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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.

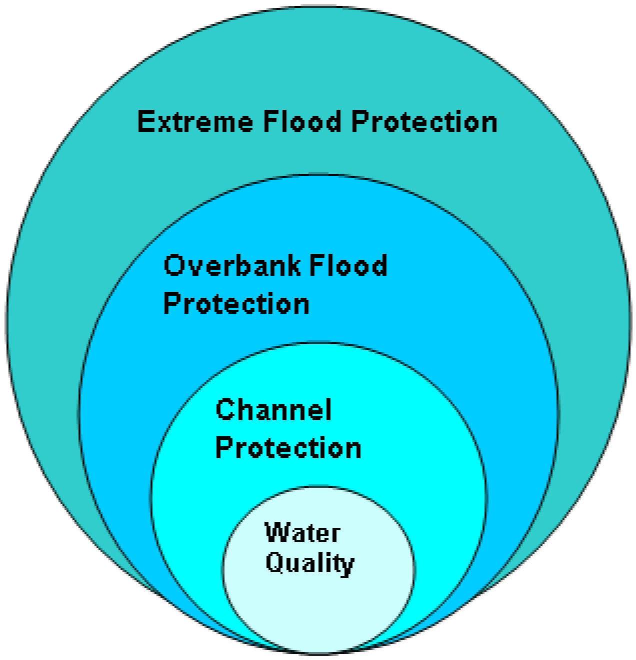
   While this manual does not address stormwater quality requirements (refer to the Iowa Stormwater Management Manual for stormwater quality design), the overall unified sizing criteria is summarized in Table 2A-2.01 and Figure 2A-2.01 below to give the designer an understanding of how each criterion fit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

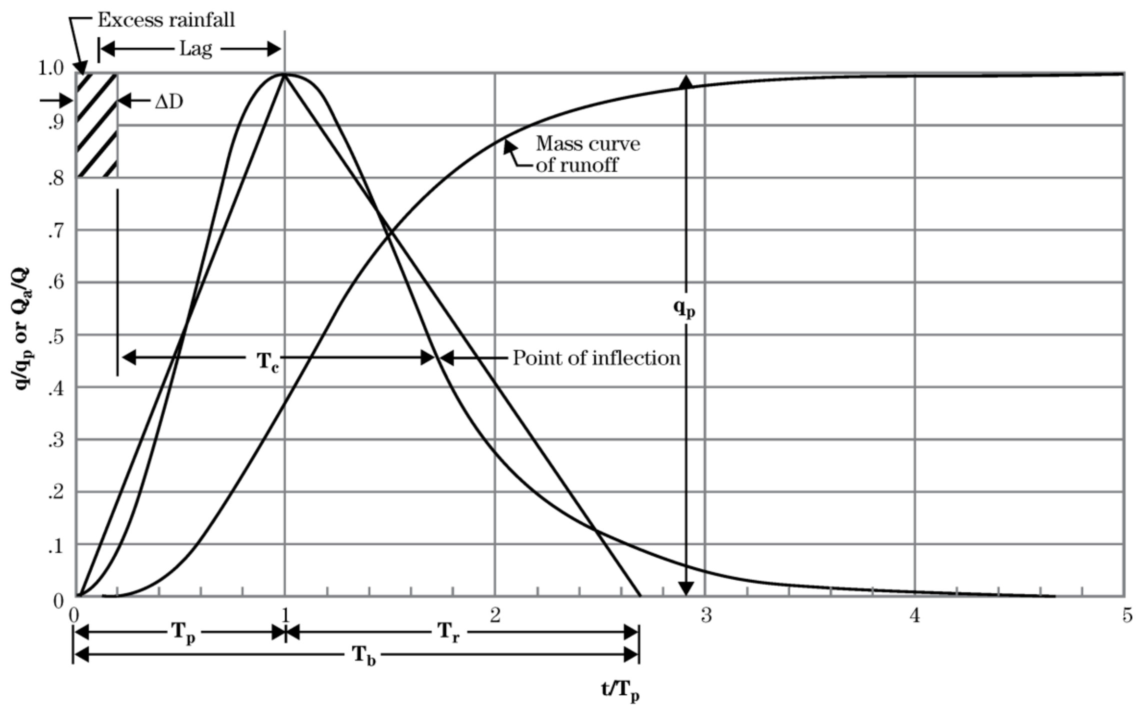
Rainfall Excess: After interception, depression storage, and infiltration have been satisfied, rainfall excess is the remaining water available to produce runoff.

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

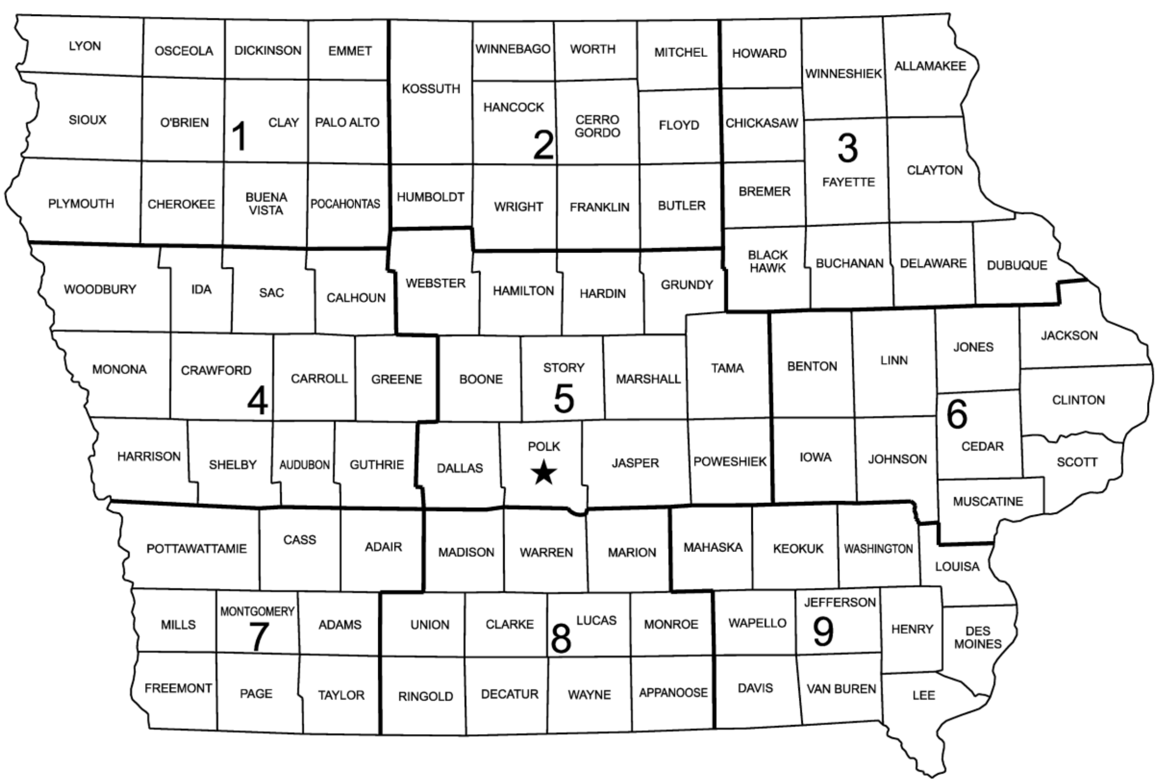
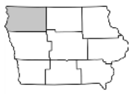


Table 2B-2.02: Section 1 - Northwest Iowa

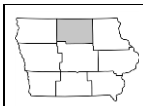
Rainfall Depth and Intensity for Various Return Periods

	**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)

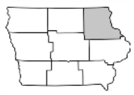
Table 2B-2.03: Section 2 - North Central Iowa

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

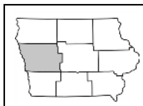
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

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)

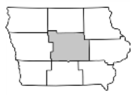
Table 2B-2.05: Section 4 - West Central Iowa
Rainfall Depth and Intensity for Various Return Periods

	**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

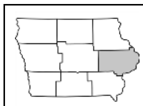
Rainfall Depth and Intensity for Various Return Periods

	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)

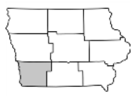
Table 2B-2.07: Section 6 - East Central Iowa

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

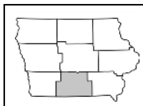
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

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

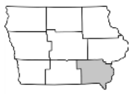
D = Total depth of rainfall for given storm duration (inches)
I = Rainfall intensity for given storm duration (inches/hour)

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

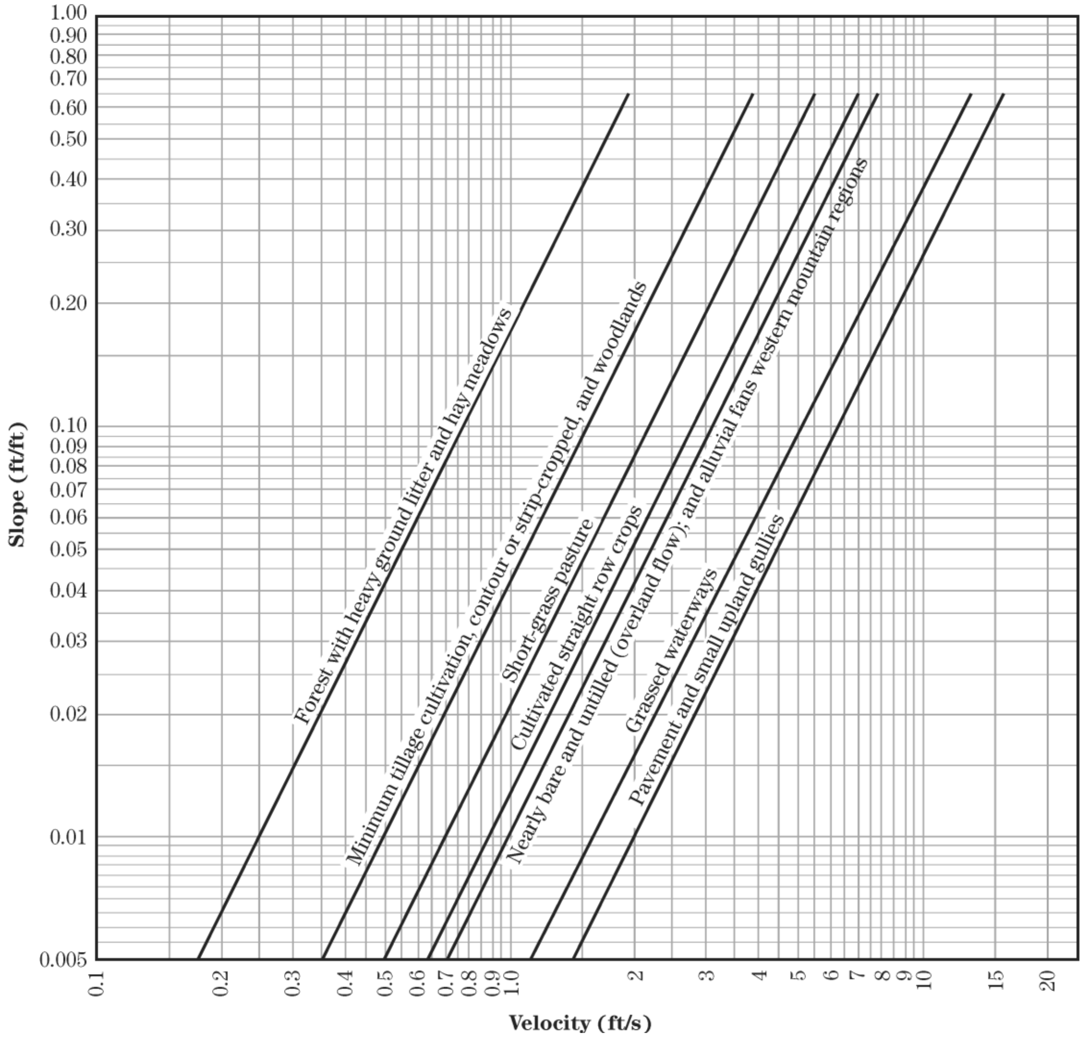
Surface Description	*n*
Smooth Surface (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[1]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[2]	
Light underbrush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

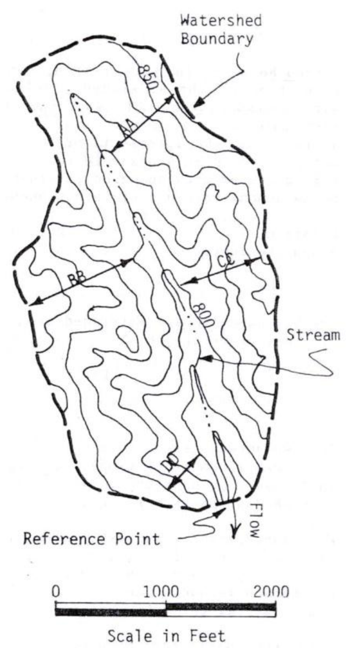
Lag is the delay between the time runoff from a rainfall event over a watershed begins until runoff reaches its maximum peak. Lag is a function of the flow length of the watershed, average land slope of the watershed, and the potential maximum retention of rainfall on the watershed.

