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FDM 13-1-1 Drainage Practice Background

February 18, 2020

1.1 Introduction

The Chief of the Design Standards and Oversight Section is the originator of this chapter.

1.2 General

Drainage has long been recognized as one of the primary considerations of highway construction. Its importance can be noted from the cost involved in providing drainage facilities for the highway, and for this reason alone a careful and scientific approach to drainage design should be taken. The purpose of this chapter is to provide a guide to existing standard procedures for drainage design throughout the state. The goal of design is to plan optimum drainage facilities considering function versus cost while meeting environmental requirements.

The methods of hydrologic and hydraulic analysis provided in this chapter will give the designer information necessary for drainage analysis. Experience and sound engineering judgment are not to be ignored and may at times differ from results obtained using methods in this chapter. Careful weighing of experience, judgment, and procedure are necessary for optimal drainage design. Terminology that is unique to this chapter and to "drainage" in general is defined in Attachment 1.1.

1.3 Basic Statewide Practice

In designing highway drainage systems, the three major considerations are:

1. The safety of the traveling public;
2. The use of sound engineering practices to economically protect and drain the highway;
3. In accordance with reasonable interpretation of the law, the protection of private property from flooding, water-soaking, or other damage.

In general, the hydraulic adequacy of pipe culverts shall be determined by the region based on sound hydrologic and hydraulic techniques and performance records at the same or similar locations. No improvement in the drainage of areas outside the right-of-way should be considered unless the state would benefit thereby, or the project is financed by others.

1.4 Design Responsibility

The Bureau of Structures (BOS) is responsible for the hydraulic and structural adequacy of all cast-in-place and precast box culverts and bridges. Preliminary hydrologic and hydraulic computations for such structures shall be performed by BOS or consultant staff. A hydraulic/sizing report shall be prepared by BOS or consultant designers (refer to FDM 13-1-10, and Chapter 8 (hydraulics) of the LRFD Bridge Manual).

In addition, a Structure Survey Report is required for all hydraulic structures designed or reviewed by BOS. Refer to Chapter 6 of the department's LRFD Bridge Manual for report procedures. The region is responsible for the hydraulic adequacy of all other types of drainage structures.

BOS should be notified whenever it is proposed to replace an existing bridge with a pipe culvert(s) so that records of existing bridges may be kept current. Refer to the bridge manual for bridge definition:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/bridge-manual.aspx

The Statewide Drainage Engineer in the Bureau of Project Development Roadway Design Standards Unit shall be notified when plans include the box-shaped storm sewer. The Statewide Drainage Engineer will consult with the Bureau of Structures to determine the design requirements for the storm sewer and whether a structure number will be assigned.

1.5 Common Drainage Law

Drainage Common law is that body of principles found in court decisions based on customs, practices, and precedents that have evolved and are unwritten in statute or code.

According to Harold H. Ellis (1), Wisconsin's common law rules relating to diffused surface waters are as follows:

1. A lower owner may legally treat diffused surface waters as his enemy and prevent them from coming onto his/her land.
2. The upper owner has a right to alter the natural flow of diffused surface waters and may discharge them upon lower land, subject to the following limitations:
   - The water must be expelled onto the lower land without malice.
   - The actions of the upper owner may extend no further than reasonably necessary to protect himself or his/her land.
   - Such water may not be diverted into another watershed.
   - The upper owner may not unduly collect such waters in a pond or reservoir and thereafter discharge them on his/her neighbor's land or on his/her own land in such proximity to his/her neighbor that they will inevitably permeate and percolate so as to permanently injure the neighbor's soil.

Because the upper owner must not be negligent, and he/she must be reasonable in his/her use and improvement of his/her land, Wisconsin has moved to a middle ground, lying somewhere between the "common enemy rule (2)" and the "reasonable use rule (2)".

1.6 Statutory Drainage Law

When the Department of Transportation constructs a highway, the natural or pre-existing flow of surface water might be changed, and the effects of these changes might extend beyond the highway right-of-way to private property. The laws governing these matters are found in Chapter 88 of the Wisconsin Statutes, Drainage of Lands.

Section 88.87 of this chapter states that a highway "*...shall not impede the general flow of surface water or stream water in any unreasonable manner so as to cause either an unnecessary accumulation of waters flooding or water-soaking uplands or an unreasonable accumulation and discharge of surface waters flooding or water soaking lowlands.*" It further states that these highways "*...shall be constructed with adequate ditches, culverts, and other facilities as may be feasible, consonant with sound engineering practices, to the end of maintaining as far as practicable the original flow lines of drainage.*" The section also provides that drainage rights or easements may be purchased or condemned to aid in the prevention of damage to property owners, which might otherwise occur because of the highway construction. (WisDOT does not intend to acquire easements as a routine solution to drainage problems (refer to FDM 13-1-5, Drainage Rights and Easements).

It is the duty of every landowner to provide, and always to maintain, a sufficient drainage system to protect the highway from water damage or flooding, by directing the flow of surface waters into existing highway drainage systems or by permitting the flow of such water away from the highway. Chapter 86, Section 86.07 (2) states that *"no person shall make any excavation or fill or install any culvert or make any other alteration in any highway or in any manner disturb any highway or bridge without a permit therefore from the highway authority maintaining the highway."*

In addition to Chapter 88, Section 86.075 covers the responsibility of a highway authority to notify the county drainage board *"Whenever a highway crossing any draining ditch of a drainage district governed by Chapter 88 is being constructed or reconstructed or a culvert in any such ditch is being replaced, the highway authority in charge of such work shall consult with the drainage board having jurisdiction of such district for the purpose of determining the depth at which such drainage ditch was laid out."* If any culvert or similar opening in a highway is installed at a grade higher than the depth at which a drainage ditch was laid out, the expenses involved in any future lowering of the culvert pursuant to Section 88.68 (4) shall be borne by the unit of government in charge of maintenance of the highway.

The Wisconsin State Statutes, Chapter 146, Miscellaneous Health Provisions also state, in Section 146.13; *"Discharging noxious matter into highway and surface waters (1) If anyone constructs or permits any drain, pipe, sewer or other outlet to discharge into a public highway infectious or noxious matter, the board of health of the village, town or city shall, and the town sanitary district commission or the county board of health, acting alone or jointly with the local board of health may, order the person maintaining it to remove it within 10 days..."* This Section further states (2)*" No person shall discharge by any means whatsoever untreated domestic sewage into any surface water as defined by s. 144.01(5), or drainage ditch governed by ch. 88; nor shall any person discharge effluents or pumpage by any means whatsoever from any septic tank, dry well or cesspool into any surface water as defined by s. 144.01(5), or drainage ditch governed by ch. 88 ..."*

The Wisconsin State Statutes, Chapter 236, Platting Lands and Recording and Vacating Plats, state, in Section 236.13, that *"approval of the preliminary or final plat shall be conditioned upon compliance with: ...(e) The rules of the Department of Transportation relating to provisions for the preservation of the public interest and investment in such highways."* This department rule is TRANS 233 that states as one of its basic principles: one of its basic principles in 233.02 (5) *"A land division map shall include provisions for the handling of surface drainage in such a manner as specified in s TRANS 233.105 (3)."* Section 233.105 (3) states (3) Drainage - The owner of land that directly or indirectly discharges storm water upon a state trunk highway or connecting highway shall submit to the department a drainage analysis and drainage plan that assures to a reasonable degree, appropriate to the circumstances, that the anticipated discharge of storm water upon a state trunk highway or connecting highway following the development of the land is less than or equal to the discharge preceding the development and that the anticipated discharge will not endanger or harm the traveling public, downstream properties or transportation facilities. Various methods of hydrologic and hydraulic analysis consistent with sound engineering judgment and experience and suitably tailored to the extent of the possible drainage problem are acceptable. Land dividers are not required by this subsection to accept legal responsibility for unforeseen acts of nature or forces beyond their control. Nothing in this subsection relieves owners or users of land from their obligations under S.88.87 (3)(b), stats.

Note: In section 88.87 (1), Stats., the Legislature has recognized that development of private land adjacent to highways frequently changes the direction and volume of flow of surface waters. The Legislature found that it is necessary to control and regulate the construction and drainage of all highways in order to protect property owners from damage to lands caused by unreasonable diversion or retention of surface waters caused by a highway and to impose correlative duties upon owners and users of land for the purpose of protecting highways from flooding or water damage. Wisconsin law, section 88.87 (3), Stats., imposes duties on every owner or user of land to provide and maintain a sufficient drainage system to protect downstream and upstream highways. Wisconsin law, section 88.87 (3)(b), Stats., provides that whoever fails or neglects to comply with this duty is liable for all damages to the highway caused by such failure or neglect. The authority in charge of maintenance of the highway may bring an action to recover such damages but must commence the action within 90 days after the alleged damage occurred. Section 893.59, Stats.

The plats should be reviewed to ensure they conform to this principle.

For further details on drainage law, the designer is referred to:

- Wisconsin State Statutes, "Miscellaneous Highway Provisions," Chapter 86.
- Wisconsin State Statutes, "Floodplain Zoning," Chapter 87.
- Wisconsin State Statutes, "Drainage of Lands," Chapter 88.
- Wisconsin State Statutes, "Water, Sewage, Refuse, Mining and Air Pollution," Chapter 144.
- Wisconsin State Statutes, "Miscellaneous Health Provisions," Chapter 146.
- Wisconsin State Statutes, "Platting Lands and Recording and Vacating Plats," Chapter 236.
- Wisconsin Administrative Code, Chapter TRANS 233.

