation, there may be some potential for surface settlement as the over-excavated bore settles around the casing pipe. This is especially true for shallow bores.

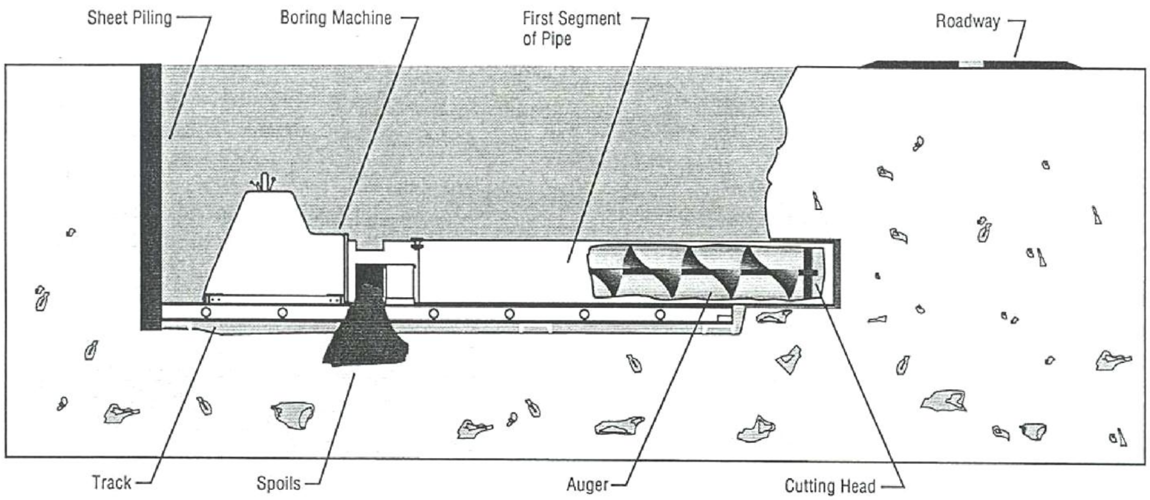
C. Auger Boring

1. **Description of Process and Equipment:** Auger boring is accomplished with an auger boring machine by jacking a casing pipe through the earth while at the same time removing earth spoil from the casing by means of a rotating auger inside the casing.

   The typical auger boring installation begins with the installation of bore pits at the beginning and end of the proposed bore. Bore pit dimensions vary depending on the size and length of the casing being used and on the depth of the boring. Generally, the length varies from 26 to 40 feet long and 8 to 12 feet wide. The bottom of the bore pit is usually over-excavated and backfilled with crushed stone in order to provide adequate support for the equipment.

   Most auger boring equipment is track mounted. The boring machine slides along this track in order to advance the casing pipe. The master track (on which the boring machine is set) is placed in the pit and set to the required line and grade of the bore. This is a critical step in the auger boring process since there is little ability to correct line or grade deviations once an auger bore is started. Steerable auger boring equipment is now common and does allow for some minor adjustment or bore direction as it progresses; however, proper setup is still critical.

Figure 14B-2.02: Typical Auger Boring Setup



Source: Isley and Gokhale, 1997

The boring machine applies thrust in order to advance the carrier pipe. This thrust is applied against the back of the boring pit with hydraulic rams. In order to withstand this thrust, a backing plate is normally installed against the back wall of the boring pit. This backing plate normally consists of steel piling, a steel plate, or wooden timbers. For long or large diameter bores, a concrete backstop may be used in addition to a steel plate.

After installation of the master track and backing plate, the auger boring machine is set on the master track. A cutting head, compatible with the soil conditions expected, is installed on the front of the first auger section. The first section of casing pipe may have a steel band welded around the top 3/4 of the outside diameter of the pipe. This process, called banding, slightly over-excavates the borehole, thereby reducing skin friction on the following casing sections.

The bore is begun by carefully installing the first section of casing pipe to the correct line and grade. After the first section has been installed and checked for accuracy, the boring machine is disconnected from the casing pipe and auger and slid to the rear of the bore pit. The next section of casing pipe and auger are lowered into position. The second auger section is coupled to the first with an auger pin. The two casing sections are lined up and either welded together, or an interlocking casing pipe jointing system may be utilized.

The bore is then advanced by applying thrust and simultaneously rotating the flight augers inside the casing in order to remove spoil. This process is repeated until the required length of casing is installed.

Once the bore is completed, the cutting head is removed at the receiving pit, and the augers are pulled out at the entrance pit, disconnected, and removed.

If required, a carrier pipe can be installed. The carrier pipe is attached to pre-manufactured casing chocks. The carrier pipe is pushed into the casing pipe using the auger boring machine, or by pulling it through with a winch.

Most auger bores are done without the ability to steer the bore, that is, make line or grade adjustments once the bore has begun. However, equipment and techniques are available that do allow minor corrections or changes in alignment to be made.

2. **Typical Applications/Materials:** The auger boring technique is used extensively throughout every segment of the United States, and it is the most common method used for crossing roadways with storm sewer, sanitary sewer, or water main pipes.

   In auger boring, the auger rotates inside the casing as it is being jacked. Consequently there is a danger that any interior pipe coatings may be damaged by the process. Due to the rotating augers and spoil removal process the interior of the casing pipe is subjected to during installation, the standard casing material used for auger boring is steel.

   Normally, a carrier pipe is installed inside of the casing pipe. This carrier pipe is protected by the structural rigidity of the steel casing and can therefore be almost any standard pipe material.

3. **Range of Applications:**

   a. **Pipe Sizes and Bore Lengths:** The most common pipe sizes installed by auger boring are from 8 inches to 36 inches. For sizes smaller than 8 inches, slurry and compaction methods are more suitable and economical, especially where the line and grade are not critical. For diameters larger than 36 inches, where the line and grade are more critical, pipe jacking with tunnel boring machines provide greater accuracy and safety and may be more cost effectively.

      This method was initially developed for bores between 40 and 70 feet, just long enough to cross under a two lane roadway. Since that time, advances in equipment capabilities have extended the range of this method. Typical bore lengths now ranges between 175 feet and 225 feet, with maximum bore lengths of greater than 600 feet possible.

   b. **Soil Conditions:** Auger boring methods can be used in a wide variety of soil conditions. However, soils with large boulders can cause problems with this method. Since the spoil is removed through the casing with an auger, any materials encountered must be able to fit between the auger flights in order to be carried out. In general, the largest boulder or other obstacle that this method can handle is limited to one third of the nominal casing diameter.

      In addition, auger boring in sandy, cohesionless soils can be difficult and may cause settlement, if not done properly, due to a loss of ground ahead of the bore as the soil flows into the pipe.

   c. **Tolerances:** The accuracy achievable with auger boring methods is usually ± 1% (both vertically and horizontally) of the length of the bore. Equipment, which allows the auger bore to be steered, is accurate within ± 0.1% vertically and ± 1.0% horizontally.

D. Compaction Boring

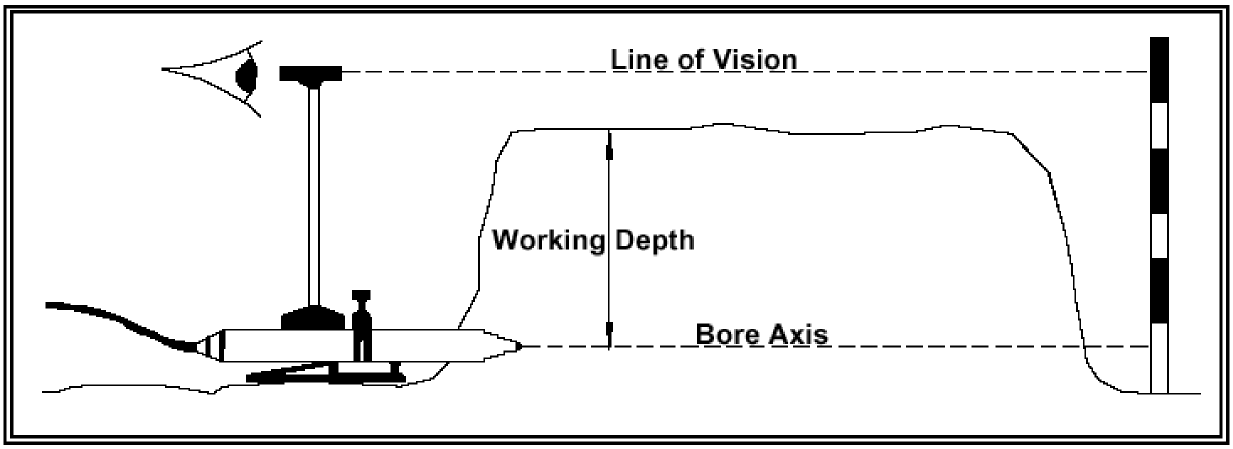
1. **Description of Process and Equipment:** Compaction boring is a method of forming a borehole by displacing and compacting the soil radially, rather than removing the soil. The compaction method can be divided into three sub classifications: the push rod method, the rotary method, and the percussion method.

   a. **Push Rod Method:** The first type of compaction boring, the push rod method, consists of a machine that pushes or pulls a solid rod through the soil by hydraulic force, simply displacing the soil. The resulting bore hole is the same diameter as the rod. Typical rod diameters range from 1 3/8 to 1 3/4 inches. To further enlarge the hole, the machine can pull a reamer back through the hole. Rods are usually about 4 feet in length and can be linked in series to achieve the desired bore length. Once the borehole is formed, a cable is used to pull the product into place.

b. Rotary Method: The rotary method is similar to the push rod method; however, the rod used is similar to a drill bit. It is rotated as it is forced horizontally through the soil.

c. Percussion Method (Impact Moling): The percussion method, also called impact moling, uses a self-propelled "mole" that is normally pneumatically powered. The mole is a torpedo shaped device that contains a reciprocating hammer in the nose. The action of this piston creates an impact force that propels the mole forward through the ground. Depending on ground conditions, the tools typically travel at a rate of 3 inches to 4 feet per minute. Once the mole exits into the receiving pit, the mole is removed and the air lines are used to pull a cable back through the borehole. This cable can then be used to pull the pipe product into place. If a rigid pipe is to be installed, it is simply pushed through the open borehole.

Figure 14B-2.03: Typical Percussion Setup



Source: Simicevic and Sterling, 2001

2. Typical Applications/Materials: Compaction boring methods are commonly used for installation of electric and communications cables, as well as gas lines, sprinkler irrigation systems, and water service lines. Since the boring process is independent of the pipe insertion process, almost any small diameter pipe or line can be installed by these methods.

3. Range of Applications:

a. Pipe Sizes and Bore Lengths: The size of product that can be installed by compaction methods is limited to the size of borehole that can be formed by the compaction method selected. The typical limit is 6 inches or less.

The size is further limited by the potential for ground disturbance above the bore. Since these compaction methods compress the surrounding soil and do not remove spoil, there is a potential for heaving. In order to avoid heaving problems, a rule of thumb to follow is to provide one foot of cover for every inch of bore diameter.

Due to the inaccuracy of the method and inability to control the direction of the bore, installation lengths are limited. The maximum practical bore length is typically 40 to 60 feet.

b. Soil Conditions: Since these methods compact the soil around the borehole, moderately soft to medium hard compressive soils are best suited for these methods.

Rocks, boulders, and other obstacles can affect the accuracy the bore and cause it to stray away from the desired course.

c. Tolerances: As mentioned previously, the accuracy of installation depends greatly on initial setup and ground characteristics. Once the bore is begun, there is no ability to control the direction of the bore. These methods are not normally used to install lines, which require a high degree of accuracy such as sanitary sewer lines.

4. Relative Cost vs. Other Trenchless Methods: Compaction methods for boring are highly economical. The equipment investment compared to other boring methods is very low. For short distance, small diameter bores, compaction methods are normally the lowest cost trenchless option available.

E. Pipe Ramming

1. Description of Process and Equipment: Pipe ramming is a trenchless method of installing a steel pipe or casing using a pneumatic tool to hammer the pipe or casing into the ground.