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.

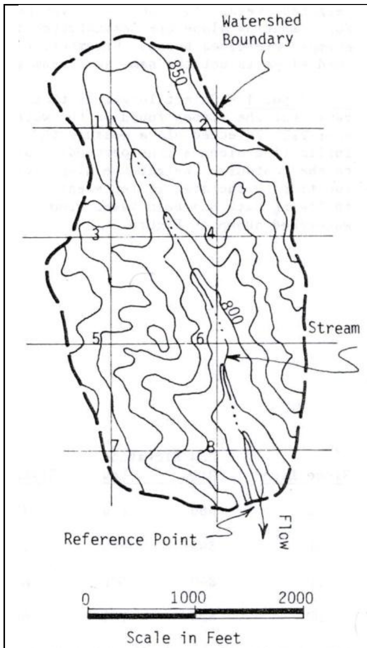
3. **Maximum Potential Retention:** The parameter S represents the potential maximum moisture retention of the soil and is related to soil and cover conditions of the watershed. It is empirically-determined using the SCS curve number (CN), which is provided in Tables 2B-4.03 through 2B-4.05 in Section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

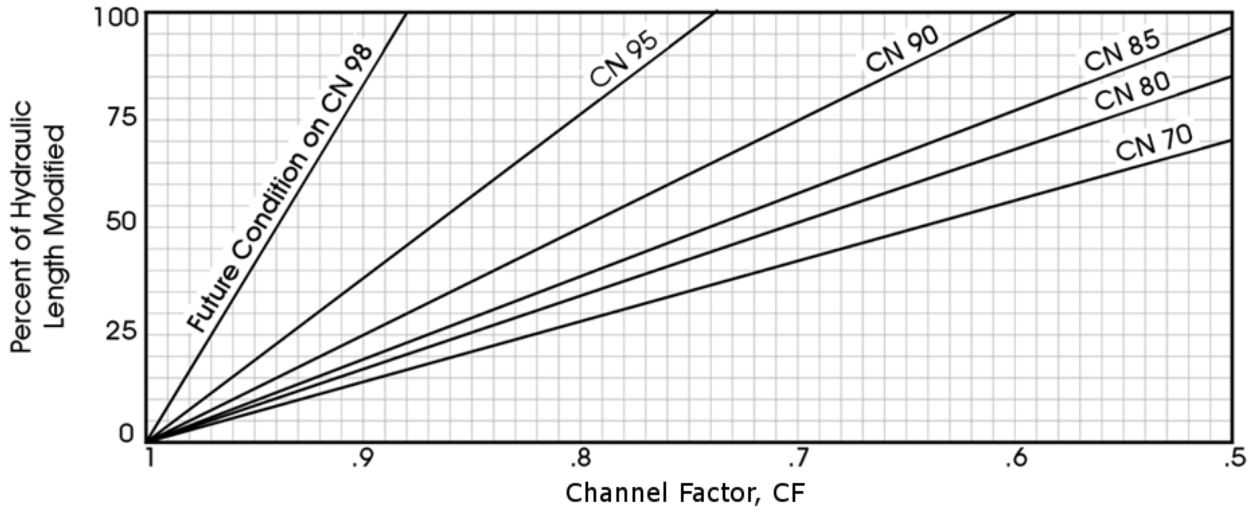
$$T'_c = T_c \times CF \times IF$$ **Equation 2B-3.07**

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

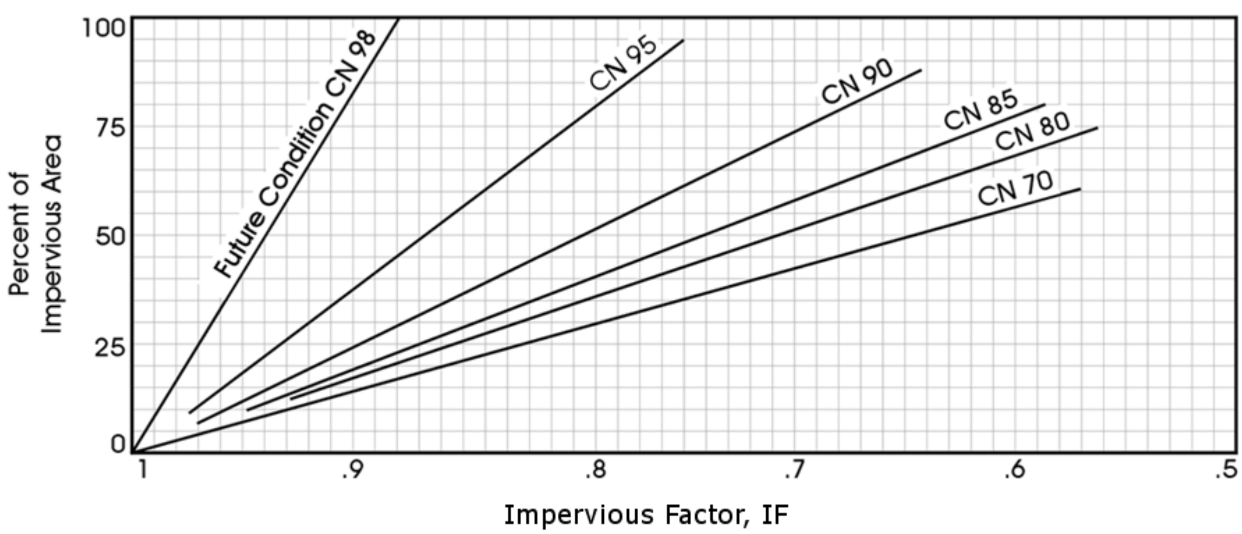
Source: FHWA Hydraulic Engineering Circular No. 19

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.

	Sheet flow (Applicable to T_c only)	Units					
		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.	$T_t = \frac{0.007(nL)^{0.8}}{(\sqrt{P_2})s^{0.4}}$ Compute T_t……………..	hr		+		=	

	Shallow concentrated flow	Units					
		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	Units					
		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	

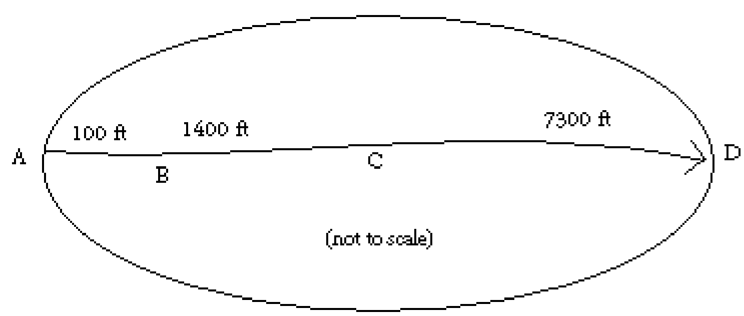
Example 2B-3.01: Time of Concentration

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.

			Unit	Segment 1	Segment 2	Total
	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:

$$Q_T = C i_T A$$ **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.

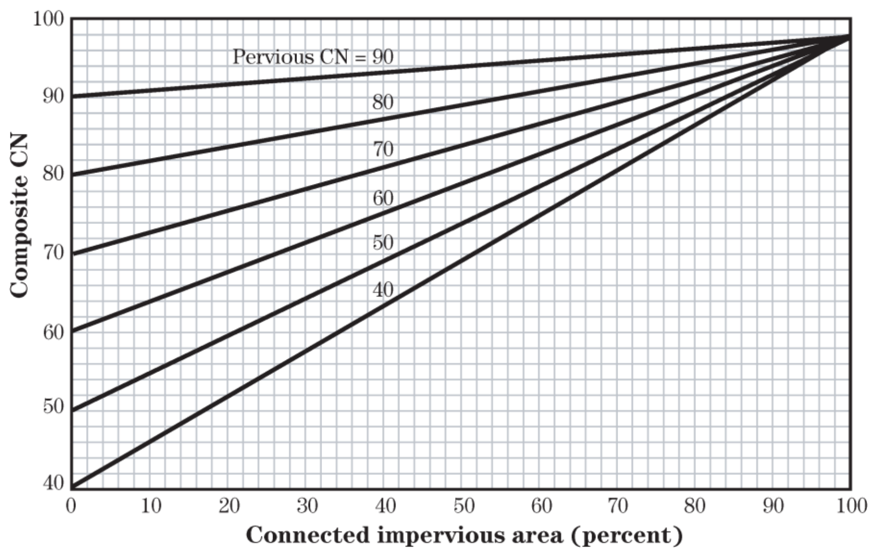
   Several factors, such as the percentage of impervious area and the means of conveying runoff from the impervious areas to the drainage system, should be considered in computing the CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto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

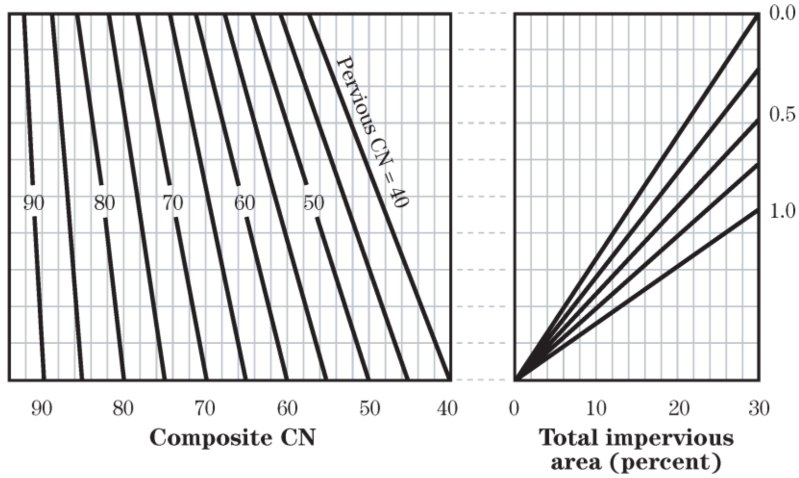
$$CN_c = CN_p + \left(\frac{P_{imp}}{100}\right)(98 - CN_p)$$ **Equation 2B-4.02**

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 $\geq$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

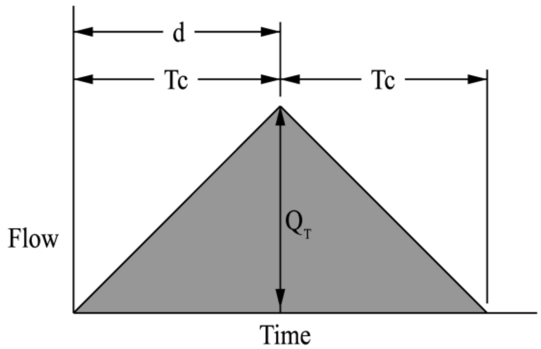
Recurrence Interval (years)	C_a
2 to 10	1.0
25	1.1
50	1.2
100	1.25

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

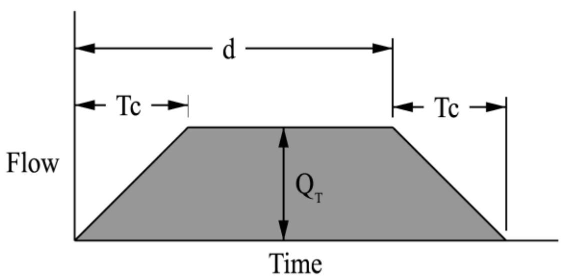


d = Duration of Storm
Q_T = Peak flow rate (=CiA)
T_c = Time of concentration

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.

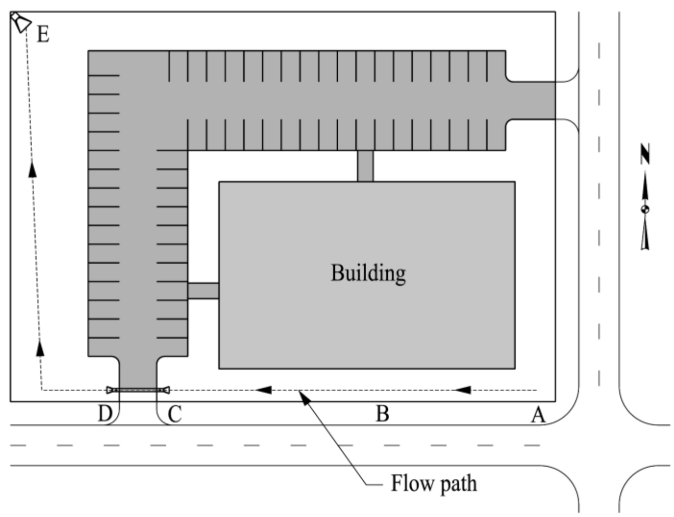
Runoff Examples

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, $V = \frac{1.49\left(r^{2/3}\right)\left(s^{1/2}\right)}{n}$, (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:

   $Q_5 = 0.75 \times 3.96 \times 2.0 = \underline{\underline{5.9 \text{ cfs}}}$

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

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 ☐mi² ☐%	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)	ft³/s/mi²	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**

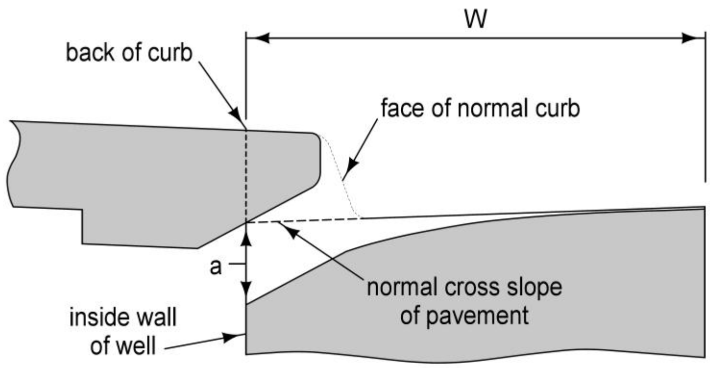
where:

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.