REFERENCES

(1) Ellis, Harold H.; Beuscher, J.H.; Howard, Cletus D.; De Braad, J. Peter; "Water-Use Law and Administration in Wisconsin," Department of Law, University Extension, The University of Wisconsin, First Edition, 1970, 694 pp.

(2) "Guidelines for the Legal Aspects of Highway Drainage," Volume V-Highway Drainage Guidelines, AASHTO, 2007, 24 pp.

LIST OF ATTACHMENTS

Attachment 1.1 Glossary of Terms

FDM 13-1-5 Major Drainage Guidelines and Criteria *March 31, 2017*

5.1 Definition

This procedure defines the major drainage issues and sets guidelines and criteria for more detailed studies, when appropriate. More detailed studies, when required, are completed in the design phase of project development. Three basic questions are asked:

1. Are major drainage problems anticipated?
2. Are the available general drainage guidelines appropriate for solving the anticipated problems?
3. What are the surface drainage alternatives?

These questions should be asked and resolved at the region. The Bureau of Project Development function is to update and clarify the major drainage guidelines, as necessary.

5.2 General Guidelines

To clarify the study of major drainage, it is helpful to consider some typical guidelines. For the most part, these are statutory "guidelines" or traditional practices set up by the Department of Transportation. They are broad practices, the changing of which would have an immediate, statewide effect on adjacent properties. Therefore, they are not subject to random change by either the region or the central office.

The general guidelines are:

1. Water Accumulation: The highway shall not impede the general flow of surface water or stream water in any unreasonable manner so as to cause either an unnecessary accumulation of waters flooding or water-soaking uplands, or an unreasonable accumulation and discharge of surface waters flooding or water-soaking lowlands (from Section 88.87, Wisconsin Statutes). This objective should be accomplished by:
    - Anticipating the amount and frequency of storm runoff.
    - Determining natural points of concentration and discharge and other hydraulic controls.
    - Determining the necessity for protection from floating trash and debris.
    - Comparing and coordinating proposed design with existing drainage structures and systems handling the same flows.
    - Removing detrimental amounts of surface and subsurface water.
    - Providing the most efficient disposal system consistent with economy, the importance of the road, maintenance, and legal obligations.
    - Culverts designed with the intent to permanently impound water may be regulated by WDNR as dams. In general, this situation should be avoided because of the potential regulatory issues and the potential barrier to aquatic organism passage. The Statewide Drainage Engineer in Bureau of Project Development should be notified of any culvert designed to permanently impound water.
2. Drainage Districts: Any work that involves drainage districts must be coordinated with the county drainage board of such district. The legal procedures for these cases are set forth in Chapter 86 of the Wisconsin Statutes and ATCP 48 of Wisconsin Administrative Code (refer to FDM 5-15-1).
3. WisDOT and Wisconsin Department of Natural Resources (WDNR) Cooperative Agreement: The Department of Transportation shall design and construct drainage facilities in accordance with the spirit and intent of the WisDOT and WDNR Cooperative Agreement, a copy of which can be found at:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/environment/formsandtools.aspx

4. 401 and 404 Permits: The necessity of 401 and 404 permits for a drainage facility should be determined by FDM 20-50.
5. Local Sewerage Commissions: Coordinate work with local sewerage commissions that are affected by the project.
6. Aquatic Organism Passage: The crossing of some streams by highways requires the construction of drainage facilities that will accommodate aquatic organism passage. In addition, other streams may require the construction of barriers at drainage structures to prevent the migration of rough fish or other invasive species. In the early stages of design, the WDNR shall be consulted when streams are involved that might require special drainage facilities. Moreover, if aquatic organism passage facilities are required at a drainage structure, a field review regarding questions on aquatic organism passage should be held with the WDNR. For culvert design including aquatic organism passage, the designer should consult with the Statewide Drainage Engineer responsible for AOP coordination.
7. Drainage Patterns: Highway reconstruction projects should match natural drainage patterns as closely as possible. New culverts should be located and designed to minimize change or disruption in the natural flow of water, commensurate with cost.

    When the highway is in fill, the amount of special ditching along the fill slope should be minimized except where required to protect the adjacent land.

    When a highway is constructed on relocation, changes in surface drainage are more significant. Culverts should be placed at natural draws or depressions. Culverts should be placed frequently enough to avoid excessive concentration of flow.
8. Headwater: Criteria for culvert headwater is generally set as 1.5 times the pipe diameter, or no overtopping of roadway for design storm event, provided there is no risk of damage to adjacent

upstream property. Headwater elevation shall have no rise in mapped zoned floodplains unless all requirements of the WisDOT/WDNR Cooperative Agreement are met.

9. Drainage Rights and Easements: Wisconsin Statutes provide WisDOT with the authority to acquire drainage rights and easements. However, WisDOT does not intend to acquire easements unless it is determined that significant damage would occur to private property or that the cost of a larger structure (designed to accommodate the regional flood) would not be justified.

10. Overflow Section: These sections are considered for special situations (refer to FDM 13-10-1).

11. Maintenance Considerations: For future maintenance considerations the designer should provide:
    - Sufficient erosion protection for channel banks, for the highway, and for culvert outlets to prevent scour or erosion on private land.
    - Large enough culverts for ease of maintenance.
    - Curbs or berms and downslope pipe or gutters along fills of erodible material.
    - Drainage easements wide enough for maintenance equipment.
    - Access at drainage facilities for power equipment.
    - Debris catches where needed.
    - Corrosion-resistant structures in areas with corrosive soils and waters.
    - Interceptor ditches along the top of cut slopes.
    - Necessary drainage structures should be located, if possible, beyond the clear zone (refer to FDM 11-15-1). Where this is not possible, suitable protective barriers should be provided.

5.3 Surface Data Collection

5.3.1 Probable Working Media for Major Drainage Studies

The designer will usually be working from an aerial mosaic with a scale of 1" = 200' to 1" = 800' (1:2400 to 1:9600). LiDAR elevation data and 2-foot contours are available for many counties through data-sharing agreements. Countywide digital orthophotos may be obtained through the USDA NAIP at:

www.wisconsinview.org

Soils data can be found in NRCS Soil Survey Maps and digitally from the NRCS Geospatial Data Gateway.

5.3.2 Input Data Requirements

Desirable data will be of a general nature, as follows:

1. Watershed characteristics
2. Stream crossing locations
3. Climate information
4. Limiting design factors
5. Information on existing structures that would be readily available from logs, etc.
6. General information from local sources as to history of flooding and obvious problem areas.
7. Land use/cover
8. Soils data

5.3.3 Output Data

Desirable output data requirements are as follows:

1. Design discharge.
2. Proposed facilities.
3. Drainage easements.
4. Cost.

LIST OF ATTACHMENTS

Attachment 5.1	Drainage Data Requirements, Design Aids and Computer Software
Attachment 5.2	Major Drainage Summary Sheet

FDM 13-1-10 Documentation of Hydrologic/Hydraulic Design November 30, 2018

10.1 Introduction

Documentation of all hydrologic data and hydraulic design computations shall be assembled for each project and retained in the project files at the Region office. This documentation should contain all pertinent information used to design the drainage facilities and should be extensive enough to verify later the hydraulic design of any structure. It should also include any information about special commitments placed on the project for environmental or public involvement reasons.

The Stormwater-Drainage-Water Quality (WQ) Report Spreadsheets along with the Channel and Chute Design Spreadsheet Worksheets are required.

Hydrology and hydraulic design documentation is to be stored in the Region Central file system for25 years (Refer to RDA 00145-000, Roadway Drainage Hydrologic & Hydraulic Studies and Design Calculation). The documentation must be provided to the project manager, who will send it to the Region Central file system.

10.2 Bridge and Box Culvert Design

The hydraulic design documentation for a bridge, box culvert or other large drainage conduit requiring structural analysis shall contain a segment entitled "Discussion of Structure Sizing." This discussion should concisely summarize the engineering judgments that determined the structure size (waterway opening). Relevant factors to be highlighted include: relative construction cost considerations, environmental concerns, compatibility with local floodplain zoning ordinances, and risk considerations such as minimization of flooding, potential damages to abutting property, and protection of the motorist and/or highway.

A stream crossing Structure Survey Report (SSR) should be prepared by the Region and e-submitted to the Bureau of Structures (BOS) if the structure is to be designed by BOS. If the structure is to be designed by a consultant, the SSR should be e-submitted to BOS along with the preliminary structure plans and the hydraulic/sizing report referred to in the preceding paragraph. See Chapter 8, Appendix A, of the WisDOT Bridge Manual (https://wisconsindot.gov/dtsdManuals/strct/manuals/bridge/ch8.pdf) for a checklist of items that need to be included in the hydraulic/sizing report.

Typically, BOS will perform the hydraulic/hydrologic analysis for all bridges and box culverts that are designed by the Department. Consultants are responsible for the hydraulic/hydrologic analysis of the bridges and box culverts they design.

10.3 Stormwater Report Applicability

Each WisDOT project that has a stormwater component must have a completed Stormwater-Drainage-Water Quality (WQ) Report spreadsheet. A Stormwater-Drainage-WQ Report is not needed for projects that have no change to the culvert or storm sewer system that drains the project or for projects that do not trigger TRANS401 water quality requirements. Typically, traffic control, ITS (Intelligent Transportation Systems), signalization, or safety projects will not need a stormwater report. Overlay projects that do not include culvert replacements, extensions, or other modifications are also exempt from the stormwater reporting requirement.