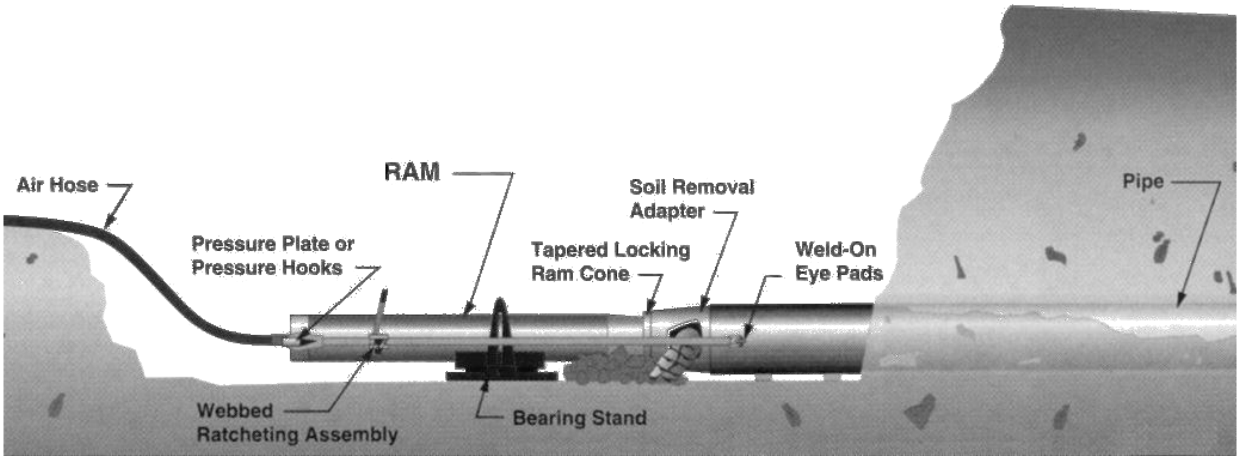
The pipe can be rammed with the leading edge either open or closed. Pipes up to 8 inches can be rammed with the end closed; however, this method is more difficult and is not normally recommended. When a closed end pipe is driven, the surrounding soil is displaced and compacted similar to the compaction methods previously described. This can result in ground heaving and is more susceptible to obstructions. More commonly, and always for larger diameters, the leading edge is left open. With the end of the pipe open, soil is allowed to enter the pipe during installation.

The lengths of pipe that can be rammed depend mainly on the space available at the site. If adequate room is available, the entire length of pipe can welded together prior to installation and rammed as a single unit. When the available area is restricted, the pipe can be rammed in short sections, welding them together as the bore progresses.

A typical Pipe ramming installation begins with the installation of bore pits at the beginning and end of the proposed bore. Guide rails are set to the line and grade of the proposed bore. The first length of steel pipe is prepared by attaching a steel band around the outside of the leading edge of the pipe. The purpose of this band is to slightly overexcavate the borehole, thus reducing friction on the following pipe sections. The first section of pipe is set in place, and the ramming hammer is attached to the rear of the pipe. The ramming hammer's percussion force drives the steel pipe into the ground along the line dictated by the guide rails. When one section of pipe has been driven, the hammer is removed and, if necessary, the next length of pipe is welded in place. This process is repeated until the leading edge of the pipe arrives at the receiving pit.

When using an open-ended pipe, a cylinder of ground, equal to the pipe diameter, is forced into the pipe as the bore advances. Once the bore is completed, this spoil must be removed. There are several methods available to remove this spoil including auger, compressed air, or water jetting.

Figure 14B-2.04: Typical Open-face Pipe Ramming Setup



Source: ACCU-Pipe Ramming Systems Inc.

2. **Typical Applications/Materials:** This method is frequently used under railway and road embankments for installation of medium to large diameter pipes.

   Steel pipe is used for the casing, as no other material is strong enough to withstand the impact forces generated by the hammer. Upon completion, a carrier pipe can be installed inside of the steel casing pipe. This carrier pipe is protected by the structural rigidity of the steel casing and can, therefore, be almost any standard pipe material.

3. **Range of Applications:**

   a. **Pipe Sizes and Bore Lengths:** Common pipe sizes installed by ramming range from 2 inches to 55 inches; however, pipe sizes as large as 147 inches have been done.

      Pipe ramming is typically used for pipe installations over relatively short distances, usually less than 150 feet. However, lengths as long as 300 feet may be successfully installed.

   b. **Soil Conditions:** Pipe ramming can be used in almost all types of soil conditions except solid rock. Pipe ramming is generally more successful than auger boring in rocky ground, as the leading edge and percussion force tends to act as a splitter to fracture the cobbles that are encountered.

   c. **Tolerances:** Pipe ramming is a non-steerable method. Once the bore has begun, there is little control over the line and grade of the installation. Soil conditions and ground obstructions such as rocks and cobbles can cause the bore to stray from the intended line and grade. The accuracy of the pipe ramming method is usually better than ± 1% (both vertically and horizontally) of the length of the bore.

4. **Relative Cost vs. Other Trenchless Methods:** Compared to other trenchless methods such as augering and directional drilling, pipe ramming can save both installation time and costs under appropriate conditions. Installation time may be shorter than for augering because the required bore pits are smaller and actual installation is faster. Pipe ramming is generally less costly than directional boring for short bores of 60 feet or less; however, directional drilling is generally better suited for longer bores.

F. Slurry Methods

1. **Description of Process and Equipment:** Slurry methods involve the use of a drilling fluid, such as water or bentonite slurry to aid in the drilling process and spoil removal.

   Slurry methods can be divided into two classifications: slurry boring and water jetting.

   a. **Slurry Boring:** Slurry boring normally begins by constructing a bore pit. The boring machine is set in the pit and adjusted to the appropriate line and grade. A pilot hole is formed by advancing drill tubing, with a drill bit attached to the end, through the ground. As the bit is advanced, drilling fluid is pumped through the tubing to the drill bit in order to lubricate the pilot drill and reduce the friction created by the advancing bore. Once the pilot bore reaches the receiving pit, a back reamer can be pulled or a forward reamer can be pushed through the ground to increase the bore to the required diameter.

      As the reamer is forced through the ground, drilling fluid is pumped into the bore. Depending on soil type, this drilling fluid may be either water or a bentonite mixture. The soil is mechanically cut by the reamer and mixed with the drilling fluid. These cuttings are held in suspension forming a slurry. This slurry helps prevent the uncased borehole from collapsing by exerting hydrostatic pressure against the walls of the bore.

      After the reaming process is completed, the bore is swabbed by pulling a plug through the bore, thereby forcing the slurry and cuttings out of the borehole. A casing pipe is inserted in conjunction with, or shortly after, the swabbing process.

      The size of the borehole is larger than the outside diameter of the casing pipe. The void between the pipe and the bore should be filled with grout to prevent ground settlement above the bore.

   b. **Water Jetting:** Another method of forming a borehole is the water jetting method. As the name implies, water jetting relies on a high speed jet of water to liquefy and remove soil. A special nozzle is attached to the end of a rod and extended forward into the bore. The jet of high-pressure water is used to perform all of the cutting and to wash the cuttings out of the bore. There is little ability to control the direction of the bore, since the jet of water will follow the path of least resistance. There is also little control over the amount of material excavated by the process and over-excavation is inevitable. Over time, this over-excavation will cause ground settlement. Due to these reasons, water jetting is rarely allowed by most jurisdictions.

      A distinction between water jetting and slurry boring should be made. Water jetting uses the force of water to erode the borehole and, therefore, is generally not recommended. Slurry boring is a mechanical process, and jetting of the soil should not occur if performed correctly.

2. **Typical Applications/Materials:** Since the boring process is independent of the pipe insertion process, almost any type of casing or carrier pipe can be installed by slurry methods.

   Slurry methods are most commonly used for placing non-gravity flow installations. Gravity flow sewers with sufficient grade may be successfully installed by this process; however, there may be some difficulty maintaining a straight grade alignment due to the tendency of the bore head to drop as the bore advances.

   Some jurisdictions may prohibit the use of slurry methods because, until the casing pipe is inserted, the bore is unsupported.

3. **Range of Applications:**

   a. **Pipe Sizes and Bore Lengths:** This method is most commonly used for small diameter bores. Pipe sizes between 2 inches to 12 inches are the most common.

      Due to the inability to steer the bore head, slurry methods are typically used for relatively short bores ranging from 40 to 75 feet.

   b. **Soil Conditions:** While the process for completing the bore may vary based upon the soil types, slurry boring can be utilized under most ground conditions.

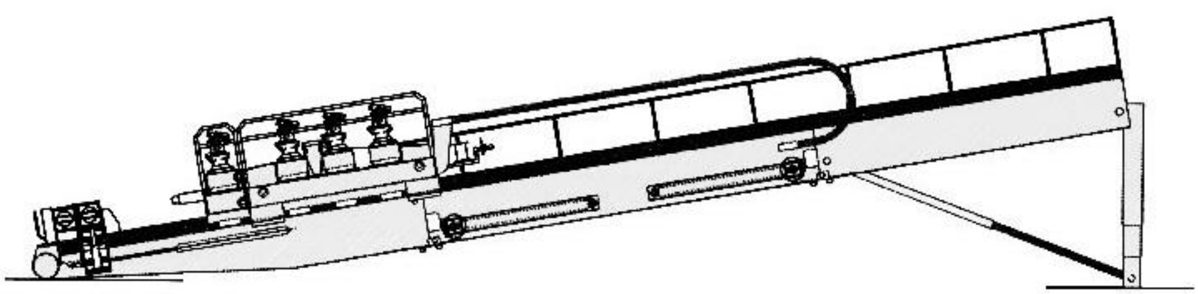
   c. **Tolerances:** Slurry boring is a non-steerable method that depends greatly on the operator's skill. For stable, homogeneous soil conditions, bores up to 60 feet can be expected to be within ± 1% (both vertically and horizontally) of the length of the bore.

G. Horizontal Directional Drilling

1. **Description of Process and Equipment:** Horizontal directional drilling can be divided into two main classes, Mini-HDD and HDD, based upon the size of the product being installed and the length of the bore. Mini-HDD is for drive lengths of less than 600 feet and pipe sizes up to 10 inches in diameter. Pipe diameters between 12 and 60 inches and pipe lengths over 2,000 feet can be installed by HDD. The distinction between Mini-HDD and HDD is made mainly due to the types of equipment involved.

   Mini-HDD systems are used extensively in the private utility industry for installing power lines, telecommunications cables, or gas lines at shallow depths. Mini-HDD equipment normally consists of an all-in-one, self-powered unit, which may be mounted on tracks and can be transported on a single trailer. The HDD systems used to install larger pipe diameters are monsters by comparison. These systems may occupy a space as large as 150 feet by 250 feet and arrive on the jobsite in as many as 10 trailers.

   Figure 14B-2.05: Typical Large Scale HDD Boring Machine (Megadrill Asia)



Source: Megadrill Asia

   Regardless of the category of directional drilling, the basic process is the same. Although they can be set in a bore pit, the directional drill rig is normally set up on the ground surface. A pilot bore is begun by pushing a drill rod through the ground at a shallow angle (approximately 12 degrees). When the drill head reaches the desired depth, the bore head is steered along a sag shaped curve until it levels out. The pilot bore then continues through the ground at the desired depth and grade until it reaches a receiving pit or the head is once again steered through a sag shaped curve to exit the ground at the surface.

As the name implies, directional drilling is a boring method that can be remotely steered. This is accomplished through the use of a slanted, or anvil shaped device, often called a duckbill. The duckbill attaches to the front of the drill head. The angle of the duckbill causes the drill head to move along a curved path. In order to change the direction of the bore, the drill stem and duckbill are rotated to a position that will cause the bore to move in the desired direction. To bore in a straight line, the drill stem and duckbill are rotated continuously as the bore is advanced. For larger diameter bores, the duckbill may be replaced with a section of slightly bent or curved pipe called a bent sub. This bent sub has the same purpose and effect as the duckbill.

Figure 14B-2.06: Typical Duckbill

Source: Sterling and Thorne, 1999

In order to steer a bore around obstacles, the operator must know the location of the borehead and the direction it is traveling. This information is provided through the various tracking systems that are available. The most common method is a “walk-over” system. A radio transmitter or “sonde” is located directly behind the bore head and transmits a signal. A receiver, similar to those used by utility companies to detect underground pipes or cables, is used to determine the location and depth of the borehead. The drawback of the walkover system is that it may be difficult to gain access to the area directly above the borehead (i.e. for water crossings, or bores under buildings). There are also “hardwire” tracking systems available. These systems relay information such as head location, depth, and inclination and orientation of the head back to a computer. Based upon this information, the operator can make any necessary adjustments to keep the bore on the desired alignment.

For small diameter bores, the product pipe or cable can often be pulled back through the pilot hole with no additional enlargement of the hole required. However, for larger diameter pipes, it is necessary to increase the diameter of the pilot hole to accommodate the product pipe. This is accomplished by back reaming.

After the pilot bore is completed, the drill head is removed and a back reamer is attached to the drill string. The back reamer serves two functions. The first and most obvious is to enlarge the diameter of the borehole to a size large enough to allow room for the product to be installed. As a rule of thumb, the size of the borehole is normally reamed to a diameter of 1.5 times the diameter of the product to be installed. The second function of the reamer is to mix the soil cuttings with the drilling fluids to create a slurry. The reamer is rotated and pulled back through the pilot hole, thereby cutting the soil and increasing the diameter of the bore. At the same time, drilling fluid is pumped through the drill string to the reamer. The cuttings mix with the drilling fluid, forming a slurry. Some of this slurry is forced out of the borehole, into the receiving pit. However, most of the slurry remains in place to support the borehole, and keep it from collapsing until the product pipe is pulled into place.