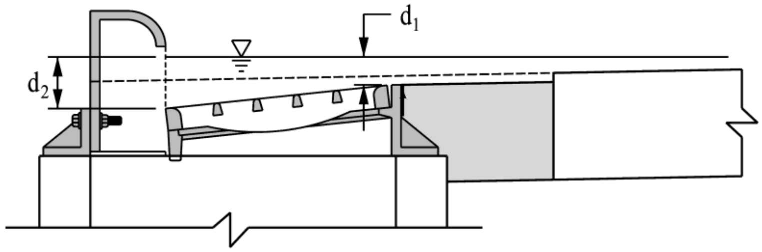
   The capacity of a grate intake operating as a weir is:

   $$Q_i = 3.0Pd^{1.5}$$ **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)

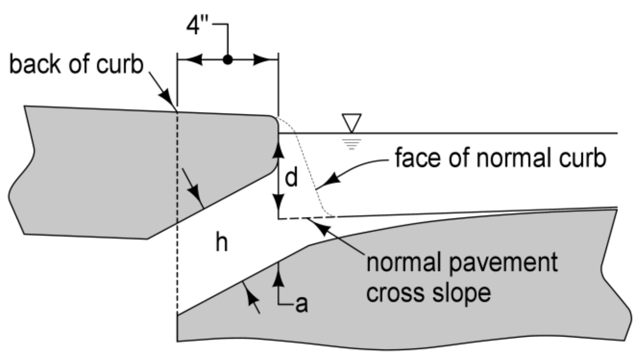
The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation 2C-3.13 for a depressed open-throat intake is:

$$d \leq h + a/12$$ **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

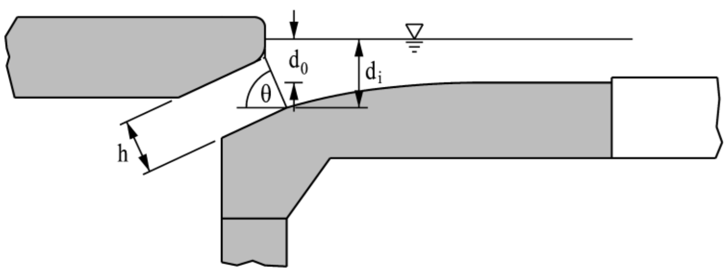
$$Q_i = 0.67A_g\left[2g\left(d_i - \left(\frac{h}{2}\right)\right)\right]^{0.5}$$ **Equation 2C-3.17**

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:**

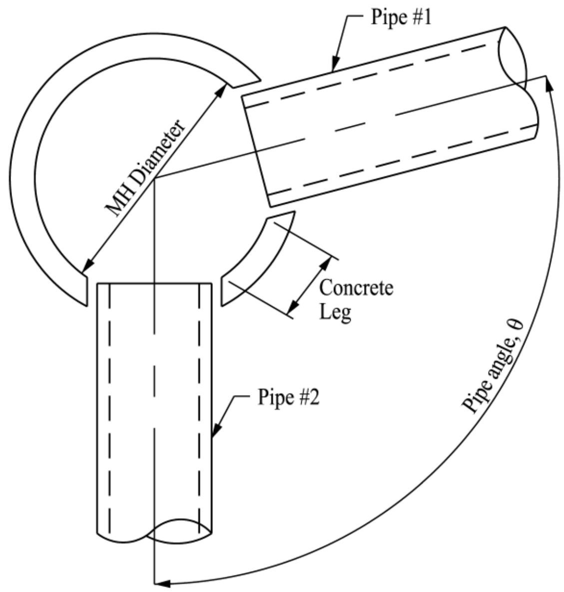
   a. Standard: The minimum size for a manhole is 48 inches in diameter. Jurisdictions require concentric manholes, without built-in steps, with the manhole opening over the centerline of the pipe or on an offset not to exceed 12 inches. Some Jurisdictions may allow for eccentric manholes.

   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**

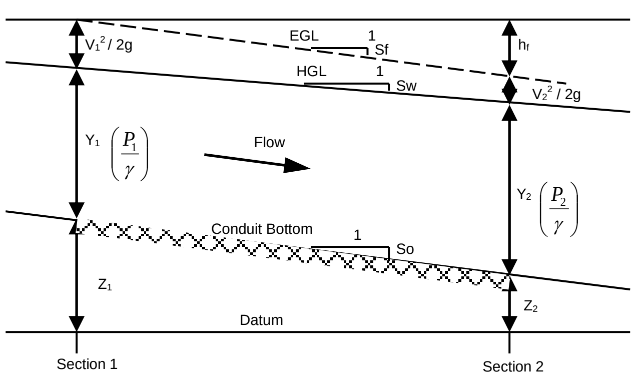
For pressure conduit flow

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_f$$ **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:

$$S_f = 0.453\frac{Q^2n^2}{A^2R^{4/3}}$$ **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.

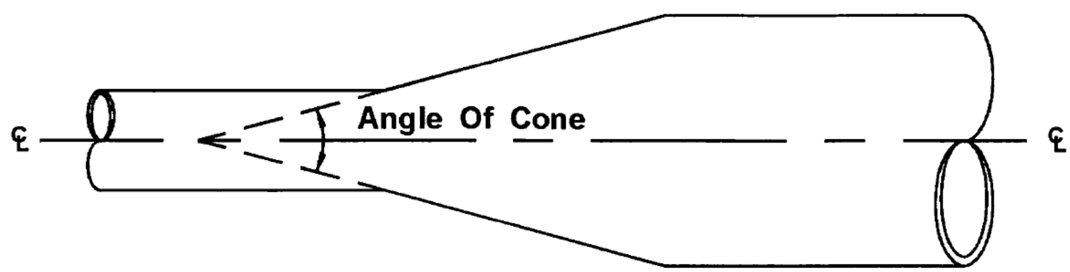
4. **Transition Losses:** A transition is a location where a conduit or channel changes size. Transitions include expansions, contractions, or both. In small storm sewers, transitions should occur within manhole or intake structures. However, in larger storm sewers, or when a specific need arises, expansions 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

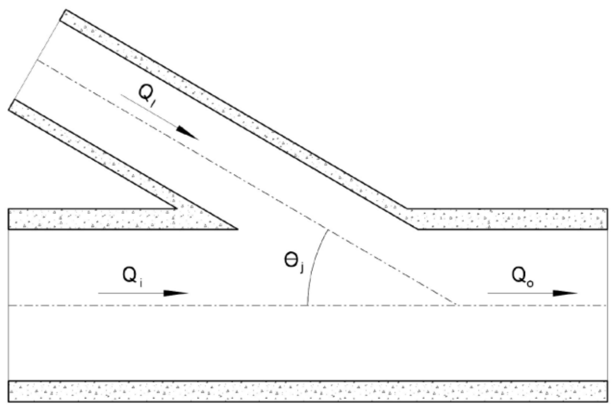
5. **Junction Losses:** A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of a manhole or other structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j = \left\{\frac{\left[(Q_o V_o) - (Q_i V_i) - \left(Q_l V_l \cos\theta_j\right)\right]}{\left[0.5g(A_o + A_i)\right]}\right\} + \frac{V_i^2}{2g} - \frac{V_o^2}{2g}$$ **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**

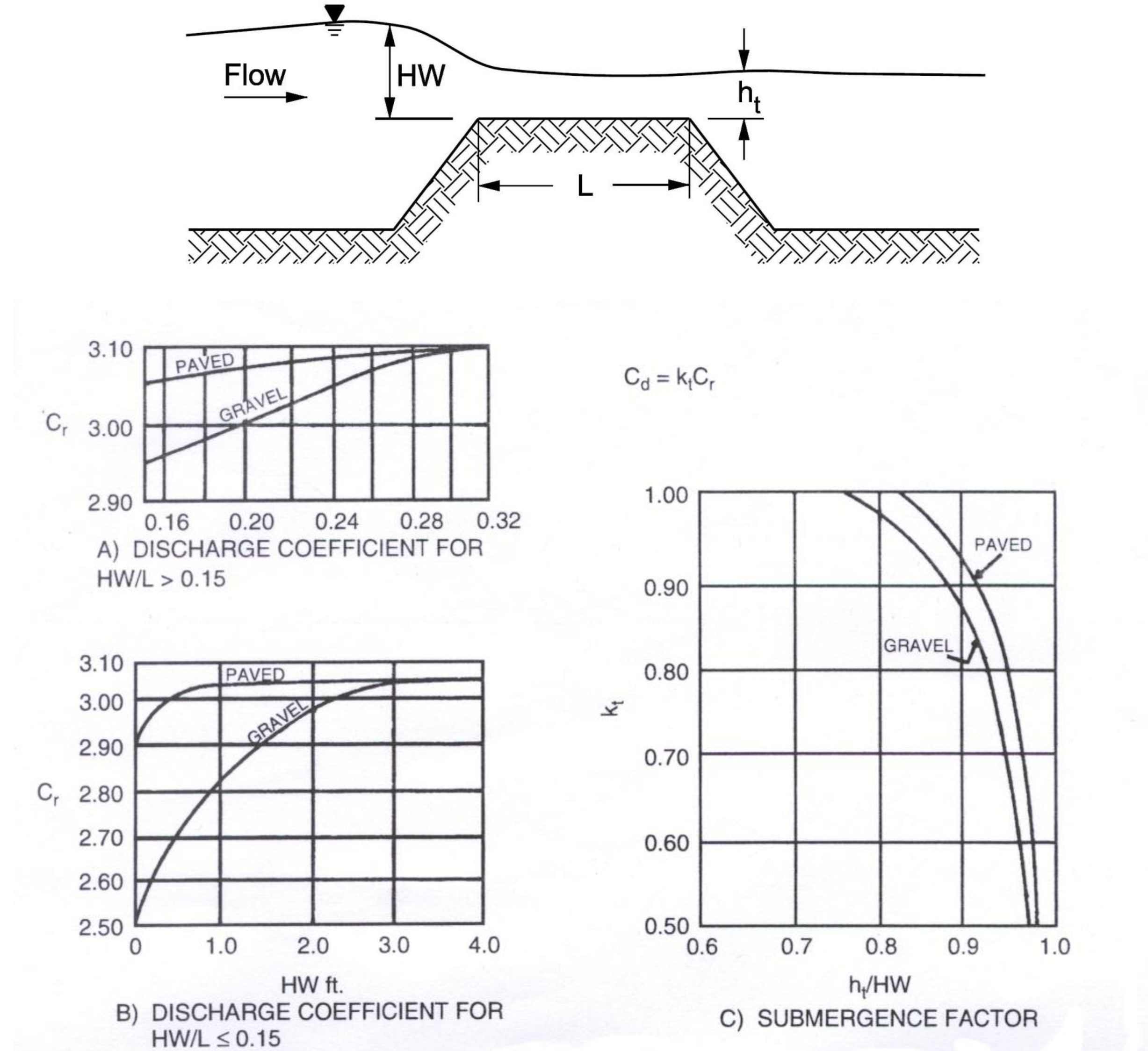
where:

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**

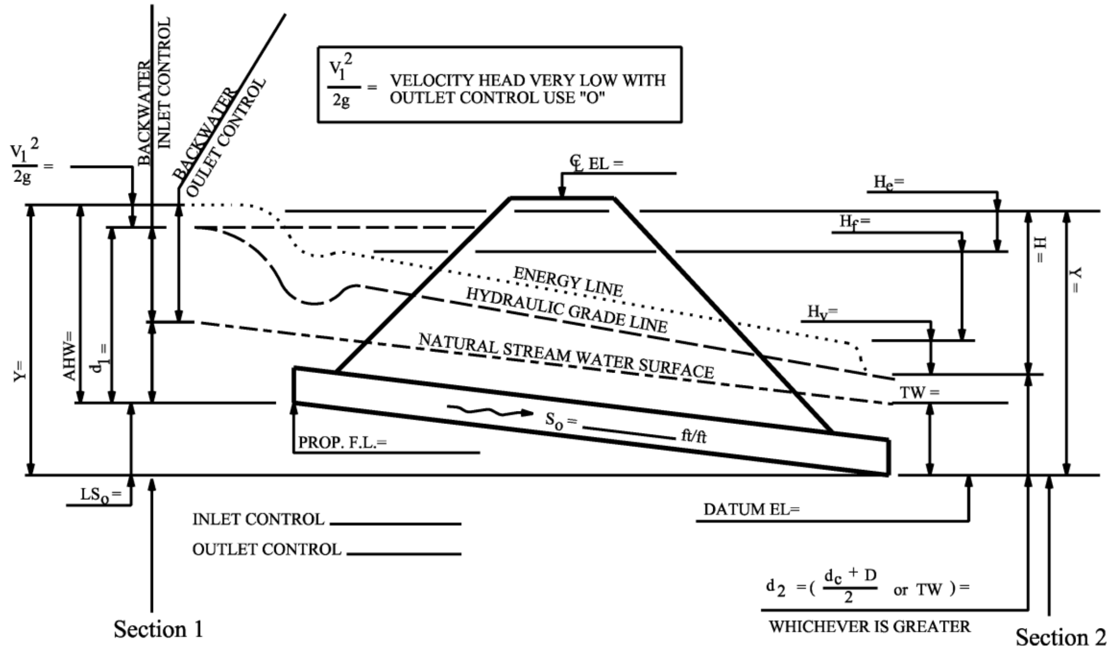
where:

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.