The Drainage Summary Worksheet should be submitted at the 30% design stage to describe any significant flow changes and what may need to be done to address the changes in flow. The intent of this early submittal is to note potential drainage problems at an early stage. The updated Summary Worksheet and the initial Data Worksheet should be submitted at the 60% design stage.

This submittal should address the concerns brought up in the previous Drainage Summary Worksheet, any new issues, and include available information in the Data Worksheet. This submittal should also include any available drainage calculations or model analyses and drainage mapping. The final design submittal includes the completed Drainage Summary and Data Worksheets as well as all supporting documentation needed to review the worksheets.

The stormwater report spreadsheet is not applicable to bridges and box culverts designed or reviewed by Bureau of Structures nor is it applicable to storm sewer design.

10.4 Design Documentation

Each WisDOT project that has a stormwater component must develop a design for those components that includes the basin hydrology and the structure or system hydraulic design. The type and extent of the documentation for these components will vary, but the basic information includes the hydrology and hydraulic design information listed below. A summary of this information should be included in the Drainage-Stormwater Report spreadsheet described below. This spreadsheet provides a way for a designer to methodically describe the objectives and design of a project drainage system.

Hydrology

1. Design frequencies.
2. Methods used to compute the flow rates and the limitations of these methods.
3. The type and extent of future development and how it was considered in the design process.
4. List of all graphs that were used to determine rainfall depth, rainfall intensity, runoff, and time of concentration.
5. Any information that is used by the designer as general criteria for the determination of flow rates for ditches and culverts.
6. Location map indicating each drainage area. Show the drainage areas in the form of a mosaic on a 1 inch = 100-foot, (1:1200) scale, photogrammetric, contour map. Large drainage areas should be shown on USGS contour maps.
7. A statement of the characteristics of the drainage pattern about soil types, land usage and relief, special controls on the flow rates, possible future development effects, past flood of record, and any information that is needed to properly analyze the flow rate for the given drainage area and detailed computations of the flow rate.

Hydraulic Design

1. Detailed hydraulic design for each culvert location and each channel and ditch on the project.
2. A statement on any information gathered during the field review of the drainage area.
3. For culverts, provide the design work sheet or the computer design sheet for the culvert, which should contain information on the discharge, allowable headwater elevation, design headwater elevation, design tail-water depth, entrance conditions, grade of flow line, discharge velocity, freeboard (allowable headwater-roadway elevation), etc. This sheet should show designs for various types of inlet conditions and culvert materials, along with the final recommendation of the culvert used in the final design.
4. For storm sewer systems, include the urbanization factors, cost analysis, and any other factors that may affect the final design of the storm sewer. Provide a layout of the storm sewer system along with the contributing drainage areas, and a detailed design tabulation sheet showing the grate inlets, flow rates contributing to each inlet, and pipe sizes.

This documentation, initiated during the preliminary design stage, must be updated to reflect the final design. The stage of the design can be noted in the Drainage-Stormwater Report submittal described below on the Drainage-Summary worksheet.

As part of design documentation, the designer should determine whether a project is located within a regulatory floodplain. Unofficial floodplain maps can be viewed on WDNR's Surface Water Data Viewer:

https://dnr.wi.gov/topic/surfacewater/swdv/

Official Flood Insurance Rate Maps can be viewed and printed in "FIRMette" form at FEMA's Map Service Center under the Product Catalog at:

https://msc.fema.gov/

10.5 Stormwater-Drainage-WQ Report Spreadsheet Instructions for Drainage Design

There are two components to the spreadsheet: drainage and water quality. This section describes how to fill out the stormwater drainage worksheets of the report. Refer to FDM 10-30-1 for instructions on how to fill out the water quality worksheet sections of the Report.

The stormwater drainage section of the spreadsheet has two parts. The first part, which is on the 'Drainage-Summary' worksheet tab, is the Summary worksheet. This worksheet includes basic project information, (project name, limits, county, etc.) and a list of questions that will help the designer determine the drainage requirements for the project.

The second part of the stormwater drainage is a table of the stormwater flow and drainage issues that typically occur in a project. This list is in the 'Drainage-Data' worksheet tab and includes the following topic areas:

1. Outfall Information
2. Basic Subbasin Drainage Information

3. Urban/ and/or TRANS 401 Project Information (see FDM Chapter 10 for TRANS 401 requirements)
4. Culvert Design
    a. Existing Culvert Data
    b. Proposed Culvert Design
    c. Floodplain Management
    d. Drainage District Issues
    e. Aquatic Organism Passage
5. Culvert Liner Design
    a. Existing Culvert Data
    b. Liner Details
    c. Floodplain Management
    d. Drainage District Issues
    e. Aquatic Organism Passage

The spreadsheet includes an outline feature that allows the user to collapse topic groups that are not relevant for the project to make the worksheet easier to use.

There are ten worksheets in the Stormwater-Drainage-WQ Report spreadsheet. The Stormwater Water Quality Summary worksheet and the water quality control practice worksheets, which all begin with the letters 'WQ', are discussed in FDM 10-30-1. The Drainage-Summary worksheet and the Drainage-Data worksheet are described below.

10.5.1 Drainage Summary Worksheet

This worksheet includes basic project information and a Drainage Summary page that includes questions that address drainage issues (refer to Attachment 10.1 and 10.2). Water quality questions and issues are addressed in FDM 10-30-1. Be sure to enable the spreadsheet Macros by clicking on the security warning "options" box on the top of the spreadsheet and then highlight the "enable this content" button.

10.5.1.1 Basic Project Information

Basic project information includes information like the project number and name. When entering this information, only enter it in columns B and C of this worksheet; the appropriate information will be copied to other worksheets by the spreadsheet.

Please note that the planning stage generally includes only the water quality component of stormwater management unless drainage considerations are part of a planning study.

10.5.1.2 Drainage - Summary Narrative

The drainage summary narrative begins with line 15 on the "Drainage-Summary" tab of the stormwater report spreadsheets. This narrative is a series of questions that will, when completed, define the drainage goals, objectives, and issues for the project and how they were met. Enter your response in the cell below each question.

Line 15: IS THERE A SIGNIFICANT FLOW INCREASE OR DECREASE WITHIN ANY SUB BASIN? IF YES, DESCRIBE THE REASON.

This question is intended to describe why any significant (greater than 5%) flow increases or decreases occur in the project. Examples of an explanation and justification could include "Outfall 3: New connection to municipal storm sewer system" or "Outfall 8: Outfall location shifted and combined with adjacent upstation drainage basin to avoid concentrated discharge to wetland.

Line 17: IS THERE A SIGNIFICANT IMPERVIOUS AREA CHANGE TO ANY SUB BASIN? IF YES, DESCRIBE THE REASON.

Increases in impervious surface area are often the result of added lanes, new alignment, or park and ride lots, etc. However, the impact on peak rate discharge may be insignificant if the impervious area is a small portion of the subbasin or if the impervious area is located near the outfall. Increased impervious surface will increase the runoff volume. The impact of the increased impervious area may be significant if the overall drainage basin is small or if the added discharge from the impervious area reaches the outfall at the same time as the peak flow from the balance of the drainage bas

Line 19: HAVE THE DRAINAGE SUB BASIN AREAS OR FLOW PATHS CHANGED SIGNIFICANTLY? IF

YES, DESCRIBE THE REASON.

Altered flow paths may change the size of the drainage basin and affect the downstream drainage system. Existing ponds and wetlands may be affected if tributary drainage is relocated. Peak discharge rate increase may increase the potential for streambed erosion. Document the reason for the drainage area re-routing and describe erosion control plans to address increased peak discharge rates.

Line 21: DESCRIBE THE PROPOSED DRAINAGE CONVEYANCE AND CONTROL SYSTEMS.

The conveyance system may include any combination of drainage swales, culverts and/or storm sewers. Control systems may include detention ponds, diversion structures, etc.

Line 23: DESCRIBE ANY AQUATIC ORGANISM PASSAGE ISSUES.

If one or more culverts in the project require aquatic organism passage design, describe the water body classification, the requesting agency, and reason for request. Complete the AOP section of the Stormwater Report Drainage-Data section for the culvert(s).

Line 25: DESCRIBE ANY EXCEPTIONS TO WisDOT FDM CHAPTER 13 DRAINAGE REQUIREMENTS,

Document and explain any exceptions to the FDM Chapter 13 drainage design requirements. Examples may include the use of 12-inch diameter storm sewer pipes or wide-bottom special ditches.

Line 27: DESCRIBE WDNR COORDINATION.

Provide name of WDNR liaison, date of correspondence, and attach printed copy of correspondence.

Line 29: DESCRIBE ACCOMMODATIONS TO MEET LOCAL, MUNICIPAL, OR REGIONAL DRAINAGE OR STORMWATER MANAGEMENT THAT EXCEED FDM CHAPTER 13 REQUIREMENTS.

Sometimes accommodations are made to meet drainage design standards that exceed WisDOT FDM Chapter 13 design requirements. For example, a community may want a detention pond to decrease peak flows in the off-DOT ROW drainage area or may want all drainage structures in their jurisdiction to meet their higher design standards, so the entire drainage system meets a consistent set of standards. If this occurs, document the accommodation, why it was made, and the source of funding for the modifications.

Line 31: DOCUMENT SIGNIFICANT IMPACTS TO THE PROJECT CAUSED BY DRAINAGE, PROJECT MANAGER CONCURRENCE IS REQUIRED. (PM SIGN AND DATE).

The project manager must acknowledge any significant drainage impacts or non-standard design changes to the project by signing this report or providing documented concurrence using, for example, an email message stating he or she has reviewed and approved of the report.