Upon completion of the boring and reaming processes, the product assembled into one full length. It is laid out in-line with the bore and pulled into place. As it is pulled into place, the required volume of slurry is forced out of the hole. The remaining slurry between the outside of the pipe and the inside of the reamed borehole remains in place permanently to provide support to the borehole.

2. **Typical Applications/Materials:** Directional drilling can be used to install a variety of pipelines, including cables, pressurized gas or water lines, sewer force mains, and water services. Steel and HDPE are the most common types of materials installed by directional drilling; however, PVC, copper, and other flexible materials can also be successfully installed.

   Although it can be done, directional drilling can be difficult for installing products at small slopes. Therefore, it may not be suitable for installing gravity pipelines.

3. **Range of Applications:**

   a. **Pipe Sizes and Bore Lengths:** As mentioned previously, Mini-HDD is for drive lengths of less than 600 feet and pipe sizes up to 10 inches in diameter. HDD can accommodate pipe diameters between 12 and 60 inches and pipe lengths over 2,000 feet.

   b. **Soil Conditions:** Directional boring techniques can be utilized under many different soil conditions. Clays, silts, and sands are considered ideal. Directional drilling in gravely or rocky ground can be done; however, speed and accuracy may be reduced considerably.

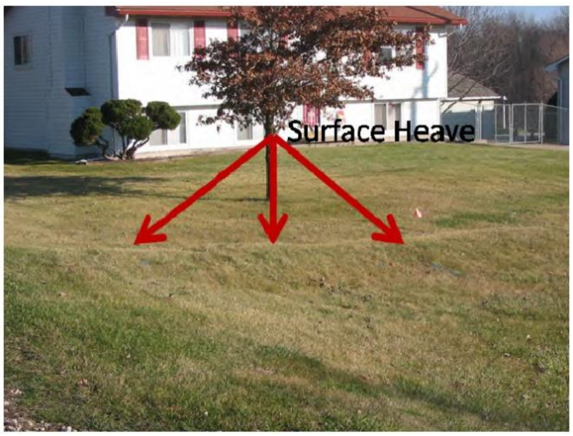
   c. **Tolerances:** The accuracy for directional drilling varies depending on ground conditions and operator experience. Normally, an accuracy of ± 1% of the length of the bore can be expected for HDD and within 6 to 12 inches for Mini-HDD.

4. **Relative Cost vs. Other Trenchless Methods:** Installation of small diameter pipe and cable by Mini-HDD techniques is very economical and is, quite often, less expensive than open cut techniques. In fact, Mini-HDD is often preferred over open cut even in wide open areas due to its lower overall cost.

   Installation by HDD of large diameter pipelines is a highly specialized operation requiring special equipment. Given this, it is not economically feasible for relatively short bores. However, given sufficient bore length, HDD can become an economical alternative due to the minimal environmental impact speed of installation.

5. **Potential Problems with HDD:** While HDD is not intended to be a compactive method of installation, a poor choice of drilling fluid or other installation errors can lead to compaction around the installed pipe. This unintended compactive effort can lead to frac out and/or lifting of the soils.

Figure 14B-2.07: Pavement Cracking Caused by HDD Installation



Source: Iowa Highway Research Board Project TR-570

Shallow HDD bores under pavements can result in pavement cracking, as seen in the left picture in the above figure. A general rule of thumb is that the depth of the installation should be one foot per inch of pipe being installed.

HDD installations that are not back-reamed to a sufficiently large diameter have been observed to cause heave. When the product pipe is pulled into the hole, some of the drilling fluid is displaced and must flow out of the hole. The drilling fluid is expected to pass in the opposite direction that the pipe is being pulled and therefore must travel through the annular space between the outside of the pipe and the edge of the hole. The rule of thumb for HDD is that the diameter of the hole should be 1.5 times the outside diameter of the pipe. However, sometimes contractors do not include the thickness of the pipe and bells or other protrusions on the outside of the pipe when they calculate pipe diameter. If the machine generates high enough pulling force, drilling fluid pressure can become high enough to heave the soil. An example of soil heaving is seen in the above figure.

H. Pipe Jacking and Utility Tunneling

1. **Description of Process and Equipment:** The processes of pipe jacking and utility tunneling are two distinctly separate methods, but are both characterized by their necessity for workers to enter the pipe to perform the excavation.

   a. **Pipe Jacking:** Pipe jacking is a trenchless technique in which a casing pipe is pushed, or jacked, into the ground, while at the same time, soil is excavated by personnel at the front of the bore.

      The setup for a pipe jacking operation begins in a manner similar to that for auger boring. Bore pits are excavated at the entrance and exit of the proposed bore. A guide rail, or jacking frame, is placed to support the pipe and the jacking equipment, and a thrust block (normally concrete) is installed.

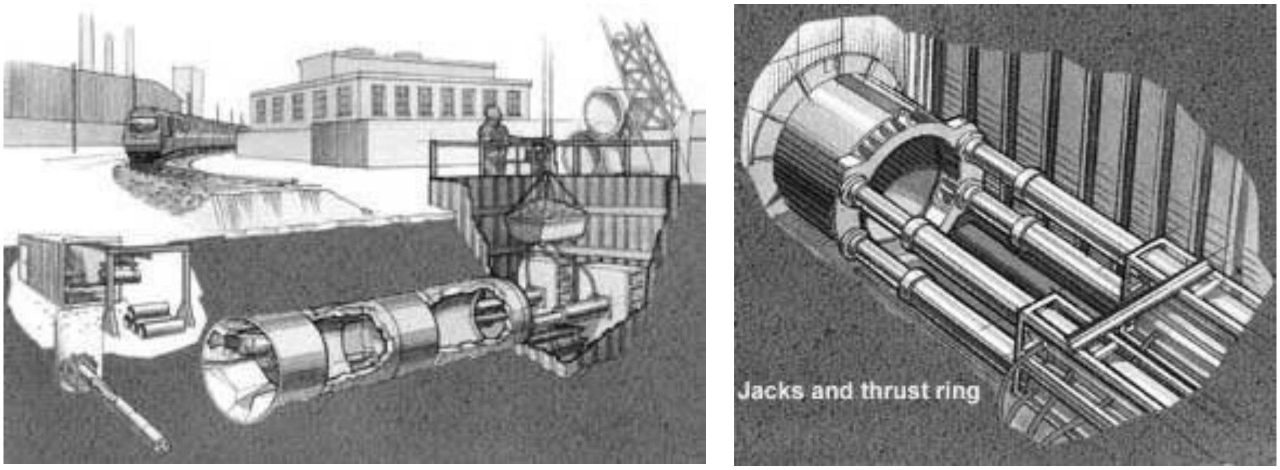
      A jacking shield is pushed into the ground, ahead of the following pipe sections. The purpose of the jacking shield is to provide a safe area for workers to perform the excavation at the face (front) of the bore. This excavation may be done manually or mechanically, as discussed below.

      Spoil is normally removed from the bore using small carts which are either battery powered, or pulled in and out with a winch. Alternatively, the spoil may be removed with small augers, or by using a conveyer system.

      As excavation takes place, hydraulic jacks at the entrance pit force the pipe through the ground. The pipe is jacked in sections. When one section is completed, the hydraulic jacks are moved back, another section of pipe is set at the entrance pit, and the process is repeated until the bore reaches the reception pit.

      Pipe jacking is a very accurate method of boring. A laser back at the bore pit is set to the appropriate line and grade and shot through the pipe to a target at the front of the bore. Workers can view the laser beam to determine what corrections need to be made. All modern equipment incorporates an automatic steering system. The bore is steered by adjusting the jacking forces back at the bore pit or at the face of the bore.

Figure 14B-2.08: Typical Pipe Jacking Installation



Source: The Pipe Jacking Association

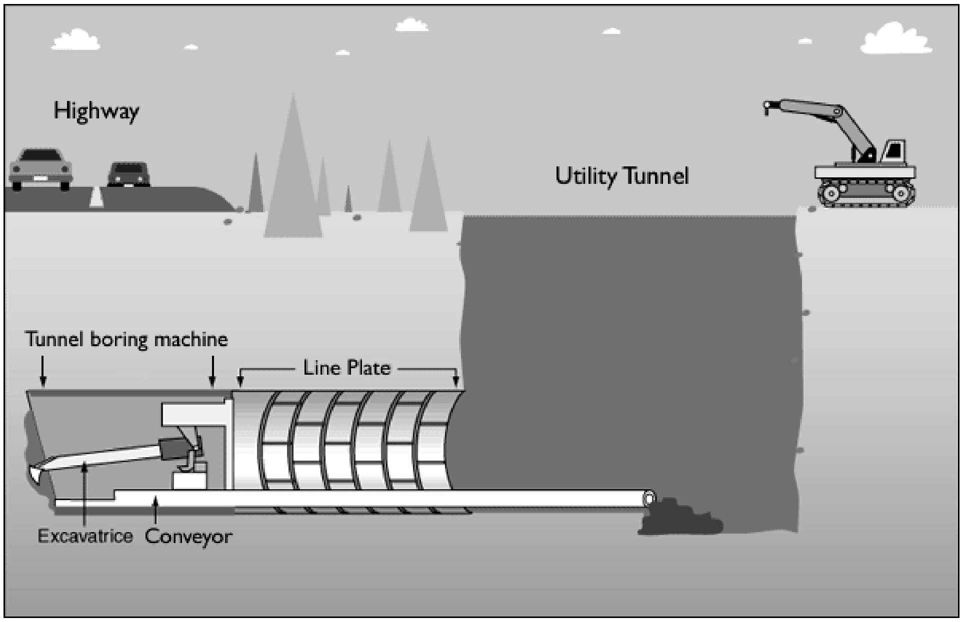
b. Utility Tunneling: Like pipe jacking, utility tunneling excavation is done inside of a specially designed tunneling shield. The method is differentiated from pipe jacking by the lining installed, and the method of jacking.

In pipe jacking, pipe forms the lining of the borehole. In utility tunneling, steel liner plates or rib and lagging form the liner. The liner plates are prefabricated modular units utilized to construct a temporary lining. This temporary lining supports the excavation until it is complete. Upon completion, the permanent utility pipe is pushed through the tunnel, and the annular space between the steel lining and the pipe is filled.

Another distinction between utility tunneling and pipe jacking is that this liner is not jacked or pushed into place. It is constructed in-place in the tail section of the shield. Hydraulic jacks or rams are not required in the bore pit. Rather, as the tunnel is extended, hydraulic jacks on the rear section of the tunneling shield thrust against the previously installed liner plates, pushing the shield forward. After the shield has been pushed forward far enough so that one or more courses of liner plates can be placed, the jacking operation ceases and the jacks are retracted so that the liner plates can be installed in the tail section of the shield.

Like pipe jacking, excavation inside of the shield is done by personnel, either manually or mechanically as discussed below. The spoil is removed from the bore with either small carts, augers, or a conveyer system.

Figure 14B-2.09: Typical Utility Tunnel Installation



Source: Nella Drilling, Inc.

c. Pipe Jacking and Utility Tunneling Excavation Methods: Several different methods are available for excavating the spoil for a pipe jacking or utility tunneling project. These methods include hand mining, open face shield, tunnel-boring machine, and the road header method.

1) **Hand Mining:** Hand mining is the most basic method of excavation. It consists of workers using either pneumatic equipment or simply picks and shovels to excavate the material away from the face of the bore. This method is very slow, but has distinct advantages. Workers can readily address mixed face conditions, boulders, and other large obstacles such as tree stumps. This method is limited to relatively short drives that do not justify the investment in tunneling equipment, or to conditions that demand hand tunneling.
2) **Open Face Shield:** As the name implies, the open face shield method consists of an exposed face at the front of the shield. Excavation is done by a small backhoe or other piece of equipment mounted inside of the shield. Like hand mining, this method allows workers full access to the face of the bore to deal with poor soil conditions or obstacles in the path of the bore.
3) **Road Header Method:** A road header is a wheel or track mounted piece of equipment with a toothed sphere attached to the end of a boom. The ball is rotated and used to excavate soil or rock from the face of the bore. This method is particularly useful in non-circular tunnels.
4) **Tunnel Boring Machines:** Excavation by tunnel boring machine is the most common method of excavation for pipe jacking; however, its use is limited to circular tunnels. A tunnel boring machine is a full face machine, which means that the face of the excavation is fully supported by the cutting head. The cutting head rotates and excavates soil from the face. This soil passes through small openings in the cutting head. An operator sits at the front of the bore, immediately behind the cutting head. From this vantage point, the operator can steer the bore and make any necessary adjustments. Tunnel boring machines are fast and efficient, especially for long bores.