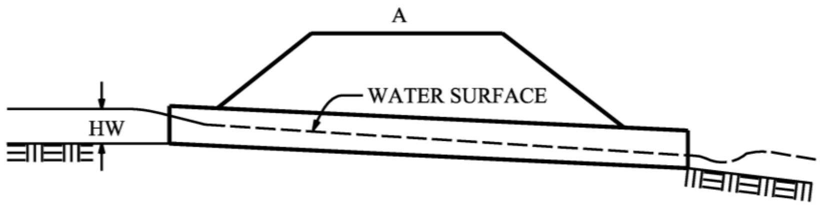
For inlet control, the entrance characteristics of the culvert are such that the entrance headlosses are predominant in determining the headwater of the culvert. The barrel will carry water through the culvert more efficiently than the water can enter the culvert. Proper culvert design and analysis requires 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.

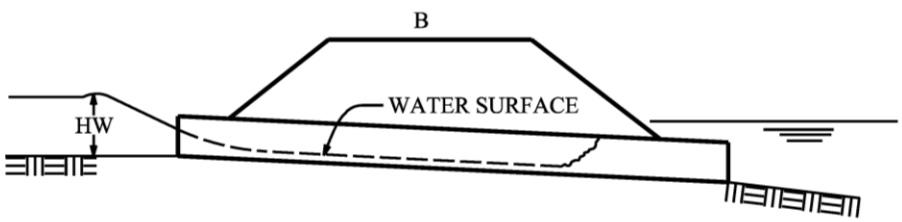
Figure 2E-2.01A: Inlet/Outlet Unsubmerged



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.

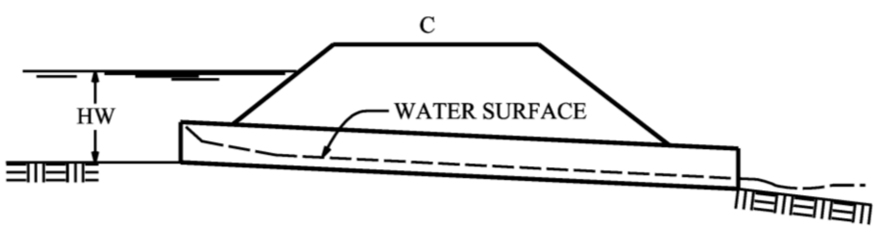
Figure 2E-2.01B: Outlet Submerged, Inlet Unsubmerged



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.

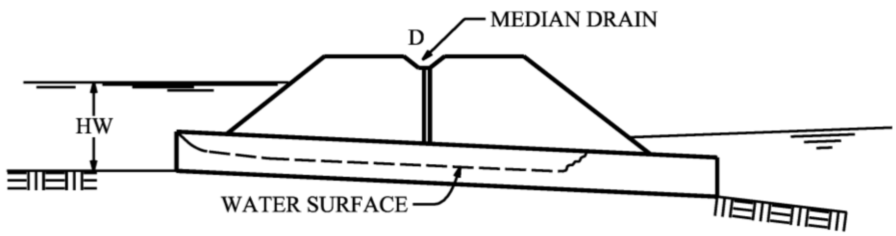
Figure 2E-2.01C: Inlet Submerged



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.

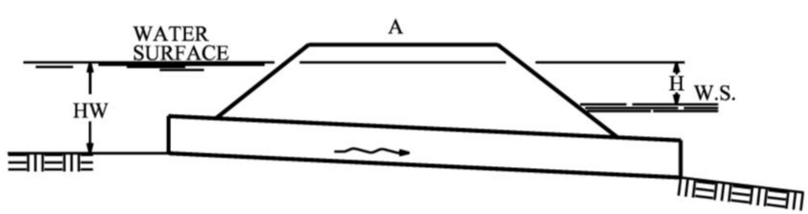
   The barrel area and barrel shape are self-explanatory. The barrel length is the total culvert length from the entrance to the exit of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of the barrel length is usually necessary to begin the 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.

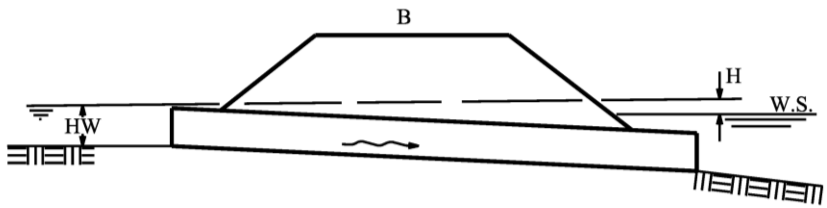
Figure 2E-2.02A: Inlet/Outlet Submerged



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.

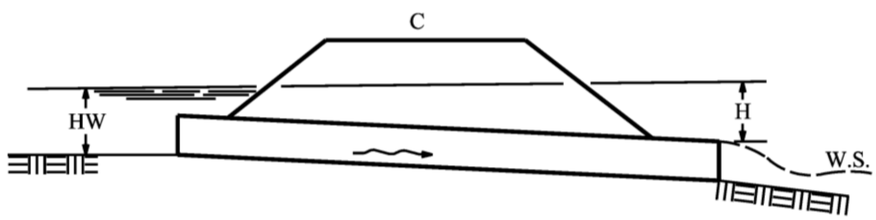
Figure 2E-2.02B: Outlet Submerged, Inlet Unsubmerged



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.

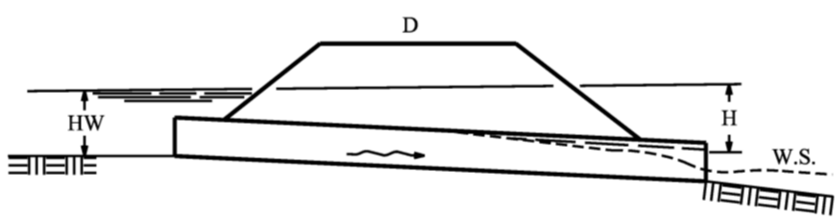
Figure 2E-2.02C: Inlet Submerged, Outlet Unsubmerged



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.

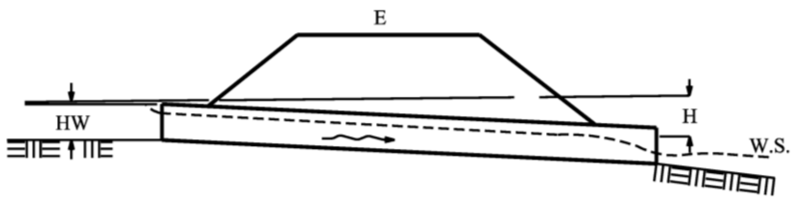
Figure 2E-2.02D: Inlet Submerged, Outlet Partially Submerged



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**

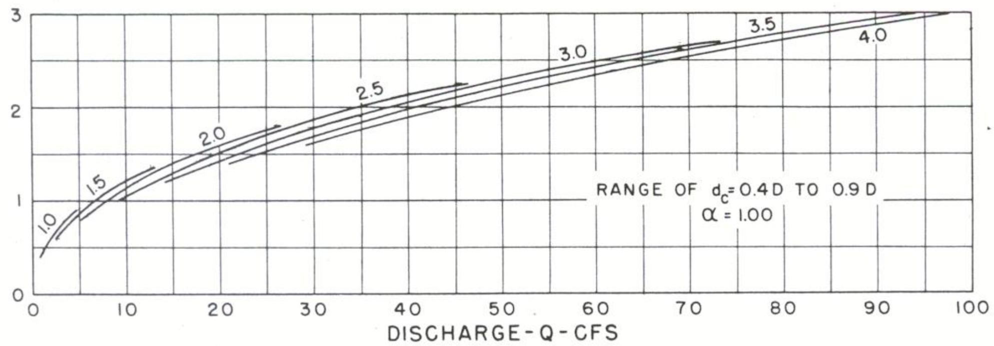
Where:

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.

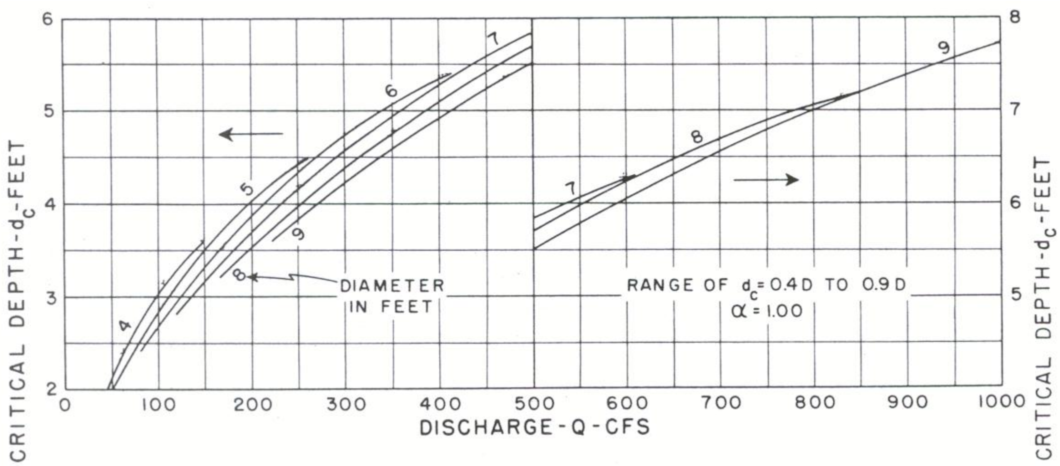
Step 5: Calculate exit velocity and expected streambed scour to determine if an energy dissipater is needed. The stream degradation may be a pre-existing condition, and the reasons and rate of degradation need to be determined. The culvert cross-sectional area may need to be increased and culvert invert 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



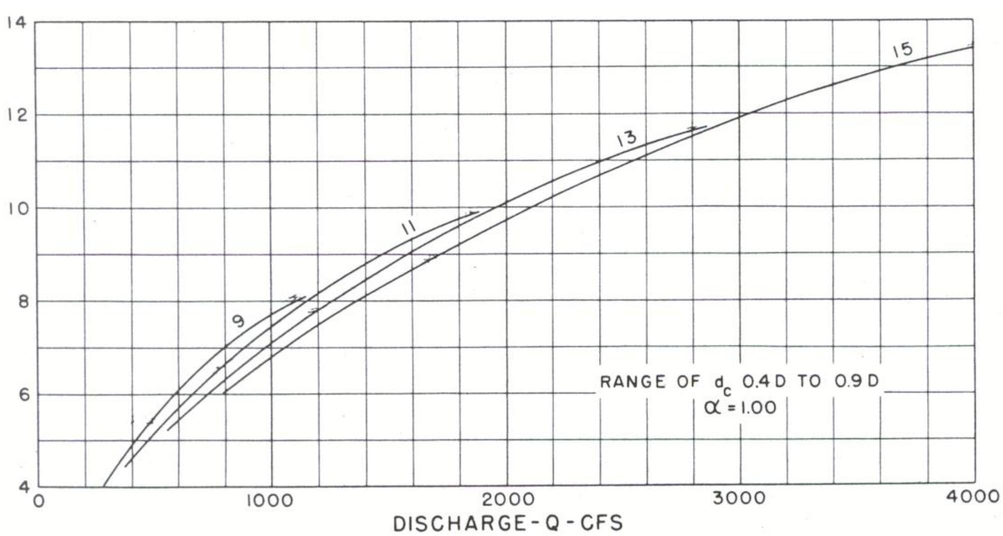
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.03B: Critical Depth Circular Pipe, Discharge = 0 to 1000 cfs