10.5.1.3 Drainage - Data Worksheet

The number of columns in the worksheet can increase as needed by highlighting the last unfilled column and dragging the small box in the lower right-hand corner of the highlighted column to the right. The outfall numbers will increase consecutively.

As noted in FDM 13-1-10.4, there are several headings in the Drainage-Data spreadsheet. This section will review the contents of each heading.

If an explanation is required as part of the response to a line item in the report, provide the explanation on a separate attachment.

Section 1: Outfall Information:

Lines 8 – 28 should be completed for all outfalls, regardless of whether culverts or storm sewer system. Documentation for storm sewer systems should be attached to the Stormwater Report, but no information past Line 28 is necessary in the Stormwater Report.

Line 8: Outfall number.

Consecutively numbered outfalls from the start to the end of the project. An outfall is any culvert, roadside ditch, or storm sewer drainage discharge point with runoff either originating from or passing through the project right-of-way.

Line 9: Outfall discharges to:

Use the pull-down menu to select the type of water body the outfall discharges to. The options are: 1) Overland, 2) Ditch, 3) Creek, 4) River, 5) Wetland, 6) Storm Sewer, 7) Combined Sewer, 8) Other.

Line 10: Waterway crossing type:

Use the pull-down menu to select the type of waterway crossing. The options are: 1) Culvert, 2) Box Culvert, 3) Storm Sewer, 4) Three-Sided Box Culvert, and 5) Bridge.

Line 11: If discharging to environmentally sensitive area, what kinds of buffers were used at outfall?

The options in the pull-down menu are: 1) Swales, 2) Filter Strips, 3) Vegetated Embankment.

Line 12: Previous flooding issues or flow restrictions?

Select yes or no from the pull-down menu. If yes, provide explanation.

Line 13: Is the drainageway in the DOT ROW a navigable waterway?

Select yes or no from the pull-down menu.

Line 14: Classify the drainageway in the DOT ROW.

The options are: 1) Wetland, 2) 303(d) Waters, 3) Exceptional Waters, 4) Outstanding Waters, 5) Waters of the U.S.

Section 2: Basic Sub Basin Drainage Information:

Line 16: Outfall number.

Consecutively numbered outfalls from the start to the end of the project. An outfall is any culvert, roadside ditch, or storm sewer drainage discharge point with runoff either originating from or passing through the project right-of-way.

Line 17: Outfall station.

The station along the reference line where the outfall is located.

Line 18: Design storm frequency.

Enter the flood frequency used for the design of the culvert or storm sewer system. This value is typically found in FDM 13-10.

Line 19: Check storm frequency.

Enter the flood frequency used to check the design for unacceptable inundation of the highway facility or flooding. Refer to FDM 13-25-20 for additional information.

Line 20: Drainage area (ac).

Line 21: Hydrologic Method.

List the method used to compute the peak discharge rates for the design and check storms. Examples include, but are not limited to: The Rational Method, TR20/55, HEC-1/HMS, regional regression equations, and basin transfer methodology.

Line 22: Time of Concentration (min).

The time required from discharge to travel from the most hydrologically remote point in the drainage area to the outfall.

Line 23: C or CN.

Runoff coefficient, C, for use with the Rational Method can be found in FDM 13-10 Attachment 5.2, and Runoff Curve Numbers, CN, for use with TR20/55 can be found in FDM 13-10 Attachment 5.6.

Line 24: Rainfall Intensity (in/hr).

Rainfall intensity is used with the Rational Method for hydrologic computations and can be found using the Intensity-Duration-Frequency (IDF) curves in FDM 13-10 Attachment 5.4.

Line 25: Rainfall Depth (in)

For hydrologic methods using a design storm to determine the peak discharge rate, record the rainfall depth of the design storm. Unless a specific rainfall distribution is required by others (WDNR or SEWRPC), use NOAA Atlas 14 rainfall data with MSE-3 and MSE-4 24-hour rainfall distributions. These rainfall distributions are available from NRCS for use with the NOAA Atlas 14 rainfall data at:

https://www.nrcs.usda.gov/wps/portal/nrcs/detail/wi/technical/engineering/?cid=nrcs142p2_025417

Where the use of WDNR or SEWRPC distribution and a critical storm duration analysis is required, use NOAA Atlas 14 data as well. SCS Type II rainfall distribution should no longer be used.

Line 28: Hydraulics design software used.

Record the design software used for drainage analysis and design.

Section 3: Urban/TRANS 401 Project Information:

This section, including lines 29-43 of the stormwater/drainage report form, is required only for urban projects or projects with TRANS 401 water quality requirements.

Line 31: DOT right of way area (acres).

Enter the area draining to the outfall that is within the WisDOT right-of-way.

Line 32: DOT right-of-way compared to sub-basin drainage area (%) (calculated).

This value is self-populated based on data in Lines 20 and 31. The relative drainage area information may be used when negotiating storm sewer cost share agreements between WisDOT and a municipality.

Line 33: DOT impervious area – existing (acres).

Enter the existing (pre-project) impervious surface area within the WisDOT right-of-way to outfall.

Line 34: DOT impervious area – proposed (acres).

Enter the proposed impervious surface area within the WisDOT right-of-way to outfall.

Line 35: Change in impervious area (calculated).

This value is self-populated based on data in Lines 33 and 34. This information may be used to determine possible reason for change in discharge and potential downstream impacts.

Line 36: Percent change DOT in impervious area (calculated).

This value is self-populated based on data in Lines 33 and 34. This information may be used to determine possible reason for change in discharge and potential downstream impacts.

Line 37: Proposed peak discharge rate (cfs), before detention

Design peak flow for the proposed drainage system, not including impacts of detention.

Line 38: Peak discharge rate change (cfs).

This value is self-populated based on data in Lines 26 and 37.

Line 39: Percent change peak discharge rate (%).

This value is self-populated based on data in Lines 26 and 37.

Line 40: Design peak discharge rate (cfs) post-detention:

Enter the design peak discharge rate for the proposed drainage system post-detention. If there is no detention, then this value is the same as the value in Line 37.

Line 41: Existing 2-year peak discharge flow (cfs):

The pre-project 2-year peak discharge rate for the drainage system.

Line 42: Proposed 2-year peak discharge flow (cfs)

The post-project, pre-detention 2-year peak discharge rate for the proposed drainage system.

Line 43: Proposed 2-year peak flow (cfs), (after detention/in-line storage/other).

The design peak discharge rate for the proposed drainage system after any detention/ in line/other storage. If there is no detention, then this value is the same as the value in Line 42.

Section 4: Culvert Replacement/Extension Project Information

Culvert Design – Existing Culvert

Line 52: Manning's n:

Roughness values for common culvert materials can be found in Table B.1 of FHWA HDS-5.

Line 53: Inlet configuration:

Typical inlet configurations can be found in FHWA HDS-5, Chapter 1, Section 3.3. Choices in the drop-down menu include: apron endwalls, mitered to slope, headwall, projecting.

Line 54: Upstream invert (ft)

Elevation of bottom of culvert at upstream end.

Line 55: Downstream invert (ft).

Elevation of bottom of culvert at downstream end.

Line 56: Length (ft).

Culvert length, not including apron endwalls or headwalls.

Line 57: Slope (%)

Value automatically calculated by dividing invert elevation difference by pipe length.

Line 58: Computed upstream water surface elevation (ft).

Upstream water surface elevation computed for design peak discharge using hydraulic computation program in steady state analysis mode or FHWA HDS-5 nomograph methodology.

Line 59: Tailwater elevation (ft).

Water surface elevation at pipe outlet, based on normal depth of downstream channel or average of critical depth and culvert diameter. See FHWA HDS-5 for more detail.

Line 60: Outlet velocity (ft/s)

Water velocity at outlet end of pipe.

Culvert Design – Proposed Culvert

Line 62 to 82: Proposed culvert information.

Lines 63-72 contain similar inventory of physical properties as lines 48-57 for the existing culvert.

Line 76: Change in upstream water surface elevation.

Value automatically computed comparing Line 77 and Line 58. Note that increases in upstream water surface elevation are generally discouraged and may be prohibited without “appropriate legal arrangement” in mapped floodplains.

Line 77: Riprap outfall.

Size of riprap at culvert outfall, if necessary.

Line 78: Maximum allowable headwater (ft)

The maximum upstream water surface elevation.

Line 79: Maximum allowable headwater design criteria.

Drop down menu includes options of: existing conditions, shoulder subgrade point, or edge of pavement elevation, or headwater to culvert diameter ratio.

Line 80: Station of lowest subgrade shoulder point.

The sag point of the vertical curve over the proposed culvert.

Line 81: Elevation of lowest subgrade shoulder point (ft).

Top of subgrade at sag point of vertical curve.

Line 82: Headwater to pipe diameter ratio.

Value is automatically calculated based on depth at upstream end (highwater elevation minus invert elevation) divided by the culvert diameter or height.

Culvert Design - Floodplain Management

Line 84: Mapped floodplain.

To determine if the culvert is in a mapped floodplain, either check with the region stormwater engineer or view unofficial maps on the Wisconsin DNR Surface Water Data Viewer - FEMA Maps/DFIRMS at:

https://dnrmaps.wi.gov/H5/?Viewer=SWDV

Official Flood Insurance Rate Maps (FIRMs) can be found on FEMA’s website at:

https://msc.fema.gov/portal/home

Line 85: Increase in headwater.

If there is an increase in water surface elevation, attach an explanation for the change and how the increased water surface profile was approved.

Culvert Design – Drainage District Issues

Line 87: Is culvert in a drainage district?