2. **Typical Applications/Materials:** Pipe jacking is used primarily for conduits that must conform to tight tolerances such as culverts and gravity storm and sanitary sewers. Jacking pipe materials must be able to withstand the high compressive forces that are involved in the process. Typical jacking pipe can be steel, reinforced concrete, centrifugally cast glass-fiber-reinforced polymer mortar (CCFRPM - Hobas) pipe, or vitrified clay. In many cases, the jacking pipe also doubles as the carrier pipe. If a separate carrier pipe is installed, it can be almost any pipe material, including plastic or other flexible pipe.

   Utility tunneling is also used for installing utility conduits such as storm sewers, sanitary sewers, and culverts. Utility tunneling should be differentiated from the major tunneling installations that are used as passageways for pedestrians or vehicles. As discussed above, utility tunneling involves the installation of prefabricated steel liner plates as a temporary support structure. The final carrier pipe needs to be of sufficient strength to withstand the forces imparted during the grouting of the annular space, and any potential earth loads transferred to the carrier pipe.

3. **Range of Applications:**

   a. **Pipe Sizes and Bore Lengths:** Since both pipe jacking and utility tunneling require workers to enter the excavation, these methods are typically limited to pipe sizes 42 inches and greater. For extremely long installations, the minimum recommended size is 48 inches.

      Theoretically, the length of pipe jacking and utility tunneling installations is unlimited. Since the liner pipe is not pushed through the ground for utility tunneling, forces do not increase as the length of the tunnel increases. Pipe jacking lengths can be increased through the use of intermediate jacking stations. The intermediate jacking stations may be installed at intervals along the pipe, and allow the pipe to be jacked in sections, rather than all at once.

   b. **Soil Conditions:** While stable consistent granular and cohesive soils are the most favorable for these methods, pipe jacking and utility tunneling can be utilized in almost all soil conditions with the appropriate equipment and precautions.

   c. **Tolerances:** Pipe jacking and utility tunneling are highly accurate, steerable methods. Installations within 1 inch of proposed line and grade are possible.

4. **Relative Cost vs. Other Trenchless Methods:** Pipe jacking and utility tunneling are specialized operations that require a significant investment in equipment. Typically, these methods are significantly more expensive than auger boring or other trenchless methods (except microtunneling) due to the equipment investment required.

I. Microtunneling

1. **Description of Process and Equipment:** The microtunneling process is essentially remote controlled pipe jacking. All operations are controlled remotely from the surface, eliminating the necessity for personnel to enter the bore.

   The excavation is made with a remotely controlled tunnel boring machine. Like pipe jacking, the tunnel boring machine is laser guided and can be steered to maintain the required grade and alignment.

   The spoil generated can be removed by either mixing the soil with water into a slurry and pumping it out of the bore or by removing the spoil with an auger inside a separate auger casing inside the jacking pipe.

2. **Typical Applications/Materials:** Like pipe jacking, microtunneling is normally utilized for constructing pipelines requiring a high degree of accuracy, such as gravity storm sewers and gravity sanitary sewers. Similar to pipe jacking, the casing pipe must be able to withstand the high jacking forces required to push the pipeline through the ground. Common pipe materials installed with microtunneling include steel, ductile iron, reinforced concrete pipe, centrifugally cast fiberglass-reinforced polymer mortar (CCFRPM) pipe, and vitrified clay pipe.

3. **Range of Applications:**

   a. **Pipe Sizes and Bore Lengths:** The term "microtunneling" can be deceiving. Microtunneling was originally developed as a pipe-jacking technique for pipe diameters too small to allow man entry. Since that time, the sizes of pipes installed by microtunneling have continued to increase due to the benefits and added safety provided by eliminating the requirement for personnel to enter the bore. Now, almost any diameter of pipe can be installed by the microtunneling technique from 10 inches to 120 inches. For example, the Chunnel, with a diameter of 25 feet, was constructed using microtunneling techniques.

      One limitation of microtunneling is that since the excavation must be done with a tunnel boring machine, only circular pipes can be installed.

   b. **Soil Conditions:** Microtunneling can accommodate a wide variety of soil conditions. Boulders or rocks up 20% to 30% of the diameter of the pipe can normally be removed.

   c. **Tolerances:** Like pipe jacking, microtunneling is a laser-guided method, which is steerable and highly accurate. Installations within 1 inch of proposed line and grade are possible.

4. **Relative Cost vs. Other Trenchless Methods:** Microtunneling equipment is highly specialized and costly. However, due to the advantages and speed that the method provides, unit prices on large projects can be in line with other trenchless methods. For relatively short bores, microtunneling tends to be costly.

Table 14B-2.01: Summary of Various Trenchless Techniques for New Construction

Method	**Diameter Range** (inches)	**Maximum Installation Length** (feet)	**Pipe Material**	**Working Space Requirements**	**Typical Application**	**Accuracy / Tolerances**
Auger Boring Steered Auger Boring	4 to 60	600	Steel	Entry and Exit pits: 26 to 36 feet long	Road and Railroad Crossing Pressure and Gravity Pipe	± 1% of Bore Length ± 12 inches
Compaction Methods	Less than 12	200	Any		Pressure Pipe/Cable	± 1% of Bore Length
Pipe Ramming	2 to 55	200	Steel	Large area required to accommodate bore pit and to lay out pipe	Road and Railroad Crossing	Not Accurate
Slurry Methods: Water Jetting Slurry Boring	Less than 4 Varies	30 Varies	Any	Minimal	Pressure Pipe/Cable	Not Accurate
Directional Methods: Mini-HDD HDD	2 to 12 12 to 60	600 Greater than 2,000	PE, Steel, PVC PE, Steel	Boring pits not generally required. For HDD, need space to set up rig: up to 400 feet long	Pressure Pipe/Cable Pressure Pipe	Varies
Microtunneling	8 and Greater	750	RCP, GPMP, VCP, DIP, Steel, PVC, PCP	Jacking Pit: 20 feet long Requires smaller retrieval pit	Gravity Pipe	± 1 inch
Pipe Jacking	42 and Greater	1,600	RCP, GPMP, Steel	Jacking Pit: 10 to 30 feet long	Pressure and Gravity Pipe	± 1 inch
Utility Tunneling	42 and Greater	Unlimited	RCP, GPMP, Steel	Launch Pit: 10 to 30 feet long	Pressure and Gravity Pipe	± 1 inch

Abbreviations:

DIP - Ductile Iron Pipe
PE - Polyethylene
VCP - Vitrified Clay Pipe
GRP - Glass-Fiber Reinforced Polyester
PVC - Poly-Vinyl Chloride
GPMP - Glass-Fiber Polymer Mortar Pipe
RCP - Reinforced Concrete Pipe

Source: Trenchless Construction Methods and Soil Compatibility Manual, 1999

Evaluation of Existing Conditions for Rehabilitation

A. Introduction

As existing sewers and manholes begin to deteriorate and fail, the traditional response has been to remove and replace them. However, there are alternatives available that can rehabilitate the sewer line or manhole, significantly extending the service life of the installation, normally at a lower dollar cost and with less inconvenience to the general public.

Rehabilitation techniques for sewer lines include cured-in-place pipe, fold-and-formed pipe, pipe sliplining, and pipe bursting. Manhole rehabilitation techniques include resin coating, centrifugally cast mortar, and structural liners.

B. Evaluating the Condition of the Existing System

The first step in deciding to rehabilitate a sewer pipe and selecting an appropriate rehabilitation technique is to conduct a thorough inspection of the sewer in order to determine the condition of the existing line. This evaluation should include an assessment of the sewer's structural condition, capacity, and the amount of inflow and infiltration (I/I).

1. **Structural Analysis:** The structural condition of the existing sewer needs to be evaluated in order to determine if the pipe can be rehabilitated, or if complete replacement is necessary. If rehabilitation is a possibility, the existing condition of the pipe may determine the rehabilitation technique necessary.

   A structural evaluation should begin by thoroughly cleaning the line by water jetting. This should be followed by a closed-circuit television (CCTV) inspection. The inspection should provide information on the location, type, size, and severity of each defect in the pipe. Defects, which may be identified, include broken sections, longitudinal or circumferential cracks, offset joints, collapses, and corroded sections. In addition to the structural defects, obstacles that might affect the rehabilitation process, such as roots or projecting laterals, should be identified during the televising process. The locations of service connections should also be noted during this process.

   Manholes should be visually inspected to identify any significant cracks in the walls, deteriorated inverts, crumbling brick or concrete, or signs of chemical attack.

2. **Capacity and Inflow and Infiltration Analysis:** A capacity analysis of the existing line should be conducted. The capacity of the existing line should be determined based upon the size and slope of the sewer. The computed sewer capacity should take into account any reduction in size due to the rehabilitation method selected. This computed capacity is compared against future projected sewer flows.

   In order to determine future flows for sanitary sewers, current flows should be measured. These flows are measured at various times in order to determine maximum and minimum flows. In addition, the flows should be measured both during dry weather conditions and during high groundwater and heavy rain conditions in order to identify potential I/I problems.

I/I flows come from several sources. Inflow is water which is dumped into the sanitary sewer system through improper connections, such as downspouts or sump pumps. Infiltration is caused by water leaking into the system through pipe joints, openings in manholes, and other sources. I/I of storm water and groundwater into the sanitary sewer cause unnecessarily high sewer loading at the wastewater treatment plant. These high flows increase treatment costs and reduce the effective capacity of the plant.

Significantly increased readings (compared to those for dry weather) during periods of high groundwater indicate that infiltration is occurring. Sharp flow peaks during rainfall events indicate inflow into the sewer. While rehabilitating the line will likely reduce the amount of infiltration that is occurring, it will do little to cut the amount of inflow. In situations where inflow is identified as a problem, further investigation, such as smoke testing, is warranted to identify the sources of inflow and eliminate them.

After completing the flow monitoring process, future flows are calculated by adjusting the current measured flows for anticipated population growth and increased industrial and commercial usage.

The sewer line should be checked to verify that it has sufficient capacity to carry future flows for the projected life of the rehabilitation.

C. Selection of a Rehabilitation Technique

Based upon the results of the existing system evaluation, a proper rehabilitation technique should be selected. If the condition and capacity of the existing pipe do not require removal and replacement, an appropriate rehabilitation method, as described in the following sections, should be selected.

Pipeline Rehabilitation

A. Cured-in-place Pipe (CIPP)

1. **Description of Process and Materials:** One of the most widely used pipeline rehabilitation methods is the cured-in-place pipe (CIPP) lining method. The CIPP process begins by thoroughly cleaning the existing pipeline, removing any debris or protruding laterals, noting locations of existing lateral connections, and diverting sewer flows, if they are high, in preparation for the installation of the liner.

   a. **Liner:** The liner consists of an absorbent, flexible, industrial grade felt tube with an impermeable membrane on the inside surface. The size and length of the tube are custom made to fit each project. This makes CIPP an ideal method for odd sized or odd shaped pipes such as old brick sewers.

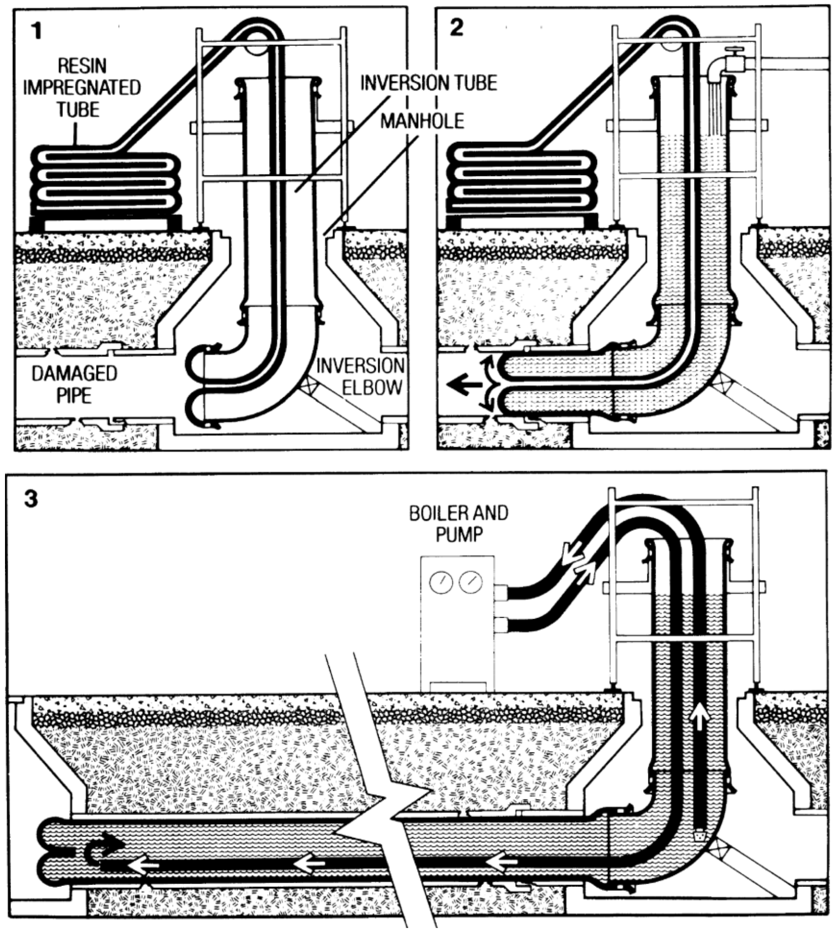
   b. **Resin:** As mentioned above, the resin is what hardens and ultimately gives the CIPP its strength. Various resins are available and each has different properties. The material property, of most interest to the designer in determining the thickness of the liner to be installed, is the strength or flexural modulus of elasticity. Resins with a flexural modulus of 250,000, 300,000, and 400,000 psi are common. Additional resin strengths may be available; it is recommended that a CIPP supplier be contacted to determine the currently available resins.