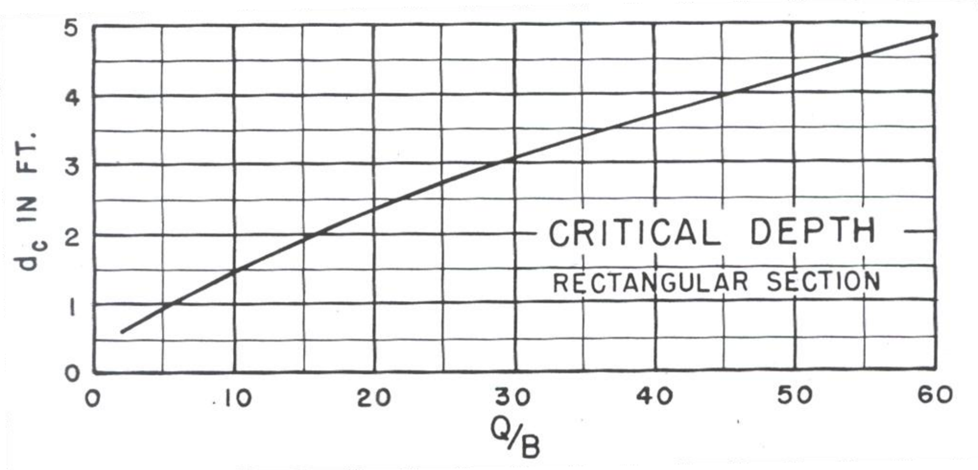
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.03C: Critical Depth Circular Pipe, Discharge = 0 to 4000 cfs



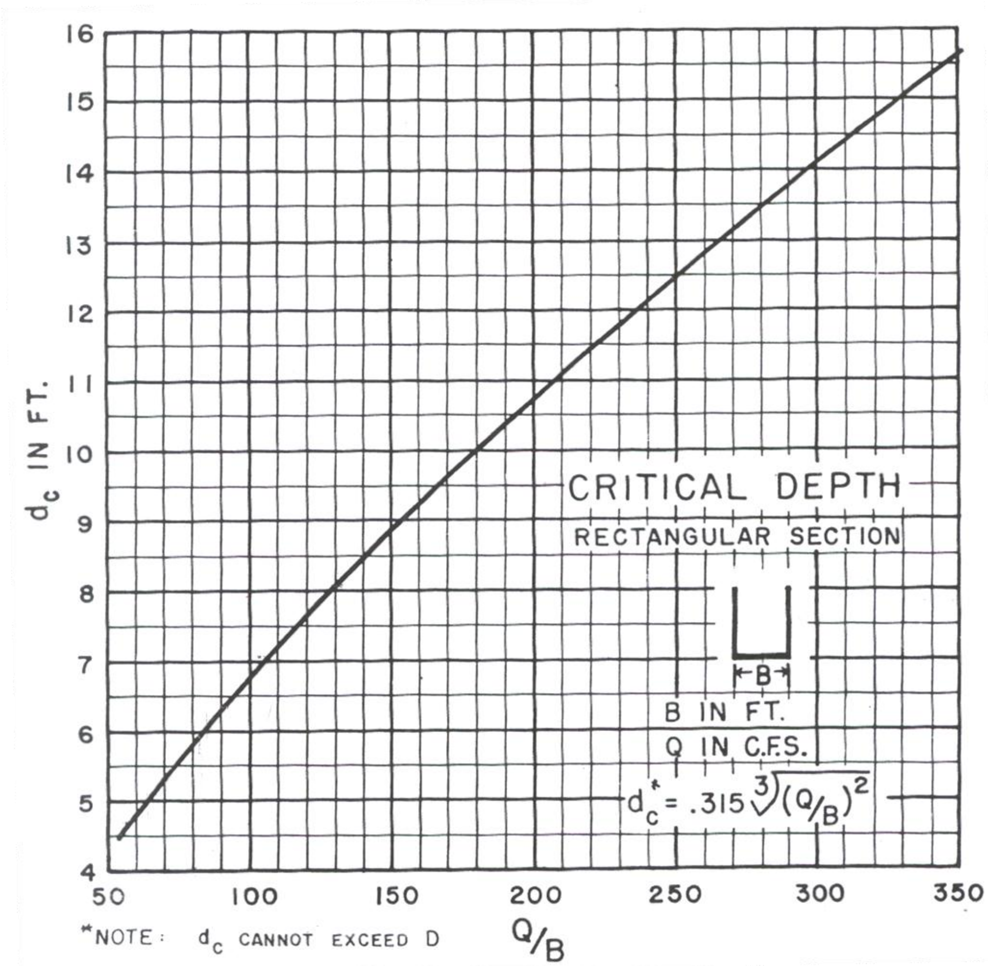
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.04A: Critical Depth Box Culvert, Q/B = 0 to 60 cfs



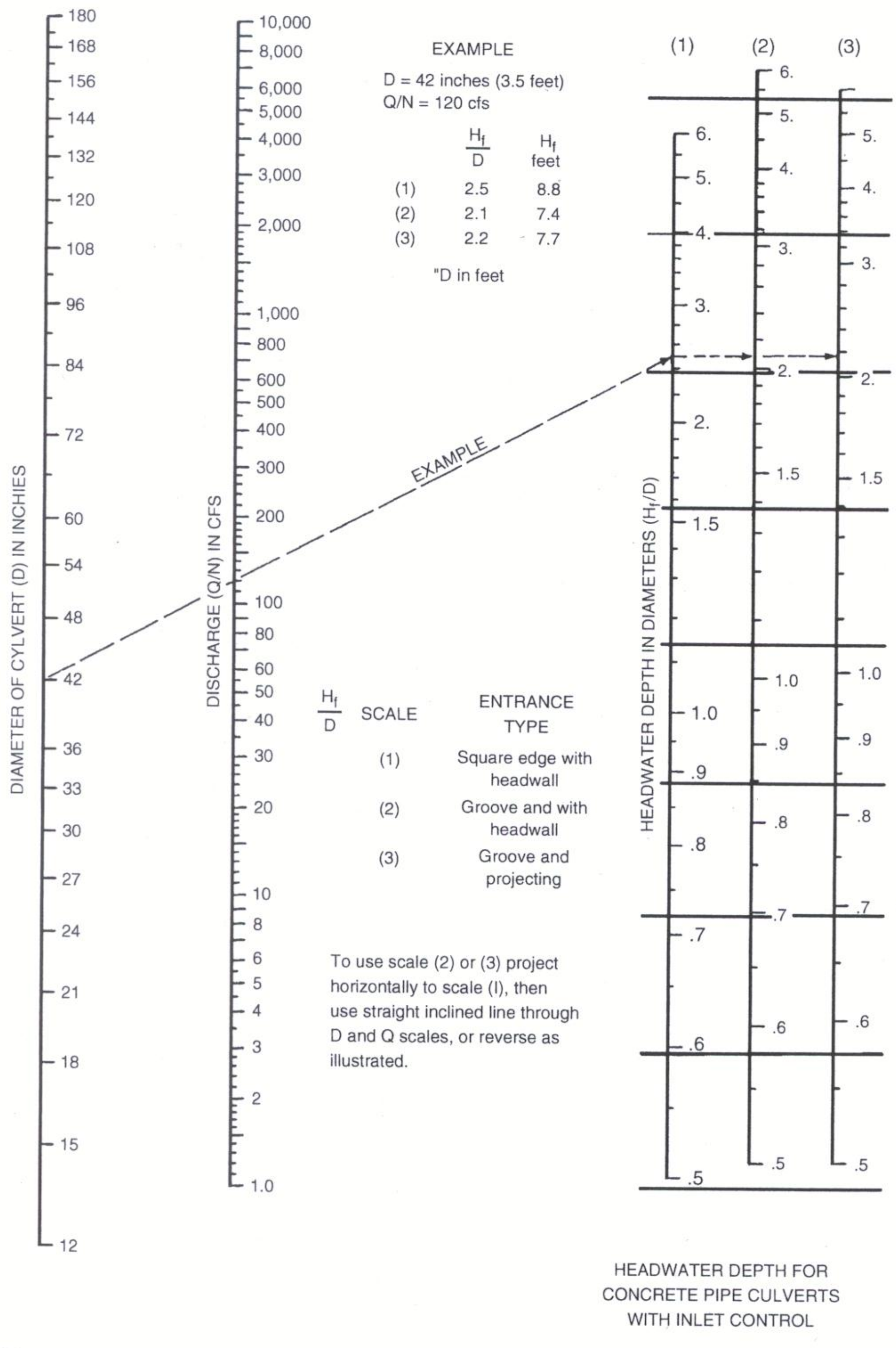
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.04B: Critical Depth Box Culvert, Q/B = 50 to 350 cfs



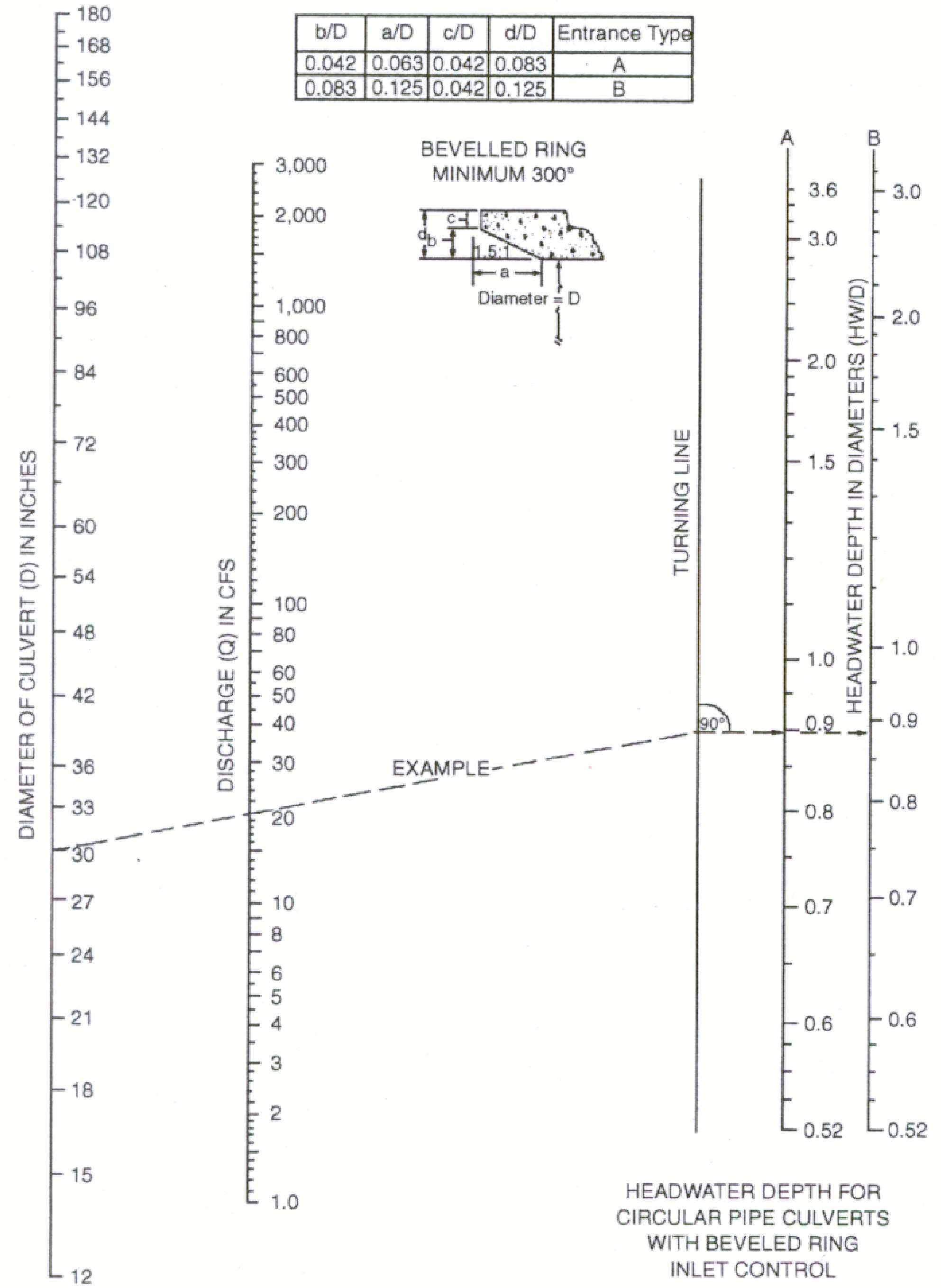
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.05: Inlet Control Nomograph