To determine if the culvert is in a drainage district, check with the region stormwater engineer or go to the web site:

https://datcpgis.wi.gov/maps/?viewer=dd

Line 89: Increase in headwater

If there is an increase in water surface elevation, attach an explanation for the change and how the increased water surface profile was approved.

Line 90: Drainage board approval?

Drainage board approval is required for increases in water surface profiles in areas located within incorporated drainage districts.

Culvert Design – Aquatic Organism Passage

Line 92: Is aquatic organism passage (AOP) a concern?

If AOP is considered in project, please include a copy of the WDNR Initial Review Letter.

Line 93: Does WDNR concur with the AOP design?

Provide documentation of WDNR concurrence.

Line 94: Embedment Depth

The depth of the inverts below the natural stream channel.

Line 95: Embedment Material

Description of the gradation of material in culvert. The material should match the native streambed material to the extent possible.

Section 5: Culvert Liner Design

Lines 99 – 133 should be completed for any project that includes a culvert liner.

Culvert Liner Design - Existing Culvert

Line 99: Outfall number.

Consecutively numbered outfalls from the start to the end of the project. This value is auto-populated.

Line 105: Manning's roughness

Use standard values for "n" (i.e. 0.013 for concrete, 0.024 for corrugated metal)

Line 106: Pipe geometry

Cross sectional shape of pipe (i.e. circular, elliptical, arch, etc.)

Line 107: Upstream invert

Elevation of the upstream end of the pipe.

Line 108: Downstream invert

Elevation of the downstream end of the pipe.

Line 109: Length (ft)

Length of pipe, not including endwalls or aprons

Line 110: Slope (%)

Automatically populated based on invert elevations and pipe length.

Line 111: Depth of cover over pipe (ft)

Minimum depth at between roadway surface and top of pipe

Line 112: Is overtopping an issue?

Based on observed erosion or reports of local residents, document any past observed overtopping.

Line 113: Upstream flooding risk?

Note and buildings or infrastructure that may be at risk if upstream water surface elevations are increased as a result of lining the culvert.

Culvert Liner Design - Floodplain Management

Line 125: Is the culvert in a mapped floodplain?

Select the pull-down menu to answer either 'Yes' or 'No'. To determine if the culvert is in a mapped floodplain, either check with the region stormwater engineer or go to the Wisconsin DNR Surface Water Data Viewer – FEMA Maps/DFIRMS at:

https://dnrmaps.wi.gov/H5/?Viewer=SWDV

Official Flood Insurance Rate Maps (FIRMs) can be found on FEMA's website at:

https://msc.fema.gov/portal/home

Line 126: Will proposed liner increase water surface profile?

Select the pull-down menu to answer either 'Yes' or 'No'. If the answer is yes, attach an explanation of the reason for the change and how the increased water surface profile was approved.

Culvert Liner Design – Drainage District Issues

Line 128: Is culvert in a drainage district?

Select the pull-down menu to answer either 'Yes' or 'No'. To determine if the culvert is in a drainage district, either check with the region stormwater engineer or go to the web site:

https://datcpgis.wi.gov/maps/?viewer=dd

Line 129: Drainage District name.

Enter the name of the drainage district.

Line 130: Has the drainage board approved the use of a liner?

Select the pull-down menu to answer either 'Yes' or 'No'. If the answer is no, attach an explanation of the reason why the drainage board did not approve the change.

Culvert Liner Design - Aquatic Organism Passage

Line 132: Is aquatic organism passage a concern?

Select the pull-down menu to answer either 'Yes' or 'No'. If the answer is 'Yes', respond to the question on Line 135.

Line 143: Does WDNR agree with the AOP design?

Select the pull-down menu to answer either 'Yes' or 'No'. If the answer is no, attach an explanation of the reason for why the WDNR did not approve the design.

10.5.2 Example Stormwater-Drainage-WQ Worksheet

A completed Stormwater - Drainage Worksheet example is provided within the downloadable zip Stormwater-Drainage files (refer to link at top of Attachment 10.1 and 10.2). It includes both an urban and a rural component. The first sheet is the drainage basin overview figure, which illustrates basins for the entire project. If the corridor is long, additional sheets may be appropriate. For this example, not all drainage areas and outfalls for the project are shown. The map sheets provide additional detail, at a closer scale, of the drainage system along the highway for selected basins. The map sheets are followed by a completed stormwater report that illustrates both grass swale and storm sewer drainage.

LIST OF ATTACHMENTS

FDM 13-1-15 Culvert Material Selection Standard — *February 18, 2020*

15.1 Application

This procedure establishes criteria for selecting the proper combination of culvert material and coating for different situations.

WisDOT has approved steel, aluminum, concrete and thermoplastic as suitable materials for culvert pipe. Coating systems for steel culvert pipe may be either zinc-coated (galvanized), aluminum or polymer. The standards in this procedure apply to all shapes of culvert pipe (circular, arch or elliptical) and to pipes in the range of 12 to 84 inches in diameter. The selection of larger drainage conduit is addressed in FDM13-1-20. Culvert replacement and analysis for perpetuation and rehabilitation projects is further discussed in FDM 13-1-30.

These standards are based on the expected service life of the material, the traffic volume to be supported, and the location of the pipe. Service life depends primarily on how durable the material is when subjected to corrosive or abrasive site conditions. Service life also depends on the proper structural design and installation of the pipe. These factors are considered in the Fill Height Tables of FDM13-1-25 as well as the standard specifications and the appropriate special provisions for individual projects.

The following table defines abbreviations commonly used throughout this chapter.

Table 15.1 WisDOT Standard Abbreviations for Pipe Materials

Material	Abbreviation
Corrugated Steel	CPCS
Corrugated Aluminum	CPCA
Corrugated Polyethylene	CPCPE
Corrugated Polypropylene	CPCPP
Reinforced Concrete	CPRC

FDM 15-1-35 contains examples of the correct notations for specifying culvert pipe on a plan and profile sheet.

FDM 15-1-30 shows how to indicate culvert types on the Miscellaneous Quantities Sheet.

15.2 Selection Standard

Selection of pipe materials is to be based on Table 15.2 with consideration given to traffic volume and fill height in addition to the special situations and site conditions as described in FDM13-1-15.3 to FDM 13-1-15.6.

As conditions allow, and with the exceptions listed, Culvert Pipe Class III-A, Culvert Pipe Class III-A Non-metal, Culvert Pipe Class III-B, and Culvert Pipe Class III-B Non-metal under Standard Spec 520 shall be specified for culverts where ADT is less than or equal to 20,000 and where the diameter is 36-Inches or less.

These Class III-A and Class III-B bid items allow the contractor to choose from steel, concrete, and thermoplastic pipe (corrugated polyethylene and corrugated polypropylene) for sizes up to 36 inches in diameter. As described in FDM 13-1-17.3.1, the intent of these Class III-A and Class III-B items is to introduce potential project cost reductions into the competitive bid process by allowing the contractor to select from multiple material options for pipes sized up to 36 inches.

The four subclasses of Class III culverts are as follows:

- Class III-A
  - includes Class II and III reinforced concrete, corrugated steel, corrugated polyethylene, and corrugated polypropylene.
  - Class III-A has a maximum fill height of 11 ft.
- Class III-B
  - includes Class III reinforced concrete, corrugated steel and corrugated polypropylene
  - Class III-B has a maximum fill height of 15 ft.
- Classes III-A, Non-metal and Classes III–B, Non-metal
  - these non-metallic subclasses are for corrosive environments where it is not advisable to use metal pipe.
  - therefore, corrugated steel is removed.

The four subclasses of Class III-A and Class III-B culverts, steel culverts and thermoplastic culverts are not allowed under Interstate Highways or divided US Highways unless for temporary use or at maintenance crossovers in the median. When any of these materials is used on Interstate Highways or Divided US Highways for temporary use or at maintenance crossovers in the median, it is at the designer's discretion and there is no ADT restriction. There are three additional exceptions to the prohibition on thermoplastic and steel pipe on the Interstate and divided US Highways. The exceptions are the use of thermoplastic materials for inlets serving bridge deck drainage (SDD 8D3), PVC pipe used for slotted vane drains (SDD 8D14), and steel pipe used for slotted corrugated metal pipe surface drains (SDD 8D13). These types of installations take place outside of the traveled way limits or are encased in concrete.

For culverts greater than 36 inches in diameter and where ADT is less than or equal to 20,000, or where special situations, fill height or site conditions preclude the use of the Class III A and Class III B bid items, the designer may select another material type from Table 15.2 for the culvert. In situations where concrete or steel pipe is appropriate for a site, consider the use of the Culvert Pipe Class III items (520.3100-3199) under Standard Spec 520. These items also introduce potential project cost reductions into the competitive bid process by allowing the contractor to select from steel or concrete pipe.

Reinforced concrete pipe is required for culverts under high volume roadways (ADT >20,000) except as provided above for bridge deck drainage, slotted vain drains and slotted corrugated metal pipe surface drains, and special situations.

Table 15.2 on the next page lists the preferred materials permitted for culvert pipe by traffic volume. Material options for culvert replacement on perpetuation and rehabilitation projects is further discussed in FDM 13-1-30.4.