      Over time, the materials used for construction of CIPP will undergo deformation when exposed to a constant load. This deformation is defined as creep. This long term creep effectively reduces the strength of the liner over time. To account for this, a reduction factor (normally 50%) is applied to the initial flexural modulus of the resin material to provide a long term modulus of elasticity (E_L). This long term modulus is the value utilized in the liner designs discussed in the following sections on design.

   c. **Preparation for Lining:** Prior to beginning the lining process, the sewer should be thoroughly cleaned and televised. All service connection locations should be identified and carefully noted. Large pieces of debris such as bricks or chunks of concrete should be removed. Any obstructions that may interfere with the lining process, such as protruding services, severely offset joints, collapsed pipes, tree root penetration, etc., should be addressed by either remote repair or by open cut point repair.

   d. **CIPP Installation Process:** Just before delivery to the project site, the felt tube is thoroughly wetted with a thermosetting resin. As the resin is applied to the felt, the liner is turned inside-out, resulting in the impermeable membrane being on the outside, and the resin-impregnated felt on the inside of the tube. The liner is loaded on a truck and delivered to the project. Since the resin cures in the presence of heat, the liner may be shipped in a refrigerated truck or packed in ice if weather conditions dictate.

Figure 14C-2.01: Typical CIPP Inversion Process



Source: Iseley and Najafi, 1995, from Insituform

The sewer is lined by inverting the tube into the sewer line. In the inversion process, one end of the tube is cuffed back and attached to an inversion ring directly above the access point (typically a manhole). The inverted tube is filled with water. The resulting pressure head is used to force the liner through the sewer line and continue the inversion process. In addition to inverting the tube, the water head also acts to expand the tube inside the pipe, forcing the resin soaked felt (once again on the outside of the liner) against the inside walls of the existing sewer pipe. The pressure head applied by the water causes some of the resin in the felt to be squeezed out, filling leaky joints and cracks in the pipe. ASTM F 1216 addresses this potential loss of resin from the felt tube by requiring an additional 5 to 10% (by volume) of resin to be added to the tube beyond its saturation point.

Rather than inverting with water pressure as described above, CIPP liners can also be pulled into place by a winch and then expanded. When the pulled-in-place process is utilized, the membrane remains on the outside of the liner during the entire process. This membrane may remain impermeable or be perforated or slotted prior to installation (this should be specified in the plans). Even though the liner may be perforated or slotted during the installation process, the exterior liner can inhibit the migration of the resin from the liner into joints and cracks.

While the pulled-in-place installation method can be successfully utilized, the inversion method may be better suited for pipelines that have high levels of infiltration through the pipe joints or cracks. The pulled-in-place and inversion methods work equally well for lining pipes whose defects are more structural in nature.

e. **Curing and Hardening the CIPP Liner:** Regardless of the method of installation, the newly installed liner is then cured by applying heat. Typically, this is done by heating and circulating the water used to invert and expand the tube, or by applying pressurized steam to the line. The applied heat causes the thermosetting resin in the felt to cure or harden. This changes the resin from a liquid to a solid. After the resin has cured, the CIPP is cooled, resulting in a new pipe with a slightly smaller inside diameter, but of the same general shape as the original pipe.

f. **Completion:** The ends of the CIPP are trimmed off, and the service laterals are reopened. Reopening the service connections can be done by man-entry for larger diameters or robotically for smaller diameters. Normally, a small dimple is left in the liner directly over each service connection, allowing them to be easily located and reopened. However, the number and locations of the service connections should be noted during the pre-lining televising process to ensure that all connections are reopened and to aid in locating those that are difficult to identify.

The result of the CIPP process is that a new pipe is formed within the existing sewer pipe. This new pipe reduces infiltration and adds structural integrity to the existing line. The expected service life of a cured-in-place liner is generally accepted to be 50 years.

2. **CIPP Design:** Cured-in-place pipe liners should be designed in accordance with ASTM F 1216 – "Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube." This standard is recognized by virtually all CIPP suppliers and contractors. The standard for the CIPP is all inclusive, covering material requirements, construction methods, and design parameters. The equations and definitions utilized in this section are taken from ASTM F 1216.

The first step in designing a CIPP project is identifying the condition of the existing pipe. ASTM F 1216 divides existing pipe conditions into two classes: partially deteriorated condition and fully deteriorated condition. The condition of the pipe affects the method of design that is utilized. According to ASTM F 1216, these conditions are defined as follows:

"*Partially deteriorated pipe* - the original pipe can support the soil and surcharge loads throughout the design life of the rehabilitated pipe. The soil adjacent to the existing pipe must provide adequate side support. The pipe may have longitudinal cracks and up to 10% distortion of the diameter. If distortion of the diameter is greater than 10%, alternative design methods are required" (see fully deteriorated pipe).

"*Fully deteriorated pipe* - the original pipe is not structurally sound and cannot support soil and live loads nor is expected to reach this condition over the design life of the rehabilitated pipe. This condition is evident when sections of the original pipe are missing, the pipe has lost its original shape, or the pipe has corroded due to the effects of the fluid, atmosphere soil or applied loads."

a. **Design for Partially Deteriorated Gravity Pipe Condition:** Generally, the partially deteriorated condition is used when the existing pipe is in good condition, but has leaky joints. For this reason, the liner is only designed to resist the hydrostatic loads due to groundwater, since the soil and live loads are still being supported by the original pipe. The required thickness of the liner is determined utilizing the following equations:

$$t = \frac{D_o}{\left[\frac{2KE_L C}{P_W\left(1-\nu^2\right)N}\right]^{\frac{1}{3}} + 1}$$

Equation 14C-2.01
(Note: this is a rearrangement of Equation X1.1 from ASTM F 1216)

where:

t = CIPP thickness, inches
D_o = Mean outside diameter of the CIPP, inches
K = enhancement factor of the soil and existing pipe, typically 7 (conservative), dimensionless
E_L = long term (time corrected) modulus of elasticity for CIPP, psi (see Section 3.2 A-1.b)
P_W = groundwater load (hydrostatic pressure), psi

$$P_W = \frac{H_W(ft) \times 62.4(pcf)}{144\left(in^2/ft^2\right)} \text{ or } P_W = 0.433\left(H_W\right)$$

Equation 14C-2.02

H_W = Groundwater height above the top of the pipe, ft
ν = Poisson's ratio (0.3 average), dimensionless
N = factor of safety (normally 2.0), dimensionless
C = ovality reduction factor, dimensionless. See Table 3.2-1, or

$$C = \left(\left[1 - \frac{q}{100}\right] / \left[1 + \frac{q}{100}\right]^2\right)^3$$

Equation 14C-2.03

q = percent ovality of original pipe - estimate from the CCTV inspection the amount of ovality (deflection from original round shape). Normally, the ovality will vary along the length of the line (use the most oval condition). The more ovality, the thicker the liner will need to be.

Table 14C-2.01: Ovality Reduction Cactor (based upon Equation 14C-2.03)

Ovality, q, %	Factor, C	Ovality, q, %	Factor, C
0	1.00	12	0.35
2	0.84	14	0.29
4	0.70	15	0.27
5	0.64	16	0.24
6	0.59	18	0.20
8	0.49	20	0.17
10	0.41		

If the groundwater table is below the invert of the pipe, the hydrostatic pressure is zero, and Equation 1 cannot be used to calculate the liner thickness. If it is determined that the CIPP will not be underwater, ASTM F1216 recommends a CIPP with a maximum standard dimension ration (SDR) of 100. When there are no external hydrostatic forces on the pipe, the CIPP thickness is calculated with Equation 14C-2.04:

Equation 14C-2.04

$$t = \frac{D_o}{100}$$

where:

t	=	CIPP thickness, inches
D_o	=	Mean outside diameter of the CIPP, inches

When the existing host pipe is out of round or is deformed in a localized area, bending stresses may be the predominant force action on the CIPP when external hydrostatic pressure surrounds the pipe. When the host pipe is oval, the CIPP must be checked to ensure that bending stresses do not exceed the long term flexural strength of the CIPP. This value should be available from the liner supplier. The bending stresses that the pipe is expected to see over its life are given by Equation 14C-2.05 below.

Equation 14C-2.05

$$\left[\frac{1.5q}{100}\left(1+\frac{q}{100}\right)\left(\frac{D_o}{t}\right)^2 - 0.5\left(1+\frac{q}{100}\right)\left(\frac{D_o}{t}\right)\right]P_W N = \sigma_L$$

where,

σ_L	=	long term flexural strength of the CIPP, psi
q	=	percentage of ovality of the original pipe, %
D_o	=	mean outside diameter of the CIPP, inches
t	=	CIPP thickness, inches
P_W	=	external water pressure, psi (see Equation 14C-2.02)
N	=	factor of safety (normally 2.0), dimensionless

b. Design for Fully Deteriorated Gravity Pipe Condition: Generally, the fully deteriorated condition is chosen when the existing pipe is showing signs of significant deterioration. The liner pipe is expected to carry all of the hydraulic, soil, and live loads by itself, as if the host pipe were not present. The fully deteriorated liner design is begun by determining the anticipated external loading on the liner:

Hydrostatic pressure:

P_W - See Equation 14C-2.02 in the preceding section.

Soil pressure:

Soil pressure is estimated by determining the prism load, P_s, acting on the pipe. This load is determined as follows:

$$P_s = \frac{\omega H_S R_W}{144}$$ **Equation 14C-2.06**

where,

P_s = Soil prism loading pressure, psi
ω = Soil density, lb./ft^3
H_S = Height of soil above top of pipe, feet
R_W = Water buoyancy factor, dimensionless

$$R_W = 1 - 0.33\left(\frac{H_W}{H_S}\right) \text{(minimum value = 0.67)}$$ **Equation 14C-2.07**

H_W = water height above the top of the pipe, feet
H_S = soil height above the top of the pipe, feet

Live load:

Live load exerted by traffic, railroads, aircraft, or from other sources should be calculated. Typical values are shown in Table 14C-2.02.

Table 14C-2.02: Live Load Pressures at Various Depths

Height of fill (feet)	**Highway HS-20 (psi)**	**Railroad Cooper E-80 (psi)**	**Airport[1] (psi)**	
			Rigid Pavement	***Flexible Pavement***
1	16	19	10	59
2	7	18	9	30
3	3	16	8	23
4	2	14	8	16
5	2	12	7	12
6		10	6	10
8		8	5	7
10		6	4	5
15		4		
20		2		

[1] 180,000 Pound Dual-Tandem Gear Assembly, 190 psi tire pressure, 26 inch c/c spacing between dual tires, 66 inch c/c spacing between fore and aft tandem tires.
Table values developed from American Concrete Pipe Association, Concrete Pipe Design Manual, 2000

Total external pipe load:

$$P_t = P_W + P_S + P_L$$ **Equation 14C-2.08**

where,

P_t = Total external pipe load, psi
P_W = Hydrostatic pressure, psi
P_S = Soil pressure, psi
P_L = Live load, psi

Once the total external load that the liner pipe must support has been determined, the liner thickness can be determined with Equation 14C-2.09.

Equation 14C-2.09
(Note: this is a rearrangement of Equation X1.3 from ASTM F 1216)

$$t = \left[\frac{0.375\left(P_t \frac{N}{C}\right)^2 D_o^3}{E_L R_W B' E'_S} \right]^{1/3}$$

where:

t = CIPP thickness, inches
P_t = Total pressure due to water, soil, and live load acting on CIPP, psi
N = factor of safety (normally 2.0), dimensionless
C = ovality reduction factor, dimensionless. See Table 1, or Equation 3
D_o = Mean outside diameter of the CIPP, inches
E_L = long term (time corrected) modulus of elasticity for CIPP, psi (see Section 14C-2, A, 1, b)
R_W = Water buoyancy factor, dimensionless (see Equation 7)
B' = Empirical coefficient of elastic support, dimensionless

$$B' = \frac{1}{1 + 4\boldsymbol{e}^{-0.065 H_S}}$$ **Equation 14C-2.10**

$\boldsymbol{e}$ = base of natural log = 2.718
H_S = soil height above top of pipe, feet
E'_S = Modulus of soil reaction, psi (see note below)

While most of the terms utilized in Equation 14C-2.09 should be known or could be calculated, the Modulus of Soil Reaction is a relatively subjective term. This is a variable that is used to reflect the amount of support being given to the new liner pipe from the surrounding soil. Since the surrounding soil is actually the host pipe, this value can be hard to determine. Use judgment after viewing the CCTV video of the sewer. Badly cracked pipe, missing bricks, and missing pipe are reasons to thicken the liner. Values to be used are between 500 psi (bad pipe, thick liner) and 2,000 psi (fair pipe, thinner liner).