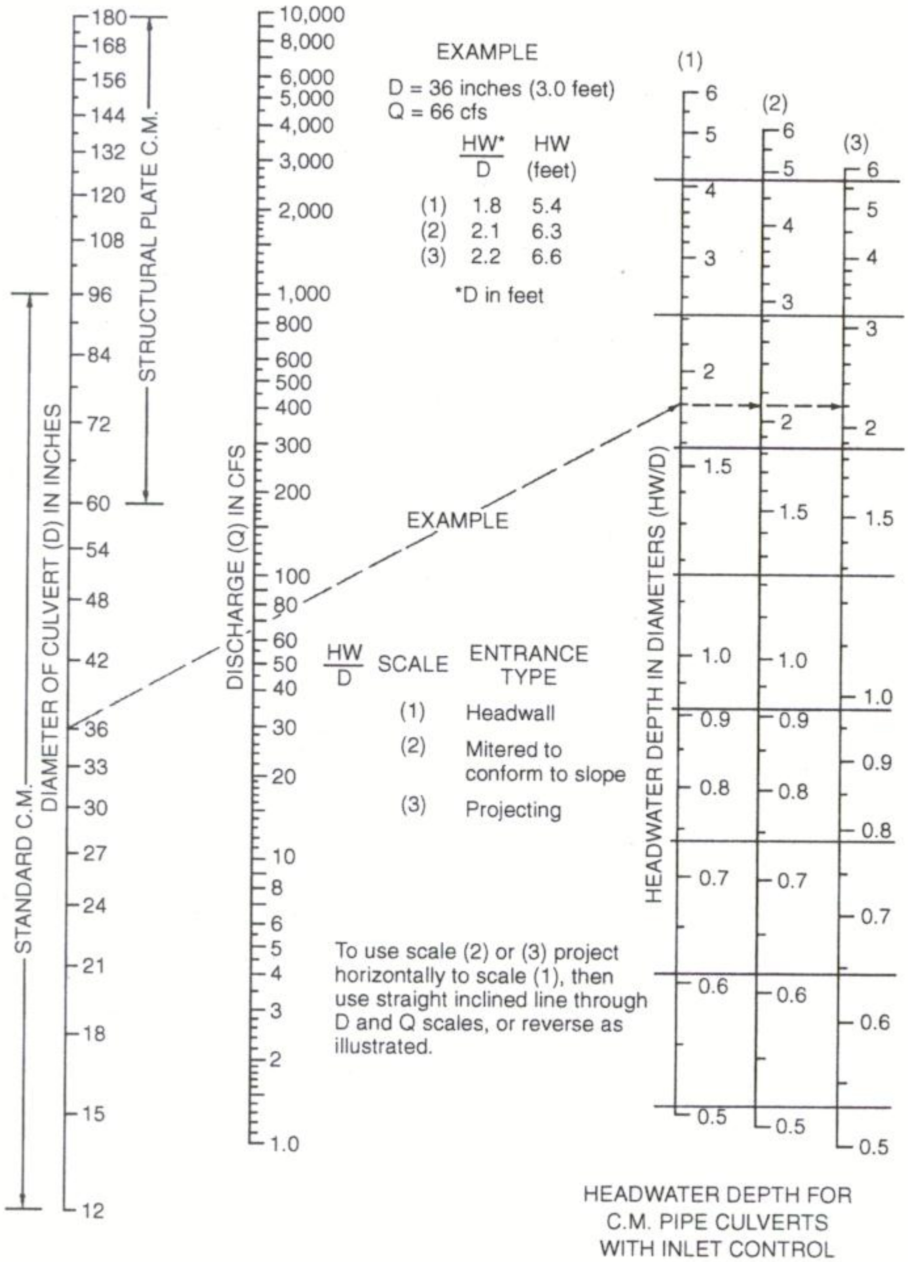
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.06: Inlet Control Nomograph



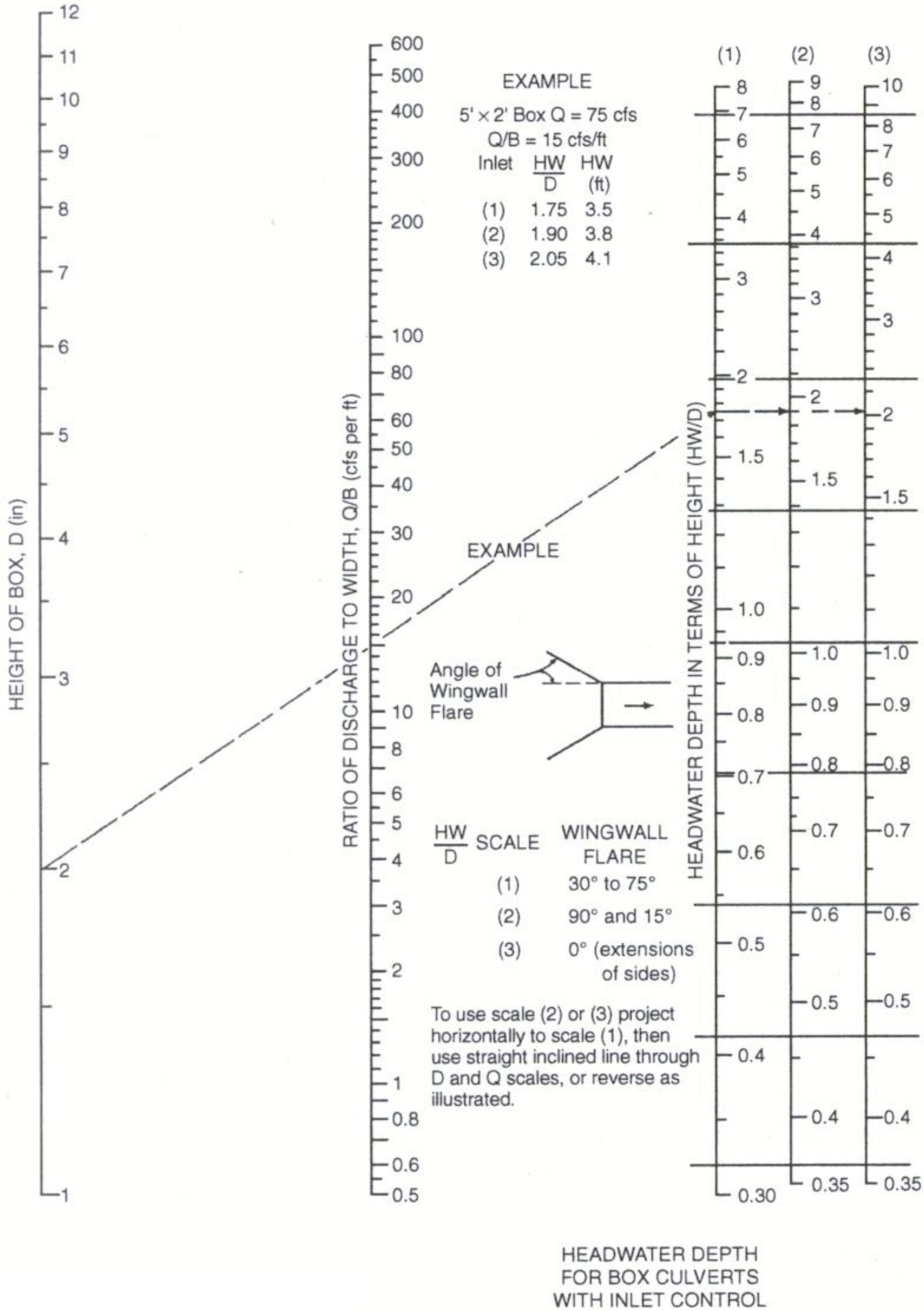
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.07: Inlet Control Nomograph



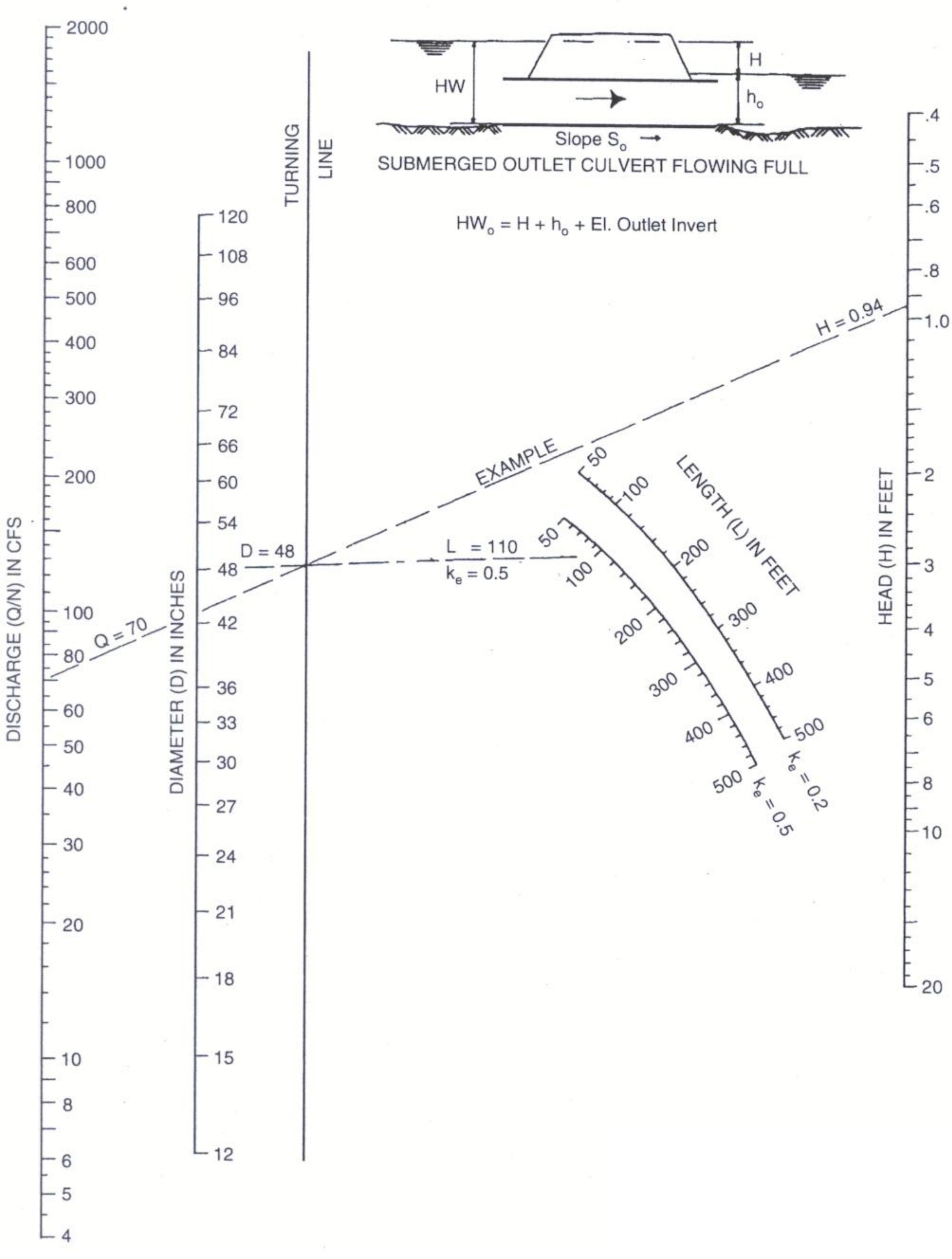
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.08: Inlet Control Nomograph