***Table 15.2** Culvert Material Selection Criteria*

All Roadways with Design Year ADT ≤ 20,000 Excluding Interstate Highways and Divided US Highways			
BID ITEM (Culvert Pipe)	**DESIGN ADT**	**ALLOWABLE SIZES (Inches)**	**NOTES**
Class III-A, Class III-A Non-metal	Up to 20,000	12 – 36	- Max fill height of 11 ft. - Min. fill height 2 ft. from top of subgrade. - For Culvert Pipe Class III-A indicate required thickness for steel culverts in Misc. Qualities. - Use non-metal bid items in corrosive environments.
Class III-B, Class III-B Non-metal	Up to 20,000	12 – 36	- Max fill height of 15 ft. - Min. fill height 2 ft. from top of subgrade. - For Culvert Pipe Class III-B indicate required thickness for steel culverts in Misc. Quantities. - Use non-metal bid items in corrosive environments.
Corrugated Steel	Up to 20,000	42 – 84	- Not to be used in corrosive environments unless polymer or aluminum coated. See FDM 13-1-15.4. - 12- 36-inch sizes can only be used in special situations. See FDM 13-1-15.3. - Refer to FDM 13-1 Attachment 25.2 and 25.3, for appropriate fill heights. - Indicate required thickness in Misc. Quantities.
Reinforced Concrete	Up to 20,000	12 – 36(1) 42 – 84	- Consider for use in corrosive environments. - (1) 12- 36-inch sizes can be considered in special situations. See FDM 13-1-15.3. - Refer to FDM 13-1 Attachment 25.1 and 25.2 for appropriate fill heights.
Polyethylene	Up to 20,000	12 – 36	- Max fill height of 11 ft. - Min. fill height 2 ft. from top of subgrade. - Consider for use in special situations. See FDM 13-1-15.3 and FDM 13-1-15.4.1
Polypropylene	Up to 20,000	12 – 36	- Max fill height of 15 ft. - Min. fill height 2 ft. from top of subgrade. - Consider for use in special situations. See FDM 13-1-15.3 and FDM 13-1-15.4.1
Corrugated Aluminum	Under 1,500	42 – 84	- Consider for use in corrosive environments. - 12- 36-inch sizes can only be used in special situations. See FDM 13-1-15.3. - Refer to FDM 13-1 Attachment 25.2 and 25.6 for appropriate fill heights. - Indicate required thickness in Misc. Quantities.
Interstate Highways, Divided US Highways or Any Class of Roadway with Design Year ADT > 20,000,			
BID ITEM (Culvert Pipe)	**DESIGN ADT**	**ALLOWABLE SIZES (Inches)**	**NOTES**
Reinforced Concrete	> 20,000	12 – 84	- Refer to FDM 13-1 Attachment 25.1 and 25.2 for appropriate fill heights.

Note: Steel and thermoplastic culverts are allowed under any roadway type at any ADT when used for temporary use, or at maintenance crossovers in the median. In addition, thermoplastic pipe is allowed when used for bridge deck drainage and slotted vain drains, and steel pipe is allowed for slotted corrugated metal pipe surface drains.

15.2.1 Local Approval of Culvert Pipe Materials

Local approval of culvert pipe materials is required for projects such as those in the local road program, STP program, or in the case where the local government is paying more than 50% of the cost of the pipe. Local approval is not required for roadways classified as State Trunk Highways, Connecting Highways or other roadways on the NHS system, unless the 50% pipe cost participation threshold is exceeded. The local approval is intended to come from the local unit of government or agency participating in the cost of the project, which may not necessarily be the entity responsible for maintenance. In addition, a participating local unit of government or agency may specifically request the installation of concrete, metal, thermoplastic, or the four subclasses of Class III pipe listed in Standard Spec 520 for projects meeting the criteria described in this part.

15.3 Special Situations

Special conditions at the proposed culvert site may require that a specific type of pipe be used. Such special conditions include acidity of soils/water or other corrosive conditions, local preference when meeting the conditions described in FDM 13-1-15.2.1, limited cover (see FDM 13-1-15.6), extending existing culvert pipes, unusual loading from high embankments, steep gradients, or other pertinent reasons.

15.4 Corrosion Concerns About Steel Culvert Pipe

Corrosion of zinc-coated (galvanized) steel pipe results from different mechanisms in different regions of the state. A Wisconsin map outlining the potential areas for bacterial corrosion of zinc galvanized steel culvert pipes is shown on Attachment 15.1. In the north and central part of Wisconsin (Area 1, Figure 15.1), corrosion of steel pipe is due mainly to the activity of anaerobic sulfate reducing bacteria (ASR) in the surface water. This region is characterized by low alkalinity of the surface water. These ASR bacteria do not attack the steel directly but create an environment favorable to corrosion. Corrosion resistant pipe should be specified for use in Area 1 except for commonly dry sites where existing zinc-coated (galvanized) steel pipe have not had a history of corrosion.

In Area 2, zinc-coated (galvanized) steel pipe should be used only at sites where surface water has a minimum alkalinity of 120 milligrams per liter or where existing zinc-coated (galvanized) steel pipe at the site have had an acceptable service history. Metal culvert pipe of any type should provide a minimum service life of 20 years before perforation occurs.

In the remainder of the state (Area 3), corrosion is more commonly related to local conditions such as high electrical conductivity of water and fine-grained soil. Other contributing factors would include high or low pH of soil or water and the presence of ASR bacteria in organic, poorly drained soil.

Corrosion resistant pipe may be necessary where drainage originates in bogs, swamps, barnyards or low-lying lands drained by ditches or tile. An acceptable corrosion resistant pipe should be specified in Area 3 when the pH is outside the range of five to nine and the resistivity is below 2000-ohm centimeters, or when the resistivity is below 1000-ohm centimeters regardless of the Ph. Acceptable corrosion resistant pipe materials are concrete, aluminum, aluminized steel, polymer coated steel, polyethylene and polypropylene.

* Note: Inspection of several aluminum drainage structures in 1993 revealed localized corrosion of the top and sides of the center sections of the structures. The corrosion appears to be related to the use of chlorides for snow and ice removal. The use of aluminum pipe should therefore be limited to side drains and highways with traffic volumes under 1500 Design AADT unless some provision is made to insulate the upper surface of the structure from infiltrating road salt.

Information about the corrosive characteristics of the soil or water at a site may already be available from region soils or maintenance records. In some cases, it may be necessary to conduct field and laboratory tests to determine whether corrosive conditions exist. The region Soils Engineer can normally advise the designer about the need for such tests and conduct them if needed.

As conditions allow, and with the exceptions listed, Culvert Pipe Class III-A Non-metal, and Culvert Pipe Class III-B Non-metal under Standard Spec 520 are to be specified for culverts in corrosive conditions where ADT is less than or equal to 20,000. Reinforced concrete pipe is required for culverts under high volume roadways (ADT>20,000).

15.4.1 Corrosion Concerns for Concrete Pipe

Where existing reinforced concrete pipe has corroded consider specifying thermoplastic pipe under Standard Spec 530 for roadways with ADT's up to 20,000. Where corrosion has occurred in concrete pipes under high volume roadways (ADT>20,000), contact one of the statewide drainage engineers in the Roadway Design Standards Unit for assistance.

15.4.2 Corrosion Concerns for Steel Endwalls

Where corrosion resistant pipe materials or coatings are specified for a project similar treatment of the endwalls may be necessary. In the case of Culvert Pipe Class III-A and Class III-B items consider the need for a special provision article requiring aluminum apron endwalls meeting the requirements of Standard Spec 525 for corrugated polyethylene and corrugated polypropylene pipe culvert installations.

15.5 Abrasion Concerns

The thickness of metal pipe should be increased, or the pipe invert paved where water velocity combined with a bed load of sand, gravel or stone is likely to cause significant erosion or abrasion of the pipe invert. The existence of abrasive conditions at a proposed culvert site can be determined from inspection of the existing metal pipe at the site or inspection of other pipes in the same general area or on the same watercourse.

15.6 Limited Clearance Installations

When a low clearance pipe is required, the designer may call for any of the following.

- Reinforced concrete elliptical pipe
- Corrugated steel or aluminum pipe arch
- Structural plate pipe arch
- Aluminum structural plate pipe arch.

Due to limited availability, use of concrete arch pipe is discouraged. However, it may be warranted based on special hydraulic or aquatic organism passage (AOP) design requirements. When specifying concrete arch pipe however, or when it is requested by a regulatory agency for AOP, be aware availability is very limited in Wisconsin and horizontal elliptical pipe may be the only viable option for a limited clearance concrete pipe.

***Table 15.3* Culvert Material for Arch or Elliptical Culverts**

BID ITEM(S)	DESIGN ADT	ALLOWABLE SIZES (Inches)	NOTES
Pipe-Arch Corrugated Steel	Up to 20,000	17 x 13 to 83 x 57 (Pipe Arch)	- Not to be used in corrosive environments unless polymer or aluminum coated. See FDM 13-1-15.4. - Indicate required thickness in Misc. Quantities. - Refer to FDM 13-1 Attachment 25.4 for appropriate fill heights - Not allowed on Interstate Highways, or Divided US Highways unless for temporary use or maintenance crossovers in the median.
Reinforced Concrete Horizontal Elliptical Pipe Culverts	All Volumes	14 x 23 to 68 x 106 (Horz. Elliptical)	- Refer to FDM 13-1 Attachment 25.9 for appropriate fill heights. - Arch sizes can be specified by SPV item but availability may be limited.
Pipe-Arch Corrugated Aluminum	Up to 1,500	17 x 13 to 71 x 47 (Pipe Arch)	- Indicate required thickness in Misc. Quantities. - Refer to FDM 13-1 Attachment 25.7 for appropriate fill heights. - Can only be specified as SPV item

15.7 Culvert Selection Justification

When special situations require the use of a non-standard type, shape or coating of pipe; relevant information to that determination should be included on the Stormwater-Drainage-WQ Report Spreadsheet (See FDM 13-1-10.4.