For shallow pipes with little or no groundwater to contribute to the load, a minimum thickness check should be completed. This is a similar design condition as previously described for partially deteriorated pipe having no groundwater. For this special case, there is a provision in ASTM F 1216 that requires the CIPP to have a minimum pipe stiffness that is 1/2 that of the value specified in AWWA C950. Based upon this requirement, the minimum pipe thickness is checked by the following equation:

Equation 14C-2.11

$$t \geq \left(D_o\right)\left(\sqrt[3]{\frac{1.116}{E}}\right)$$

where:

- t = CIPP thickness, inches
- D_o = Outside diameter of CIPP
- E = initial modulus of elasticity for CIPP

One final check of the design needs to be completed. As described for partially deteriorated pipe in the preceding section, the bending stresses need to be checked for pipes which are out of round. The bending stresses that a fully deteriorated pipe is expected to see over its life are calculated in the same manner as for a partially deteriorated pipe and are given by Equation 14C-2.05.

3. Project Considerations:

a. Contractor Review: Prior to bidding any CIPP project, it is always a good idea to review the project with an experienced CIPP contractor. There are construction and performance related limitations to the use of CIPP for pipeline rehabilitation. These limitations relate to the condition of the existing pipeline, the maximum practical thickness of the liner, and the point where CIPP lining is no longer a cost effective option. The contractor may be able to recommend alternatives to reduce the cost or improve the performance of a CIPP liner.

Before designing a project with unusual conditions (odd-shaped pipe, deep pipe, severely deteriorated pipe, difficult access, etc.), it may also be wise to meet with a local CIPP installation contractor, visit the site, and review the sewer tapes with the contractor.

b. Preparing Contract Documents for a CIPP Project: One of the most important things to consider when designing a CIPP liner and preparing the contract documents for the project is that there are a variety of different strength resins available. These different resins can be used in conjunction with different thicknesses of felt to produce multiple liner designs, which meet the requirements of a particular project. The point is to find the combination of resin strength and liner thickness that meets the requirements of the project and has the lowest cost.

For this reason, a single resin strength / liner thickness should not normally be specified on a particular project. Rather, multiple resin strengths / liner thicknesses should be allowed. The combination of resin and liner that is the most economical for one contractor may be different than that of another contractor.

There are two different ways to allow each contractor the flexibility of selecting their own resin/liner combination while assuring that the product being bid meets the requirements of the job. The first method is to allow the contractor to design the liner thickness themselves. If this is done, the engineer should state on the plans that the liner shall be designed in accordance with ASTM F 1216 (or F 1743 if pulled-in-place installation is allowed). In addition, the engineer should require that each potential bidder submit their calculations prior to the letting for review. During this review, the engineer should verify that all bidders are using the same design criteria, and that this criterion matches the site conditions. The alternative method for specifying a liner is to give various combinations of resin and liner thicknesses, allowing the contractor to select the one that is most economical. This method requires the engineer to calculate several liner thicknesses for different resins, but eliminates the possibility that a contractor may use an incorrect value or make an error in their calculations. Typical information provided in the plans is shown in Table 14C-2.03.

Table 14C-2.03: Typical Plan Information for a CIPP Project
(Partially deteriorated condition)

Between Manholes	Length, (ft)	Pipe Dia., in	Water above Pipe H_W, ft	Ovality Factor, C	Safety Factor, N	Min. liner thickness, in		
						E_L(psi) = 125,000	E_L(psi) = 150,000	E_L(psi) = 200,000
41 to 47	1100	12	12	.64	2.0	.240	.226	.205

Table 14C-2.04: Typical Plan Information for a CIPP Project
(Fully deteriorated condition)

Between Manholes	Length, (ft)	Pipe Dia., in	Live Load P_L, psi	Total Load P_T psi	Water Buoyancy Factor R_W	Water above Pipe H_W, ft	Soil above pipe H, ft	Coeff. Of Elastic Support, B'	Ovality Factor, C	Safety Factor, N	Modulus of Soil, E'$_S$, (psi)	Min. liner thickness, in		
												E_L(psi) = 125,000	E_L(psi) = 150,000	E_L(psi) = 200,000
41 to 47	1,100	12	2.0	6.5	.87	2	5	.257	.64	2.0	1,000	.211	.199	.181
47 to 173	1,600	12	7.6	14.8	.88	3	8	.296	.84	2.0	500	.366	.345	.313

The CCTV video of the sewer to be lined should be made available to any potential bidders for review prior to the bid. The contract documents can state that a video is available for viewing and who to contact to arrange a time to view the video, or copies of the video can be distributed to each potential bidder along with the contract documents.

Any time restrictions or additional requirements of the work should be clearly spelled out in the contract documents. These include bypass pumping requirements, time restrictions such as night or weekend work, coordination with other projects, etc. In addition, the contract documents should indicate what testing of the liner will be required.

c. Testing: ASTM F 1216 and F 1743 both address the testing requirements for CIPP linings. When specified by the owner, these specifications require the contractor to prepare samples in one (or both when specified) of two ways. The first is to cut a section of the cured CIPP from the liner where it passes through an intermediate manhole or from the terminating end of the CIPP liner. Alternatively, a section of the uncured liner, which is made of the same felt and resin as the liner, can be bolted into a frame and placed inside the pipe during the curing process.

The samples provided may be tested for short-term flexural (bending) properties and delamination. In addition, exfiltration testing for sewer lines 36 inches and smaller, which do not have any service laterals, may also be required.

d. Costs: The main concern with CIPP is normally the cost. Depending on the situation, the cost for a CIPP liner may approach or exceed than that for open cut removal and replacement. However, one must consider the main reason for considering a trenchless technique in the first place (reduced disruption to the public). The disruption caused by CIPP lining is minimal (the process can often be completed with no excavation of any kind required).

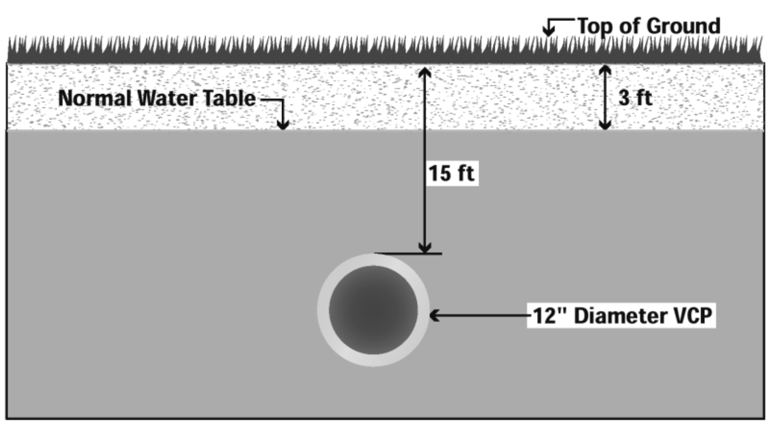
There are several recommendations for reducing the costs associated with a CIPP lining project. The first is to avoid lining small quantities of pipe. Lining large quantities (1,000 feet or greater) under a single project significantly reduces the unit cost of CIPP lining. One of the most significant costs involved with the lining process is mobilization of people and equipment. If two or three separate lines are being considered for lining, it is much more cost effective to do them all as a single project than to do them each separately. Check with surrounding communities to see if they are planning any lining projects that could be combined and let together. This allows the contractor to install two projects with only one mobilization cost.

The second recommendation is to allow a large time frame for a lining project. Most CIPP contractor's territories cover large portions of the country. Providing a flexible time frame allows the contractor to schedule and complete several projects in a particular area at the same time. Again, this reduces mobilization costs.

Also consider other rehabilitation methods which may be less expensive. For larger pipe (>42 inches), sliplining may be more economical if the line has sufficient capacity to account for the reduced cross section created by the sliplining process.

4. CIPP Design Examples:

Figure 14C-2.02: Partially Deteriorated Design Example



Source: Inliner Technologies

Given: A partially deteriorated 12 inch diameter sanitary sewer pipe with some cracking and offset joints is leaking and in need of renewal (see Figure 14C-2.02). Through CCTV inspection, it appears that in at least one area, the pipe is no longer round and has deflected approximately 5 percent. The water table, through investigation, is found to normally be at 3 feet below the surface. The pipe is buried at a depth of 15 feet to the top of the pipe. Assume a modulus of elasticity for the resin of 300,000 psi (long term modulus, $E_L = 0.5(300{,}000) = 150{,}000$ psi) and a long term flexural strength of 2,500 psi.

Required: Determine the wall thickness required for a CIPP liner.

Solution:

Step 1: Determine the hydrostatic pressure acting on the CIPP
H_W = 15' (pipe depth) - 3' (depth of water table) = 12'

$$P_W = \frac{12(\text{ft}) * 62.4(\text{pcf})}{144\left(\text{in}^2/\text{ft}^2\right)} = 5.2\text{psi}$$ From Equation 14C-2.02

Step 2: Pipe ovality

From Table 14C-2.01 or Equation 14C-2.03, C = 0.64

Step 3: Determine the CIPP design thickness from Equation 14C-2.01

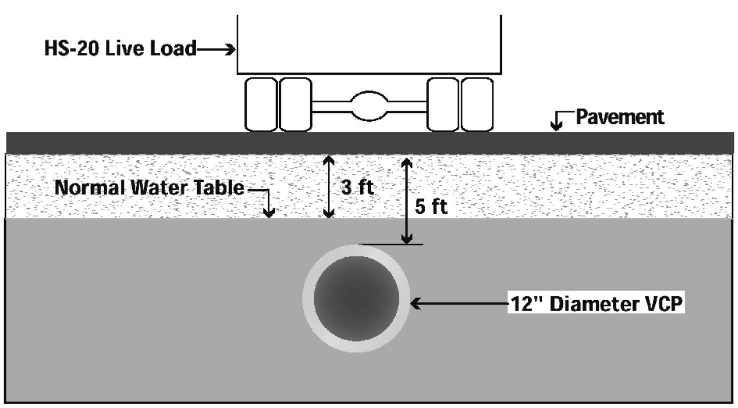
$$t = \frac{D_o}{\left[\dfrac{2KE_LC}{P_W(1-\nu^2)N}\right]^{1/3}+1} = \frac{12}{\left[\dfrac{2(7)(150{,}000)(0.64)}{(5.2)(1-0.3^2)(2)}\right]^{1/3}+1} = 0.226 \text{ Inches}$$

Step 4: Because the pipe has some out-of-roundness, bending stresses must be calculated to ensure they do not exceed the long-term flexural strength of the CIPP. From Equation 14C-2.05:

$$\sigma_L = \left[\frac{1.5(5)}{100}\left(1+\frac{5}{100}\right)\left(\frac{12}{.226}\right)^2 - 0.5\left(1+\frac{5}{100}\right)\left(\frac{12}{.226}\right)\right]5.2(2) = 2020\text{ psi}$$

Since the long term bending stress of 2,020 psi is less than the long term flexural strength of the product, the initial design is OK. However, if the bending stresses had exceeded the long term flexural strength, then the bending stresses would control the thickness design and Equation 14C-2.05 would need to be solved for thickness.

Figure 14C-2.03: Fully Deteriorated Design Example



Source: Inliner Technologies

Given: The 12 inch VCP sanitary sewer pipe shown in Figure 14C-2.03 is cracked and leaking. The severity of the cracking has progressed to the point that some pieces of the pipe wall have fallen into the pipeline and voids have begun to form around the pipe. The pipe is shallow and located under a busy city street subject to HS-20 loading. Assume that the pipe is buried under a clay soil with a unit weight of 120 pcf. As in the previous example, the pipe is no longer round and has deflected approximately 5%. The water table is normally located 3 feet below the surface. Assume a modulus of elasticity for the resin of 300,000 psi (long term modulus, E_L = 0.5(300,000) = 150,000 psi), a long term flexural strength of 2,500 psi, and a soil modulus of 1,000 psi.

Required: Determine the wall thickness required for a CIPP liner.

Solution:

Step 1: Determine the total load, P_t, acting on the pipe above.