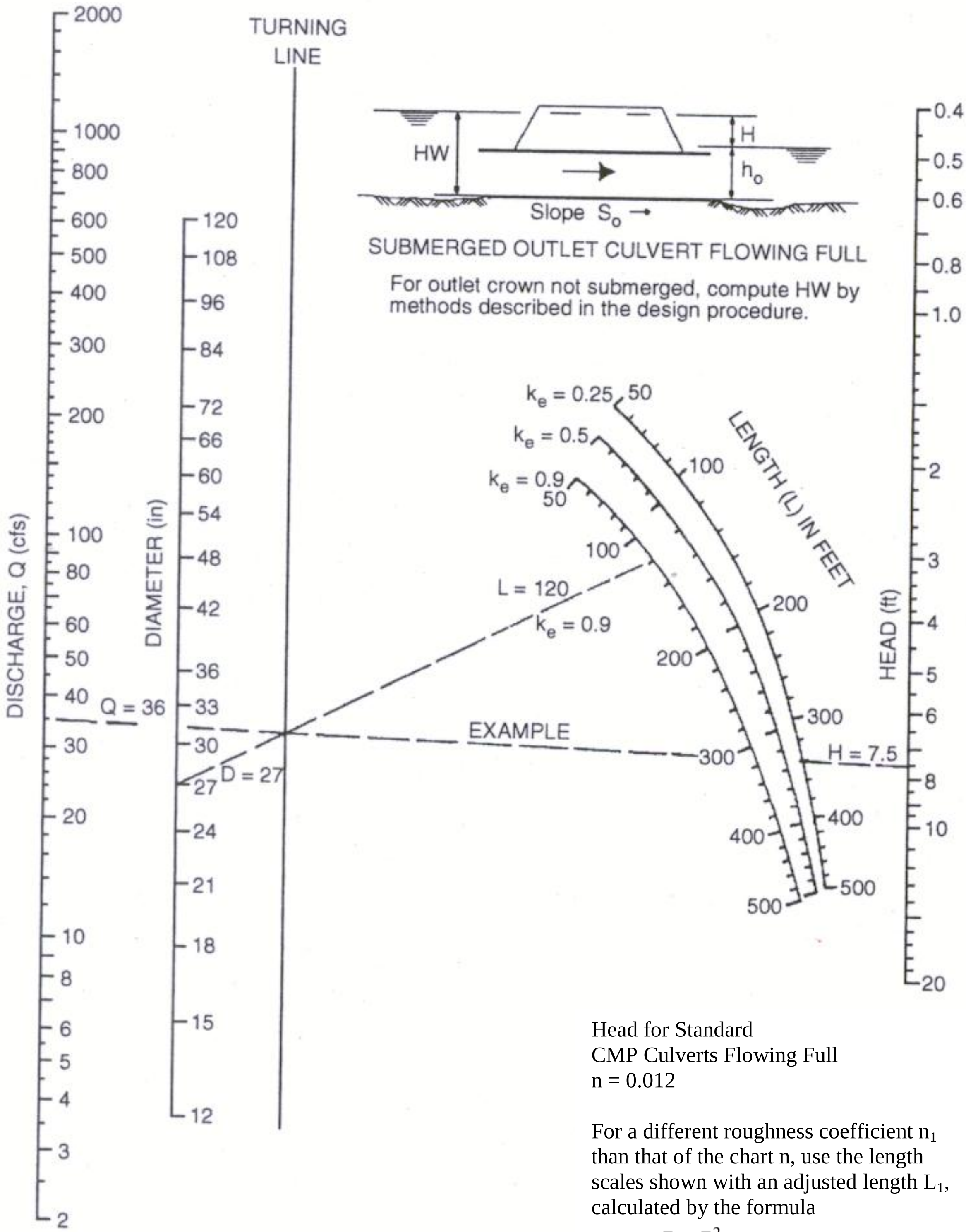
Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.09: Outlet Control Nomograph



Source: *Hydraulic Design of Highway Culverts*, FHWA

Figure 2E-2.10: Outlet Control Nomograph



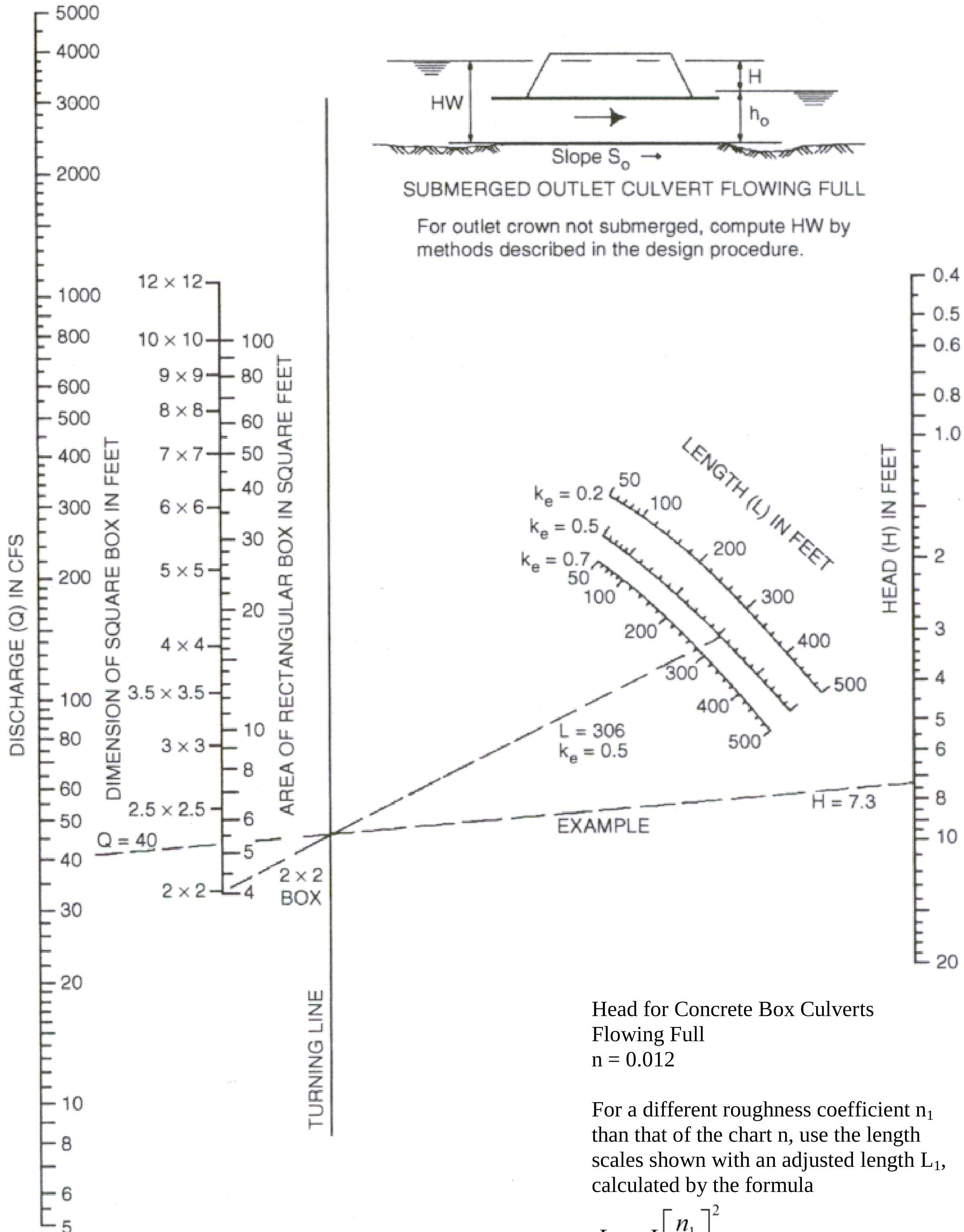
Head for Standard
CMP Culverts Flowing Full
n = 0.012

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 = $[(14 \text{ sq ft} \times 4) / 3.14]^{0.5}$ 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:

$$HW = H + h_o - LS$$

$$HW = 5.2 + 4.0 - (100) \times (0.0015) = 9.05 \text{ ft}$$

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

CULVERT DESIGN FORM

PROJECT: ______________________

STATION: ______________

SHEET ______ OF ______

DESIGNER/DATE: __________ / ______

REVIEWER/DATE: __________ / ______

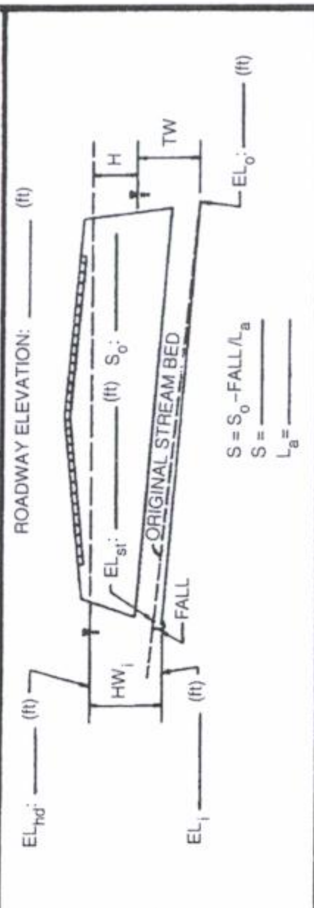
HYDROLOGICAL DATA

- ☐ METHOD: ______________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ______________
- ☐ ROUTING: ________ ☐ OTHER: ________

SEE ADD'L. SHTS.

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)



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)	TW (5)	OUTLET CONTROL 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.

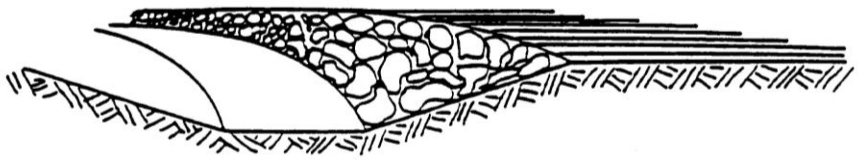
2. Vertical drops should be constructed of concrete or gabions (see Figure 2F-1.02 for example).

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.

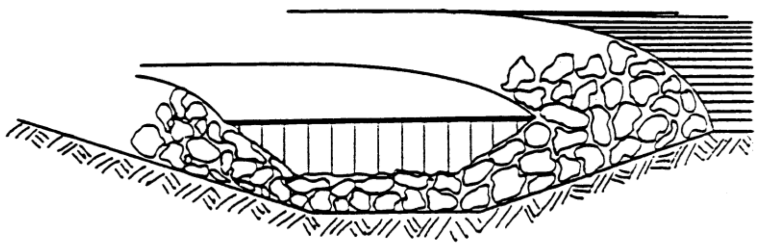
4. At drop structures, both the channel bottom and banks should be protected from erosion.

Figure 2F-1.01: Sample Channel Cross-Sections

TYPE 1 CHANNEL



TYPE 2 CHANNEL



TYPE 3 CHANNEL

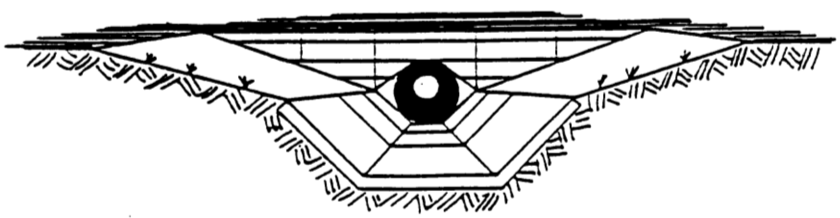
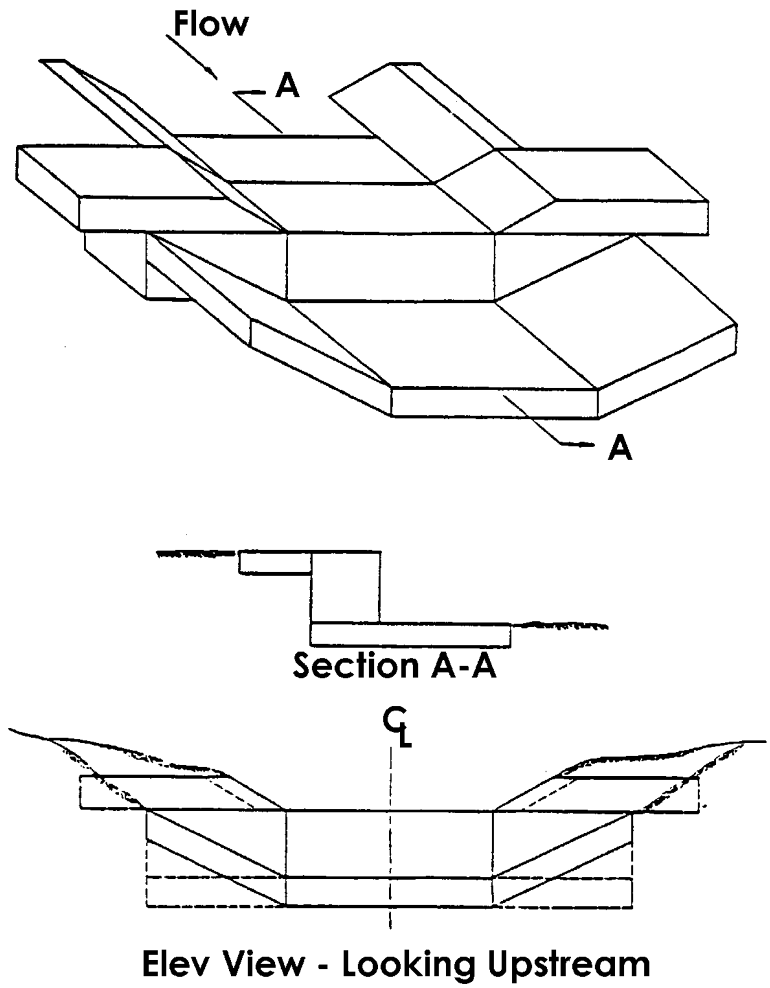