15.8 Tied Joints

Reinforced concrete pipe culverts are required to be tied at the joints with joint ties to prevent separation of adjacent pipe sections. This is required at the last three joints on the upstream and downstream ends of concrete culvert and concrete cattle pass installations. If using apron endwalls, the joint is tied at the endwalls and the next two pipe to pipe joints. No ties are required on culverts with masonry endwalls unless the plans show otherwise. Refer to Standard Spec 520 - pipe culverts. Include the standard detail drawing "Joint Ties for Concrete Pipe" when using concrete culvert and concrete cattle pass pipe.

Restraining all the joints in a pipe installation with ties is very costly and should rarely be necessary. Where soil conditions or past experience with separation of RCCP sections at joints seems to justify an extensive use of pipe ties, a metal or thermoplastic pipe may be a more cost-effective pipe material.

Joint ties are not required for thermoplastic pipe where a full (+/- 20 foot) pipe section is utilized from the infall and outfall to the first joint. Where a partial pipe section must be used at the infall or outfall end, it should be restrained with a manufacturer supplied external mechanical coupling, a mastic impregnated geotextile wrap with mechanical fastening bands, or concrete collar. Apron endwalls shall be secured to the pipe. No ties are required on pipes with masonry endwalls unless the plans show otherwise.

15.9 Height of Cover for Culvert Pipes

Height of cover for the pipe materials in Table 15.2 and Table 15.3 shall be in accordance with the fill height tables referenced in the table notes and as described in FDM 13-1-25.

Required minimum cover for Culvert Pipe Class III-A, Culvert Pipe Class III-A Non-metal, Culvert Pipe Class III-B, and Culvert Pipe Class III-B Non-metal shall be 2 feet measured from top of pipe to top of subgrade.

For steel and concrete pipe, the desired minimum cover shall be 2 feet measured from top of pipe to top of subgrade. Exception to this requirement can be made, and minimum cover reduced, based on pipe class and the minimum cover values listed in the fill height tables.

When breaker run or a similar material is placed for subgrade stabilization, and it is not a part of the pavement structure, it can be counted towards required subgrade cover for the purposes of compliance with this part.

Where less than two feet of subgrade cover is provided special measures may be required during construction to minimize equipment loading impacts on the pipe. At a minimum, locations with reduced subgrade cover should be identified on the plans so that the contractor can take precautionary measures.

15.10 Roughness Coefficient for Culvert Pipe

If a specific pipe material is specified by the designer, a Manning's roughness values appropriate to the material shall be selected. For example, when a reinforced concrete culvert is specified for a project with ADT>20,000 a Manning's roughness value of 0.013 should be used for the design.

Where the contractor is allowed to select from two or more pipe materials, the more restrictive Manning's roughness value should be used for design. For example, a Culvert Pipe Class III-A culvert allows steel, reinforced concrete, high density polyethylene or high density polypropylene pipe. In this case a Manning's roughness of 0.024 for corrugated steel should be used for design. For Culvert Pipe Class III-A Non-metal culverts only reinforced concrete, high density polyethylene or high density polypropylene pipe are allowed and the designer should use a Manning's roughness value of 0.013 accordingly.

LIST OF ATTACHMENTS

Attachment 15.1 Potential for Bacterial Corrosion of Zinc Galvanized Steel Culvert Pipe (Map)

FDM 13-1-17 Storm Sewer Material Selection Standard

February 18, 2020

17.1 Application

This procedure provides guidelines for the selection of storm sewer materials.

WisDOT has approved concrete and thermoplastic pipe as suitable materials for storm sewers. The standards in this procedure apply to both round and elliptical storm sewers.

These standards are based on the expected service life of the material, the highway facility type, and the location of the pipe. Service life depends on the proper structural design and installation of the pipe. These factors are considered in the Fill Height Tables of FDM 13-1-25 as well as the standard specifications and the appropriate special provisions for individual projects.

17.2 Selection Standard

Selection of pipe materials is to be based on Table 17.1 with consideration to size, facility type and fill height in addition to the special situations and site conditions as described in FDM 13-1-17.3 to FDM 13-1-17.5. Storm sewer material selection does not have an ADT restriction.

As conditions allow, and with the exceptions listed, Storm Sewer Pipe Class III-A, and Storm Sewer Pipe Class III-B under Standard Spec 608 shall be specified for storm sewers where the diameter is 36-Inches or less.

These Class III-A and Class III-B bid items allow the contractor to choose between concrete pipe and thermoplastic pipe (corrugated polyethylene and corrugated polypropylene) for sizes up to 36 inches in diameter. As described in FDM 13-1-17.3.1, the intent of these Class III-A and Class III-B items is to introduce potential project cost reductions into the competitive bid process by allowing the contractor to select from multiple material options for pipes sized up to 36 inches.

Class III-A and Class III-B storm sewer differ as follows:

- Class III-A
    - includes Class II and Class III reinforced concrete, corrugated polyethylene, and corrugated

polypropylene.
  - Class III-A has a maximum fill height of 11 ft.
- Class III-B
  - includes Class III reinforced concrete and corrugated polypropylene
  - Class III-B has a maximum fill height of 15 ft.

Reinforced concrete pipe is required for storm sewers greater than 36-Inches in diameter although some exceptions are allowed as described in FDM 13-1-17.3.1.

Once it has been determined which storm sewer materials are suitable for a specific project or site, it may be required to get the approval of affected local government officials prior to developing final plans and specifications. FDM 13-1-17.3.2 describes when local approval is required for projects.

17.3 Approved Materials

The materials shown in Table 17.1 below may be used with the following restrictions.

TABLE 17.1 Storm Sewer Materials Selection Criteria

Diameter ≤ 36-Inches on all Roadways Excluding Interstate Highways or Divided US Highways			
BID ITEM (Storm Sewer Pipe)	**DESIGN ADT**	**ALLOWABLE SIZES (Inches)**	**NOTES**
Class III-A	All Volumes	12 - 36	- Max fill height of 11 ft. - Min. fill height 2 ft. from top of subgrade.
Class III-B	All Volumes	12 - 36	- Max fill height of 15 ft. - Min. fill height 2 ft. from top of subgrade.
Reinforced Concrete	All Volumes	12 - 36 (Round) (1) 42 - 108 (Round) (2)	- (1) 12-36-inch sizes can only be used in special situations. See FDM 13-1-17.4. - (2) Maximum size for concrete pipe varies by pipe class. - Refer to FDM 13-1 Attachment 25.1 and 25.2 for appropriate fill heights for round pipe.
Composite	All Volumes	6 - 15	- Min. fill height 2 ft. from top of subgrade. - Consider for use in special situations. See FDM 13-1-17.4.
Diameter > 36-Inches, Interstate Highways or Divided US Highways, or Horizontal Elliptical Pipe			
BID ITEM (Storm Sewer Pipe)	**DESIGN ADT**	**ALLOWABLE SIZES (Inches)**	**NOTES**
Reinforced Concrete	All Volumes	12 – 108 (Round) (2) 14 x 23 to 68 x 106 (Horz. Elliptical) (2)	- Refer to FDM 13-1 Attachment 25.1 and 25.2 for appropriate fill heights for round pipe. - Refer to FDM 13-1 Attachment 25.9 for appropriate fill heights for horizontal elliptical. - (2) Maximum size for concrete pipe varies by pipe class.

Note: Thermoplastic pipe is allowed under any roadway type at any ADT when used for bridge deck drainage, slotted vain drains, temporary use, or at maintenance crossovers in the median.

17.3.1 Criteria for Use of Storm Sewer Pipe Class III-A and Class III-B Bid Items

The objective of Class III-A and Class III-B bid items is to take advantage of advances in materials technology. When new materials are approved for use on WisDOT projects, the competitive bidding process is enhanced. The Storm Sewer Pipe Class III-A and Class III-B bid items allow contractors to bid based on total installed cost for multiple materials options which should result in the lowest total cost for the project. Therefore, the Storm Sewer Pipe Class III-A and Class III-B bid items shall be utilized on all WisDOT projects, regardless of ADT, where conditions allow and subject to the following:

1. Local approval is granted when required for projects meeting the criteria in FDM 13-1-17.3.2.
2. The diameter of the pipe may not exceed 36 inches.
3. Unless a special situation as defined in FDM 13-1-17.4 applies.
4. Storm Sewer Pipe Class III-A and Class III-B is not allowed on Interstate Highways or Divided US Highway unless for temporary use or at maintenance crossovers in the median. When any of these materials is used on Interstate Highways or Divided US Highway for temporary use or at maintenance crossovers in the median, it is at the designer's discretion and there is no ADT restriction. There are two additional exceptions to the prohibition on thermoplastic storm sewer on Interstate and divided US Highways. The exceptions are the use of thermoplastic materials for inlets serving bridge deck drainage (SDD 8D3) and PVC pipe used for slotted vane drains (SDD 8D14). The reason being is that these types of installations take place outside of the travelled way limits or are encased in concrete.

Exceptions to these conditions may be granted at locations determined in cooperation with the Bureau of Technical Services and Roadway Design Standards Unit for gaining additional experience with the materials in a variety of conditions.

While Class III-A and Class III-B bid items shall be used whenever possible, some discretion is left to the designer on roadways with fill height, high groundwater, or other material limitations. Designers are not expected, for example, to change materials back and forth between manholes as fill heights change, or call out a few individual short runs of Class III-A or B pipe on a site where it otherwise doesn't fit the conditions. In addition, there may be situations where the selection of a specific material is justified such as specifying concrete pipe or thermoplastic pipe to match an existing pipe material.