Hydrostatic Pressure: $P_W = \dfrac{2(\text{ft}) * 62.4(\text{pcf})}{144\left(\text{in}^2/\text{ft}^2\right)} = 0.87\ \text{psi}$ From Equation 14C-2.02

Soil Load: $R_W = 1 - 0.33\left(\dfrac{2}{5}\right) = 0.87$ (min. = 0.67) From Equation 14C-2.07

$P_s = \dfrac{(120)(5)(0.87)}{144} = 3.63\ \text{psi}$ From Equation 14C-2.06

Live Load: From Table 14C-2.02, the live load at 5 feet for HS-20 loading = 2 psi

Total Load:

$P_t = 0.87 + 3.63 + 2 = 6.5\ \text{psi}$ From Equation 14C-2.08

Step 2: Determine the coefficient of elastic support

$B' = \dfrac{1}{1 + 4e^{-0.065(5)}} = 0.26$ From Equation 14C-2.10

Step 3: Pipe ovality and ovality reduction factor

From Table 14C-2.01 or Equation 14C-2.03, C = 0.64

Step 4: Determine the CIPP thickness from Equation 14C-2.09

$$t = \left[\frac{0.375\left(P_t \frac{N}{C}\right)^2 D_o^3}{E_L R_W B' E_S'}\right]^{1/3} = \left[\frac{0.375\left(6.5\frac{2}{0.64}\right)^2 12^3}{(150{,}000)(0.87)(0.26)(1{,}000)}\right]^{1/3} = 0.199 \text{ inches}$$

Step 5: Minimum thickness check from Equation 14C-2.11

$$t \geq (D_o)\left(\sqrt[3]{\frac{1.116}{E}}\right) = 12\left(\sqrt[3]{\frac{1.116}{300{,}000}}\right) = 0.186 \text{ Inches}$$

Since the design thickness calculated in Step 4 is greater than the minimum thickness given in Step 5, the calculated design thickness is OK.

Step 6: Bending stress check from Equation 14C-2.10

$$\sigma_L = \left[\frac{1.5(5)}{100}\left(1+\frac{5}{100}\right)\left(\frac{12}{0.20}\right)^2 - 0.5\left(1+\frac{5}{100}\right)\left(\frac{12}{0.20}\right)\right](0.4)(2) = 204 \text{ psi}$$

The calculated bending stress (204 psi) is less than the allowable long-term flexural strength of the product (2,500 psi). If the bending stresses had exceeded the long term flexural strength, then the bending stresses would control and Equation 5 would need to be solved for thickness.

B. Fold-and-formed Pipe (FFP) and Deformed Reformed Pipe (DRP) Lining

The fold-and-formed pipe (FFP) and deformed-reformed pipe (DRP) techniques are pipeline rehabilitation methods that were once common, but are seeing decreased usage today. FFP and DRP are both names for a process which consist of folding or deforming a circular polyethylene (PE) or Poly Vinyl Chloride (PVC) pipe into a "U" shape during the time of manufacture, to reduce the cross sectional area of the pipe. The deformed pipe is then coiled on a spool for transport.

At the project site, the coiled pipe is pulled into the existing pipe. The reduced cross section of the folded pipe allows it to fit inside of the host pipe. At this point, steam or hot water is circulated through the pipe to heat it until it becomes pliable. The pipe is then rounded utilizing steam or water pressure, or by pulling a rounding device through the pipe. The pipe is then allowed to cool and harden in place. Service laterals are then reopened robotically or by digging them up and re-establishing them.

As mentioned above, use of the FFP/DRP method is declining, especially in the northern climates. This is due, at least in part, to problems encountered with the pipe developing a "memory" during the deformation process. Over time, the rounded pipe starts to return to its original deformed shape. The action appears to be accelerated by the cold weather conditions experienced in the northern states. This can cause the liner to pull away from the host pipe, reducing support and separating from service laterals.

Given the declining usage of the FFP/DRP process, the design and construction process is not discussed in detail herein. Should the designer wish to pursue a FFP/DRP project, the following ASTM references are provided:

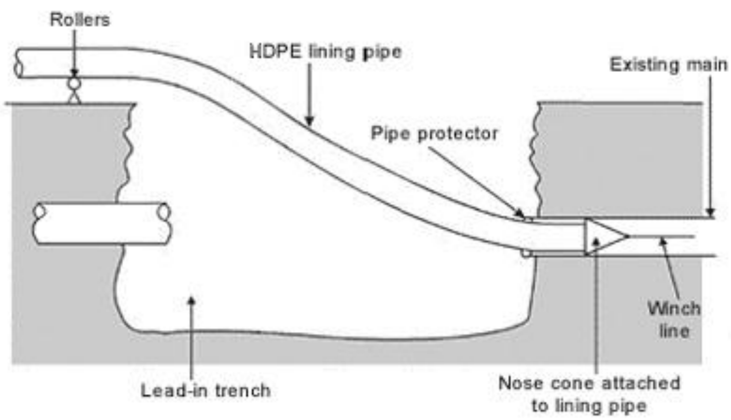
- F 1504 Standard Specification for Folded Poly Vinyl Chloride (PVC) Pipe for Existing Sewer and Conduit Rehabilitation
- F 1947 Standard Practice for Installation of Folded Poly Vinyl Chloride (PVC) Pipe into Existing Sewers and Conduits
- F 1871 Standard Specification for Folded/Formed Poly Vinyl Chloride Pipe Type A for Existing Sewer and Conduit Rehabilitation

F 1867 Standard Practice for Installation of Folded/Formed Poly Vinyl Chloride (PVC) Pipe Type A for Existing Sewer and Conduit Rehabilitation
F 1533 Standard Specification for Deformed Polyethylene (PE) Liner
F 1606 Standard Practice for Rehabilitation of Existing Sewers and Conduits with Deformed Polyethylene (PE) Liner

C. Sliplining

1. **Description of Process and Materials:** The concept of sliplining is one of the simplest techniques for rehabilitating an existing sewer line. Sliplining basically involves pushing or pulling a new pipe into the old one, reconnecting the services, and possibly grouting the space between the pipes.

Figure 14C-2.04: Typical Sliplining Installation



Source: PROFUNDIS Presse

a. **Polyethylene Pipe:** The most common material used for sliplining is polyethylene (PE) pipe. Its ability to be fusion welded into long jointless sections, which can be quickly pulled into place, make it a popular choice for sliplining.

The process of sliplining with PE pipe begins by excavating a starter trench at one end of the line. This trench allows the pipe to be pulled from the ground surface into the pipe without requiring sharp bends. Prior to installation, individual sections of PE pipe are fusion welded together to form a single continuous section of pipe that matches the length of the pipe to be lined. This pipe is laid out in a long line on the ground surface. At this point, sewer flows must be diverted. A cable is fed through the host pipe and attached to a towing head. This towing head is attached to the end of the new liner pipe and the liner is pulled into place with a winch. After the pipe has been pulled into place, the annular space between the host pipe and the lining pipe is normally filled with grout. Grouting this space helps to restrain the liner pipe and increases its stiffness. This can be the most difficult part of a sliplining project. Finally, any service lines must be reconnected. These lines cannot normally be reconnected from inside the pipe like CIPP lining. The connections must be excavated and a new wye installed on the lining pipe.

There are several disadvantages to sliplining with PE pipe. The first is that the pipe stretches under the high tensile forces developed during the installation process. After the pipe is in place and the tensile forces are relieved, the pipe slowly shrinks back to its original shape. This can cause problems if service lines are re-established before the liner pipe has had a chance to relax. If a service connection is re-opened in the liner pipe too soon, the liner pipe will shrink, and the opening in the liner will move past the connection, creating a blockage. It is generally recommended that the liner pipe be allowed at least 24 hours to relax prior to reestablishing service connections. This may require the establishment of temporary services.

Since most polyethylene pipe used for sliplining is black in color (to protect it from ultraviolet degradation) it is a common practice to specify that the pipe be lined with a light colored material to facilitate future CCTV inspection.

b. Other Materials: As indicated above, sliplining pipe is most commonly polyethylene; however, it may be any common sewer material that can be inserted into the host pipe. The main criteria is that, in order to minimize the reduction in cross sectional area of the pipe, joints or socket protrusions beyond the barrel of the pipe should be minimized or eliminated. Fortunately, there are a wide variety of pipe products that meet this criterion. Most are intended for pipe jacking, microtunneling, or directional boring; however, their bell-less or low profile bell configurations also make them well suited for sliplining.

Rigid products such as vitrified clay, concrete, ductile iron, and centrifugally-cast glass-fiber reinforced polymer mortar (CCFRPM) pipe can all be pushed into the pipe from a relatively small access pit. These products are pushed into the pipe utilizing jacking equipment similar to that for microtunneling or pipe jacking. Flow bypassing is not normally required since the line remains open during the insertion process. In addition, due to the inherent structural integrity of these rigid pipe products, grouting the annular space is not as critical as for flexible pipe; however, it is still normally done in order to lock the pipe in place.

Additional pipes products with restrained joints, such as PVC or ductile iron, can also be pulled into the line in a manner similar to that for PE pipe. These materials are usually installed a section at a time, rather than being completely assembled prior to installation like polyethylene. Again, flow bypassing is not normally required. Grouting of the annular space is not required for ductile iron and is optional for PVC depending on the condition of the original pipe.

2. Additional Considerations:

a. Limitations: Probably the biggest issue to contend with when considering a sliplining project is the reduction in pipe diameter that will result. Since the sliplining pipe must fit inside of the host pipe, the outside diameter of the lining pipe should be slightly smaller than the inside diameter of the host pipe at its narrowest point. The capacity of the smaller diameter liner pipe must be checked to verify that it can carry future anticipated flows. If sliplining the pipe causes too much of a reduction pipe capacity, alternative rehabilitation methods such as pipe bursting or CIPP may be considered.

For large diameter pipes (42 inches and greater), it may be possible to reestablish services by man-entry in the pipe; however, in most cases, the service connections must be excavated and re-established.

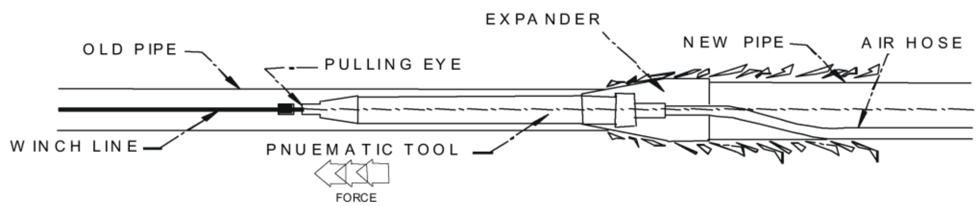
b. Cost: Sliplining is a relatively cost effective method of rehabilitation, especially for larger diameters. Many times, it may be the least costly method of pipeline rehabilitation. This is due in part to the relatively minimal equipment investment required and the commonly available materials utilized.

D. Pipe Bursting

1. **Description of Process and Materials:** Pipe bursting is a trenchless technique for replacing worn out and undersized pipes with a new pipe of the same or larger diameter. The pipe bursting process involves the insertion of a conical shaped bursting head into the old pipe. This head fractures the existing pipe and displaces the pipe fragments outward into the surrounding soil. At the same time, a new pipe is pulled in behind the bursting head. There are several different classes of pipe bursting systems. It should be noted that most of these systems are patented processes. Most contractors pay a licensing fee to be allowed to use the technique.

   a. **Pneumatic Bursting:** Pneumatic pipe bursting is the most commonly used system for pipe bursting. This method utilizes a percussion head (similar to an impact mole) to fracture and break the pipe. In addition to the percussion force, cable is attached to the front of the mole and a winch provides tension to keep the bursting head pressed against the pipe wall and to aid in pulling the new pipe in behind the mole.

Figure 14C-2.05: Typical Pneumatic Bursting System



Source: Simicevic and Sterling

   b. **Hydraulic Expansion:** The hydraulic expansion system utilizes a bursting head, which can be expanded outward to break the existing pipe. In this process, the contracted head is pulled into the pipe. Hydraulic pressure is used to expand the head radially outward, breaking a section of the pipe, and pushing the fractured pieces into the surrounding soil. The head is then contracted and pulled forward with a winch, pulling in the new pipe behind it. This process is repeated in steps until the entire pipeline has been replaced.

   c. **Static Pull:** With the static pull method, the force for breaking and displacing the pipe comes only from pulling the bursting head forward. The cone shaped bursting head converts the horizontal tensile forces into radial forces that fracture the pipe. The tensile forces required to burst the existing pipe and pull in the new pipe are significant. For this reason, a pulling rod assemble may be used in lieu of a winch and cable system.

   d. **Implosion (Pipe Crushing):** The implosion system incorporates a crushing head, which fits around the outside diameter of the existing pipe. As the head is pulled forward, the crushing head breaks the existing pipe and forces the fragments inwards (into the pipe void). A steel cone follows the crushing head and pushes the pipe fragments outward, making room for the new pipe, which is pulled in behind the steel cone.

   e. **Pipe Splitting:** Pipe splitting is a method used for pipes that are not brittle, such as steel, ductile iron, and plastic. Rather than bursting the pipe, it is split open and expanded. This is accomplished with a three step process. Rotary slitter wheels make an initial longitudinal cut along the bottom of the pipe. Next, a hardened sail blade splits the pipe along the bottom. Finally, the pipe is “unwrapped,” or expanded, creating a hole immediately behind the splitter for the new pipe. The old pipe is displaced to a position immediately above the new pipe.

f. **Rigid Pipe Replacement Methods:** All of the methods previously described require the new pipe to be pulled in behind the bursting head. As a result, the new pipe must be flexible (typically polyethylene). There are methods available which allow the installation of rigid pipes (clay, ductile iron, concrete, GPMP, etc.) instead of polyethylene.