Figure 2F-1.02: Example Drop Structure for Open Channel Flow



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

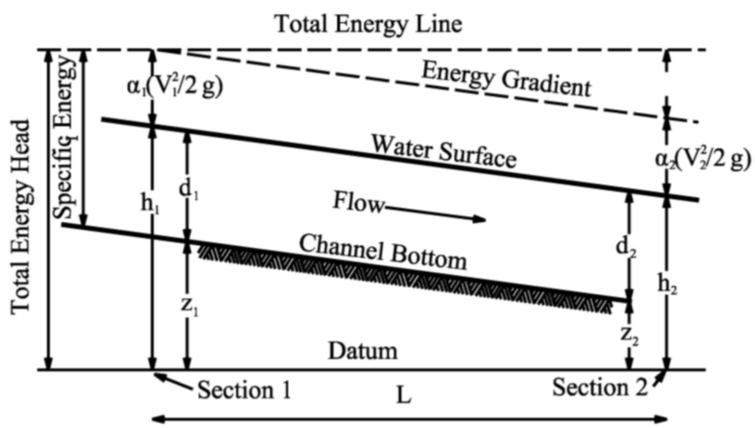
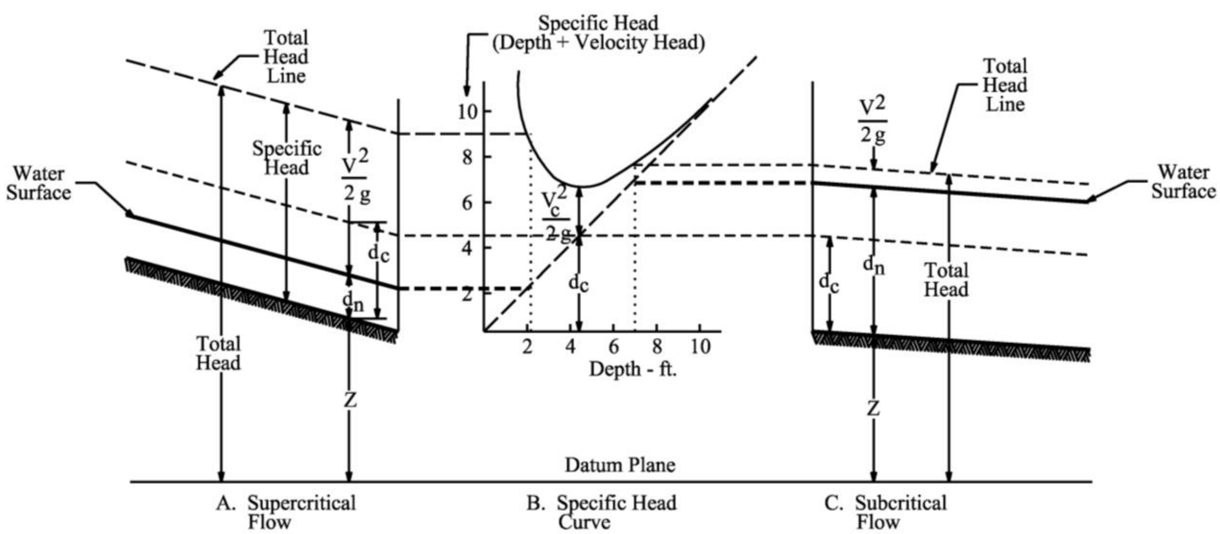


Figure 2F-2.02: Definition Sketch of Specific Head



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.

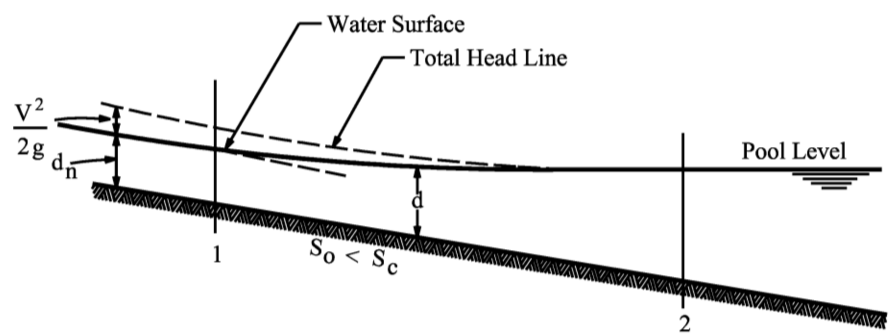
With subcritical flow, a change in channel shape, slope, or roughness affects the flow for a considerable distance upstream, and thus the flow is said to be under downstream control. If an obstruction, such as a culvert, causes ponding, the water surface above the obstruction will be a smooth curve asymptotic 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

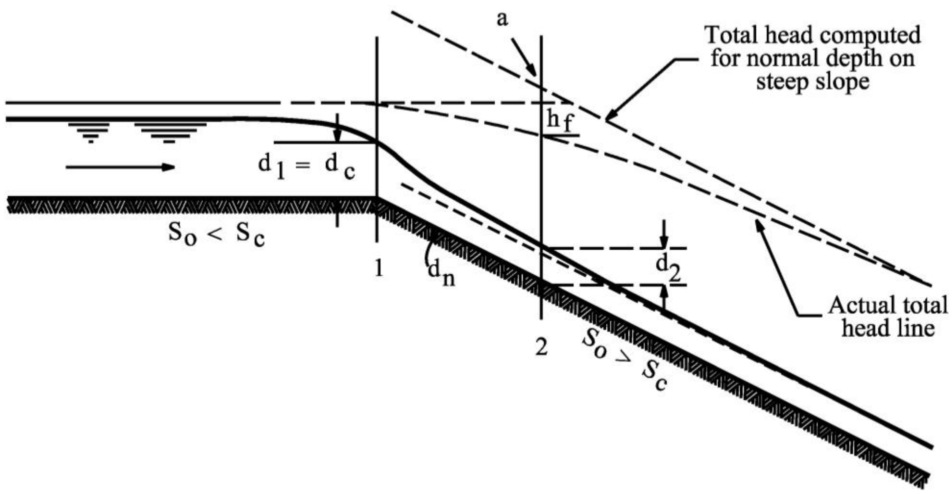


Source: *Design Charts for Open-Channel Flow,* FHWA

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.

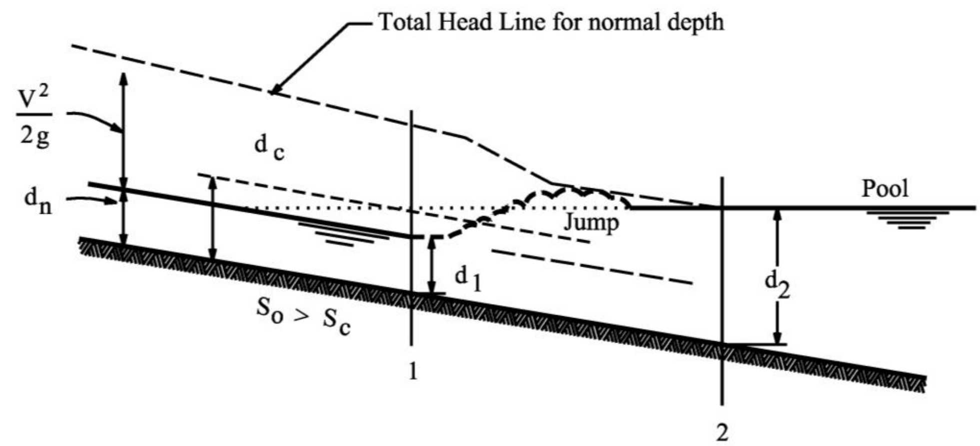
Figure 2F-2.04: Water Surface Profile in Changing from Subcritical to Supercritical Channel Slope



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.

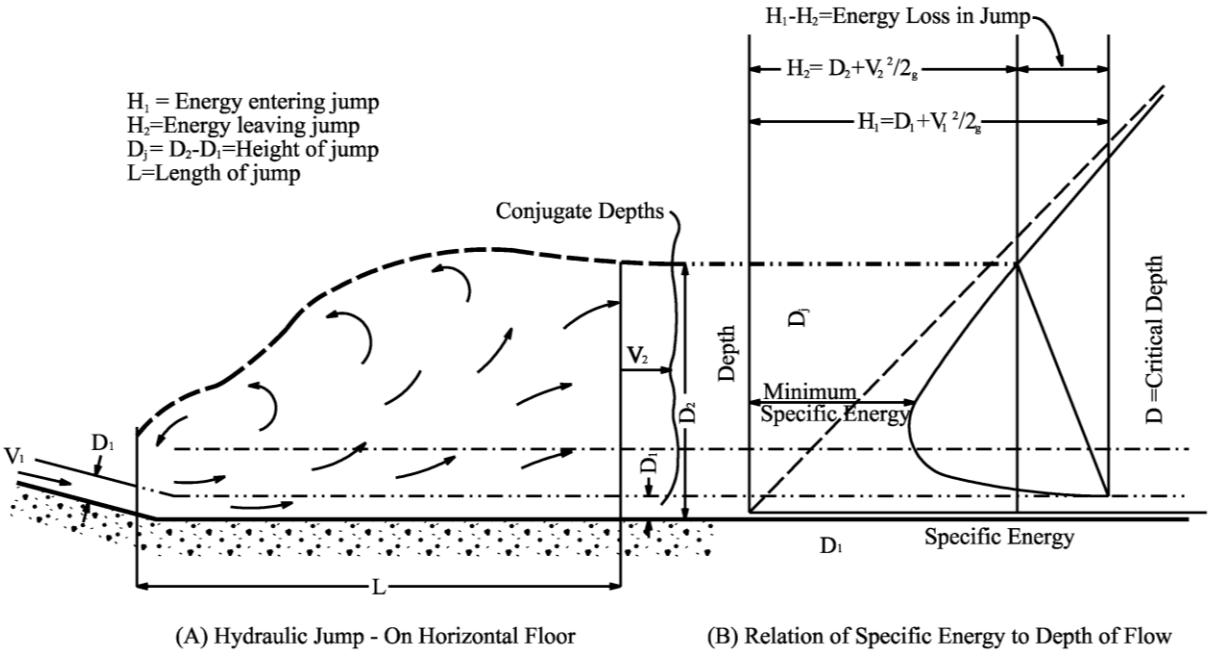
   b. A short distance upstream from channel constrictions such as those caused by bridge piers.

   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.

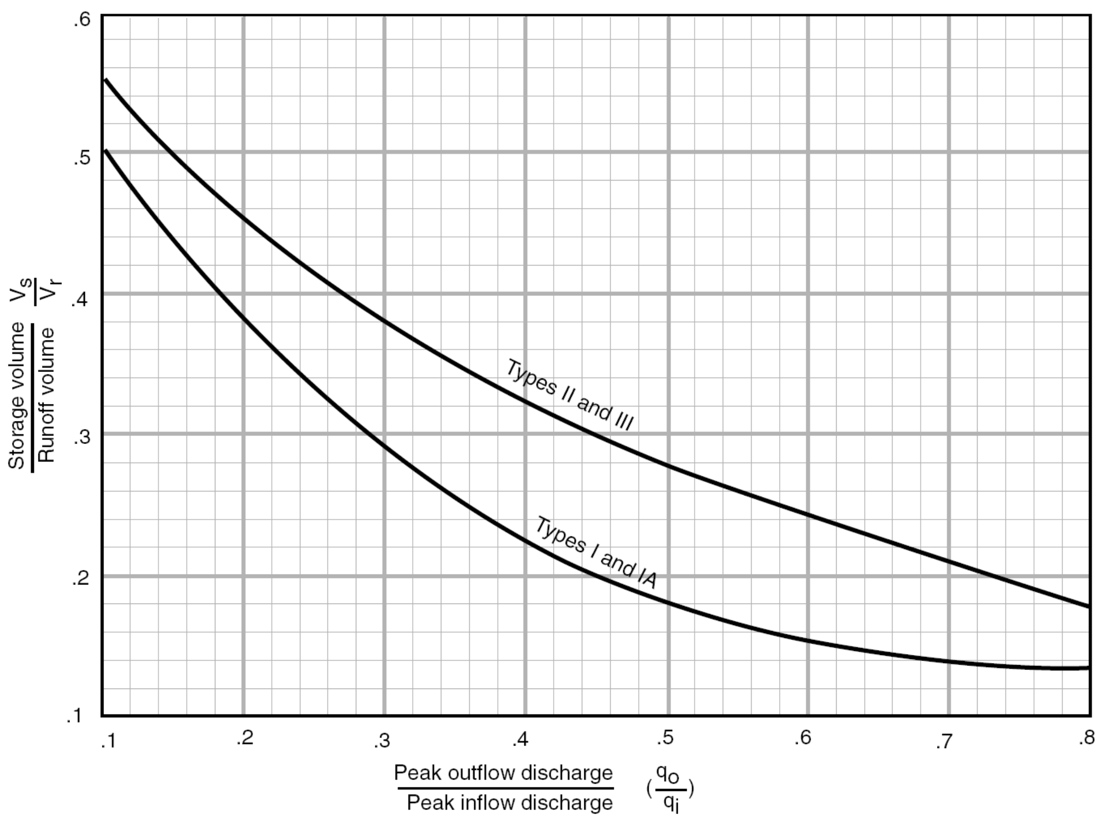
    b. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that:
        1) Each stage requires a design storm and a computation of the storage required for it.
        2) The discharge of the upper stage(s) includes the discharge of the lower stage(s).

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.