One method, based in part upon microtunneling techniques, utilizes jacks to push new sections of pipe in behind the breaking head. Another method utilizes a cable that is thread through the host pipe from the receiving pit to the launch pit and through the next section of pipe to be installed. The cable is attached to a pull plate on the end of the pipe furthest from the receiving pit. This causes the rigid pipe system to act as if it were being pushed instead of pulled.

g. **Pipe Removal Techniques:** Several other techniques, which result in removal of the existing pipe, are also available.

1) **Pipe Eating:** Pipe eating is a modified microtunneling system in which the existing pipe is crushed by the microtunneling head and, along with any excess soil, removed through the new pipeline by a slurry system. The new pipe is jacked in immediately behind the microtunneling machine. This system also allows line and grade adjustments to be made.
2) **Pipe Reaming:** Pipe reaming is a modified version of the back reaming method used for directional drilling. A drill string is fed through the existing pipe and attached to a specially designed reaming head. The head is pulled back through the pipe as the reamer crushes and pulverizes the existing pipe. The pipe fragments and any excess soil required for upsizing are removed via a slurry system.
3) **Pipe Ejection:** Pipe ejection uses modified pipe jacking techniques to remove the old pipe. The replacement pipe is placed against the old pipe and, as the new pipe section is jacked, the old pipe is pushed out into the reception pit. This method requires that the structural condition of the existing pipe be in sufficient condition to withstand the jacking forces produced.

2. Design Considerations:

a. **Range of Applications:** As previously mentioned, the types of pipe suitable for bursting are typically brittle materials such as vitrified clay, cast iron, asbestos cement, and plain concrete. Lightly reinforced or heavily deteriorated reinforced concrete pipe may also be able to be replaced by pipe bursting. Ductile iron, steel, and plastic are not suitable for pipe bursting, but can be replaced by pipe splitting.

The normal bursting length is between 300 and 400 feet, the typical distance between manholes. The size of the pipes currently being replaced by pipe bursting ranges from 2 to 36 inches and is increasing as bursting equipment and techniques are improved. The most common pipe replacement is size-for-size; however, the pipe can be upsized up to 3 pipe diameters or greater.

b. **Pipe Materials and Design:** Replacement pipes should be designed to withstand earth loads and live loads in the same manner as they would be for an open cut situation (see Chapter 9, Part 9B). For most installations, standard sewer or water main pipe materials with special restrained or bell-less joints are suitable. Installation forces are not normally a concern. Flexible pipes with restrained joints, intended for directional drilling and other trenchless construction, are designed to withstand high tensile forces, and are therefore suitable for the moderate tensile forces typically experienced during the pipe bursting process. Likewise, rigid bell-less pipes intended for pipe jacking and microtunneling are designed to withstand very high compressive forces and are, therefore, also suitable for pipe bursting.

While standard sewer and water materials may be used for pipe bursting, the most common pipe material utilized is polyethylene. The main benefits of polyethylene are its flexibility and ability to be fused into long sections. The wall thickness for PE pipe should be designed to withstand earth and live loads. Since polyethylene is softer than other pipe materials, it is more susceptible to damage by the broken fragment of existing pipe. For this reason, the design thickness is commonly increased by 10% to account for scarring of the outer surface. When polyethylene pipe it used, it is a common practice to specify that the pipe be made of or lined with a light colored material to facilitate future CCTV inspection.

c. **Effect of Pipe Bursting on the Surrounding Environment:** All pipe bursting operations result in the displacement of soil. Even when the replacement is size for size, soil is displaced since the bursting head has a diameter greater than that of the replacement pipe. The soil expands in the direction of least resistance and can cause heaving at the surface. The amount of displacement depends on the degree of upsizing, the existing soil properties, and the depth of the bursting.

   Heaving of the existing ground surface is most likely when the existing pipe is shallow or already large diameter pipes are upsized. The potential for heaving must be carefully considered, especially when bursting under existing pavements or structures.

   In addition to heaving, adjacent utilities can also be affected by pipe bursting. In general, if there are deteriorated utilities within 2-3 pipe diameters of the bursting operation, there is potential for damage. Damage to adjacent services or structures can be minimized by creating a temporary excavation along the service or structure to protect it from the effects of the surrounding ground displacements.

d. **Service Connections:** Prior to bursting, all service connections should be excavated and disconnected. The purpose of this is to provide a temporary service connection and to prevent the service line from being damaged by the bursting operation. If the replacement pipe is polyethylene, it is likely that the new pipe has been stretched during the installation process. Prior to reestablishing the service connections, the new pipe should be allowed to relax and return to its original shape for at least 24 hours.

3. **Cost and Comparison to Other Techniques:** Open cut replacement is normally preferred and more cost efficient than pipe bursting when the pipe is shallow and the excavation does not create a major inconvenience. When the depth of the existing pipe, and the resulting excavation required becomes a significant factor in the cost of replacement, pipe bursting may have a significant economic advantage over open cut replacement. Compared to other rehabilitation techniques, such as CIPP and sliplining, the main advantage of pipe bursting is that it does not result in a loss of capacity due to reduced pipe diameter and can significantly increase capacity by upsizing the line.

Manhole Rehabilitation

A. Introduction

Manhole deterioration is a problem that is becoming much more prominent as public infrastructure ages and the number of manholes in service increases. Concrete manholes are often susceptible to attack by sulfuric acid, which eats away at the concrete surface and eventually the steel reinforcing in the manhole, creating the potential for collapse. Brick manholes also deteriorate as the mortar joints are eaten away and begin to leak. Collapsing and leaking manholes require attention to avoid possible street collapses and other problems.

In certain circumstances, manhole rehabilitation, rather than replacement, may be a preferred option. Several rehabilitation techniques are described below. While many other rehabilitation techniques and products are available, the ones listed below are included in the SUDAS Specifications.

B. Corrosion Resistant Chimney Sealant

1. **Typical Applications:** A brush applied, corrosion resistant, aromatic urethane sealant is utilized for sealing existing manhole chimneys, which are showing signs of infiltration or deterioration due to sulfuric acid attack.

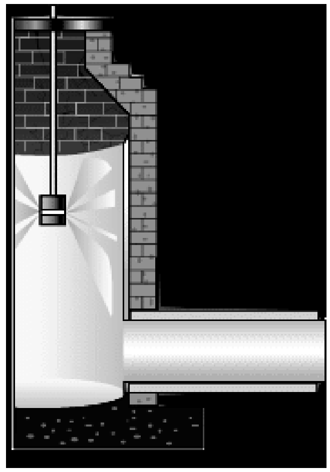
2. **Description of Process and Materials:** The existing surface is prepared by removing all protruding brick, mortar, and other debris. The manhole chimney is sandblasted and pressure washed. Active leaks in the chimney area are sealed with hydraulic cement prior to application of the sealant. The area is then primed, the sealant is mixed, and applied with a brush or trowel. The sealant forms a flexible membrane over the chimney area, which seals out infiltration and protects the area from further chemical attack.

C. Centrifugally Cast Mortar

1. **Typical Applications:** Centrifugally cast mortar linings are utilized to rehabilitate and extend the life of existing manholes, which are still structurally sound, but are experiencing groundwater infiltration and/or moderate deterioration due to the presence of sulfuric acid. This rehabilitation method is used for lining both brick and mortar and concrete manholes.

2. **Description of Process and Materials:** The lining process begins by first cleaning the existing manhole walls of any loose material or debris. This is normally accomplished by washing the interior surface of the manhole with a high pressure washer. Any actively leaking joints or cracks are plugged with hydraulic cement. Next, a rotating applicator is lowered into the manhole. As mortar is pumped through the applicator, it spins with sufficient speed to cast the mortar against the manhole wall. The mortar is of sufficient stiffness that it sticks to the wall without sloughing. As the rotating applicator is raised through the manhole, the entire interior surface of the manhole is coated. Multiple passes of the applicator are made until the desired liner thickness is achieved. The centrifugal casting process creates a slightly rough "orange peel" surface. Normally, the mortar should be smoothed with a brush or trowel to ensure a solid bond to the existing manhole and to create a more finished appearance.

Figure 14C-3.01: Centrifugally Cast Mortar Application



Source: AP/M Permaform

After the liner has been applied to the walls of the manhole, the bench and invert are rehabilitated with the application of 3 inches of hand applied mortar.

If the manhole is being rehabilitated due to deterioration caused by attack by sulfuric acid, an epoxy lining should be applied to protect the new liner from future attack. Typically, this epoxy lining is applied using a rotating centrifugal applicator or an airless sprayer to prevent air entrainment.

The mortar used for the lining process is a high strength, high build corrosion resistant mortar with a 28 day compressive strength of 10,000 psi (24 hours = 3,000 psi). The epoxy coating consists of a two-component, 100% solid epoxy formulated for use in sewer systems.

3. **Design of Liner Thickness:** It should be emphasized that centrifugally cast liners are not intended to be structural in nature. They are only to be applied to existing manholes, which are structurally sound but beginning to deteriorate due to infiltration and/or chemical attack. The liners act to stop any further deterioration of the existing manhole.

   The design of the liner thickness is highly dependent on several factors, including the depth of the water table, traffic loads, and time of opening to traffic. Traffic should be kept off of the area surrounding the manhole for a minimum of 12 hours. The longer the liner has to cure and gain strength prior to applying traffic loads, the thinner the liner may be as indicated in the table below.

Table 14C-3.01: Liner Thicknesses
(Based upon a 48 inch diameter manhole)

Time to opening to Traffic	**Manhole Depth** (feet)	**Minimum Liner Thickness** (inches)
12 hrs (min)	0 to 30	1.25
24 hrs	0 to 30	1.00
7 days	0 to 15	0.75
7 days	15 to 30	1.00

Source: J. Pitt, 1995

D. In-situ Manhole Replacement

1. **Typical Applications:** In-situ manhole replacement is utilized to rehabilitate existing manholes that are severely deteriorated due to infiltration or chemical attack and are no longer structurally sound.

   Since the process results in a new structure inside the manhole, it may even be utilized on structures that are completely missing portions of the walls or bench. Forms are available for both round and rectangular manholes and can be custom fabricated for other odd shaped structures.

2. **Description of Process and Materials:** In-situ manhole replacement consists of constructing forms inside an existing manhole and pouring new walls, which are structurally independent of the existing walls.

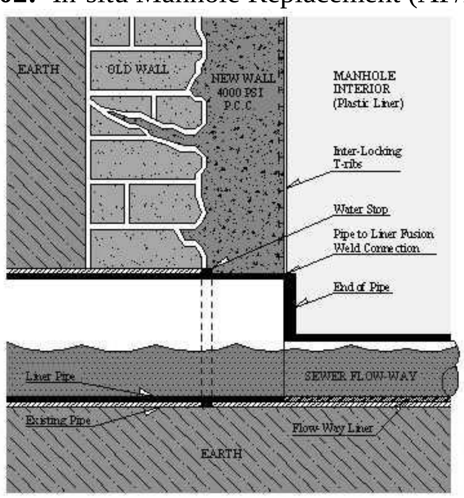
   In-situ manhole replacement begins by removing any loose material or debris. If steps are present, they are cut off at the wall. Pipe extensions through the structure are placed to maintain flow during construction. If significant infiltration is present, it should be controlled by plugging holes with hydraulic cement. After the manhole has been prepared, steel forms are erected inside of the manhole, creating a 3 inch gap between the existing manhole wall and the new form.

   If previous manhole deterioration was the result of chemical attack, a plastic liner may be placed around the exterior of the forms. This plastic liner will eventually form the inside surface of the new wall. The liner has ribs on the back side to anchor it into the concrete.

   After the forms have been erected and the plastic liner secured, if applicable, the annular space between the forms and the existing wall are filled with 4,000 psi concrete. When the concrete has cured sufficiently, the forms are disassembled and removed. If a plastic liner is utilized, any joints in the material are fusion welded to create an airtight seal.

   The bench of the manhole is then overlaid by hand with 10,000 psi concrete and epoxy coated. Sand is spread over the wet epoxy coating to create a non-slip surface. The final step is to remove any pipe extension though the manhole and properly seal around any manhole penetrations.

Figure 14C-3.02: In-situ Manhole Replacement (AP/M Permaform)



Source: AP/M Permaform

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CHAPTER 15
Miscellaneous

Miscellaneous

RESERVED FOR POTENTIAL FUTURE DEVELOPMENT