WisDOT has traditionally taken a conservative approach to the implementation of the use of new materials such as thermoplastic pipe. However, thermoplastic pipe is not a new material to the Department as it has been utilized throughout the state for many years without significant issues in advance of the development current standards. Continued monitoring of the performance of these materials in the field will take place, and standards will be adjusted as necessary.

17.3.2 Local Approval of Storm Sewer Materials

Local approval of storm sewer materials is required for projects such as those in the local road program, STP program, or in the case where the local government is paying more than 50% of the cost of the pipe. Local approval is not required for roadways classified as State Trunk Highways, Connecting Highways or roadways otherwise on the NHS system, unless the 50% pipe cost participation threshold is exceeded. The local approval is intended to come from the local unit of government or agency participating in the cost of the project, which may not necessarily be the entity responsible for maintenance. In addition, a participating local unit of government or agency may specifically request the installation of concrete, thermoplastic, Storm Sewer Pipe Class III-A or Storm Sewer Pipe Class III-B for projects meeting the criteria described in this part.

17.4 Special Situations

Special conditions at the proposed storm sewer site may require that a specific type of pipe be used. Such special conditions include; local preference when meeting the conditions described in FDM 13-1-17.3.2, limited cover, extending existing storm sewer, unusual loading from high embankments, steep gradients, or other pertinent reasons. Additional special situations where a particular pipe material, such as composite pipe, may be desirable include storm water control BMP's outside of traffic areas, very short pipe runs between adjacent inlets or where a pipe less than 12 inches in diameter is required.

17.5 High Groundwater and Buoyancy of Thermoplastic Pipe

All pipe materials, including concrete, are subject to buoyant forces and floatation in saturated conditions. Buoyancy is of concern for thermoplastic pipe due to its light weight. When covered even with minimal roadway pavement, floatation of thermoplastic pipe is not a significant concern. For installations outside the pavement structure, however, high groundwater can be a concern. Examples of this condition are storm sewer running in a median, ditchline, terrace, or other "soil only" areas of cover.

Where high groundwater and fully saturated soil conditions are anticipated, the minimum cover for storm sewer outside the roadway shall be 48 inches for thermoplastic pipe, otherwise reinforced concrete pipe should be specified. For locations where storm sewer is under the roadway pavements, the required minimum 2-foot subgrade cover specified in the FDM is sufficient. Additional depth of cover may be necessary if backfill materials other than the standard foundation and trench backfill materials described in Standard Spec 608 are employed.

The risk of high groundwater conditions can be found from soil boring data such as depth to groundwater or soil morphology. Other resources include; soil mapping (presence of hydric soils), standing water, historic aerial photography, presence of dry weather infiltration in existing storm sewer systems, local well drilling records, USGS data, wetland mapping, field review, and local knowledge.

17.6 Storm Sewer Pipe Connections

17.6.1 Storm Sewer Joints

Standard Spec 608 lists several acceptable joint types for the range of allowable storm sewer materials. In general, these joints are intended to be soil tight. Standard Spec 608 does not specifically require joints to be watertight currently.

Watertight joints are required however in areas of contaminated groundwater and/or soil and may be necessary in areas of high groundwater. In these cases, a special provision article will be necessary to specify watertight joints. Often the AASHTO or ASTM material designations referenced in standard spec 608 contain standards for watertight joints and should be reviewed for applicability to the project conditions. Applicable sections of the Bridge Construction Specifications also reference requirements for watertight joints and can be referenced.

17.6.2 Tied Joints

In certain circumstances, concrete pipe storm sewers are required to be tied at the joints with joint ties to prevent separation of adjacent pipe sections. This is required at the last three joints of the system infalls and outfalls. If using apron endwalls, the joints are tied at the endwall and the next two pipe to pipe joints. No ties are required on storm sewers with masonry endwalls unless the plans show otherwise (refer to Standard Spec 608 - Storm Sewers). Include the standard detail drawings "Joint Ties of Concrete Pipes" when using concrete pipe storm sewers with infalls or outfalls.

Restraining all the joints in a pipe installation with ties is very costly and should rarely be necessary. Where soil conditions or past experience with separation of RCP sections at joints seems to justify an extensive use of pipe ties, the use of thermoplastic pipe materials may be more cost effective.

Joint ties are not required for thermoplastic pipe where a full (+/- 20 foot) pipe section is utilized from the infall and outfall to the first joint. Where a partial pipe section must be used at the infall or outfall end, it should be restrained with a manufacturer supplied external mechanical coupling, a mastic impregnated geotextile wrap with mechanical fastening bands, or a concrete collar. Apron endwalls shall be secured to the pipe. No ties are required on pipes with masonry endwalls unless the plans show otherwise.

17.6.3 Connections at Structures

Currently, WisDOT Standard Specifications and standard detail drawings do not require watertight connections for storm sewer at catch basins, manholes and inlets. Mortared connections between the structure and sewer pipe are required. In areas of groundwater and/or soil contamination or areas otherwise designated as requiring watertight joints, a special provision will be necessary. In preparing a special provision article to address groundwater infiltration into a structure, consider the need for additional waterproofing at joints between structure sections and for joints at risers and castings. A cautious approach should be used when specifying the manner of waterproof connection between the sewer pipe and structure. On projects where multiple material types can be allowed (i.e. Storm Sewer Pipe Class III-A and Class III-B) constructability issues could arise if a specific, or proprietary manner of connection is specified.

17.7 Height of Cover for Storm Sewer

Height of cover for the pipe materials in Table 17.1 shall be in accordance with the fill height tables referenced in the table notes and as described in FDM 13-1-25.

Minimum cover for Storm Sewer Pipe Class III-A, Storm Sewer Class III-B and composite pipe shall be 2 feet measured from top of pipe to top of subgrade where the pipe is under pavement. Additional cover is required when high groundwater may be encountered per FDM 13-1-17.5.

For concrete pipe, the desired minimum cover shall be 2 feet measured from top of pipe to top of subgrade. Exception to this requirement can be made based on pipe class and the minimum cover values listed in the fill height tables and whether the pipe is located outside the limits of current or potential future roadway pavements.

When breaker run or a similar material is placed for subgrade stabilization, and it is not a part of the pavement structure, it can be counted towards required subgrade cover for the purposes of compliance with this part.

Where less than two feet of subgrade cover is provided special measures may be required during construction to minimize equipment loading impacts on the pipe. At a minimum, locations with reduced subgrade cover should be identified on the plans so that the contractor can take precautionary measures.

17.8 Roughness Coefficient for Storm Sewer

A constant coefficient of roughness value of 0.013 should be used in the Manning Formula for all the storm sewer materials described in this procedure.

FDM 13-1-20 Large Drainage Conduit
December 18, 2015

20.1 Introduction

Large drainage conduit is defined in general as conduit larger than 84 inches in equivalent diameter, which equates in cross-sectional area to 38.5 square feet. This size was selected because it is near the top of the range of sizes at which pipe can be factory assembled while still being a practical size for transporting.

The types of large conduit available include structural plate pipe and structural plate pipe arch (AASHTO m167), aluminum alloy structural plate pipe and pipe arch (AASHTO m219), steel pipe with 3" x 1" corrugations (AASHTO m36), reinforced concrete pipe (AASHTO m170), reinforced concrete arch pipe (AASHTO m206), reinforced concrete elliptical pipe (AASHTO m207), and cast-in-place or precast box culverts (AASHTO m259).

The selection of a specific type of large conduit should be made based on economics unless other considerations dictate the need for a particular type of large conduit. Other factors that should be considered include the availability of the conduit in the area of the project; foundation conditions at the project site; time available for construction, including consideration of how traffic will be handled; and the existence of corrosive or abrasive conditions at the site. Special hydraulic requirements, aquatic organism passage, or limited clearance conditions may require the use of a corrugated steel pipe arch, structural plate pipe arch, or wide box culvert.

Two or more conduit types may be specified as equal alternates when either type will satisfy design requirements. For example, aluminum structural plate pipe arch and (steel) structural plate pipe arch could be specified as equal alternates.

Multiple lines of pipe culverts or pipe arches may also be a feasible alternative to large drainage conduit.

FDM 13-1-21 Precast Box Culverts
December 18, 2015

21.1 Introduction

Precast box culverts are one of the large drainage conduit alternatives the designer may choose to resolve a given drainage problem. The choice of this option should be based on the criteria given in FDM 13-1-20 as well as sound engineering judgment. One factor that must be considered is earth cover. Fill height criteria for similarly sized cast-in-place culverts may be used, except precast box culverts may be used only in those situations which provide for at least two feet of earth cover under the traffic areas.

The broad range of sizes offers the designer many choices when studies indicate large drainage conduit is suitable. Multiple cell installations are permitted.

When determining whether a box culvert should be precast or cast-in-place, an analysis should be conducted to compare the options. This analysis should attempt to identify all the factors involved, including costs, many of which are not readily apparent.

Generally, initial cost of a cast in place box is less expensive than a precast box culvert. However, precast box culvert installation can be completed in a much shorter time than a cast-in-place option. This is especially of value where a detour is not feasible, and a short-term closure can be allowed. Precast box culverts may be used in emergency situations. In situations where complete closure is impossible, precast units can be used in a bypass, and then left in place or reset to a new position. Some local roads can carry detour traffic for short durations but cannot sustain long-term use without costly maintenance and repair. Road user costs, such as delays due to indirection, may be a factor. Grading projects may realize a cost advantage by providing early access to an entire project, expediting movement of embankment materials and other construction operations. The minimum time and amount of disruption to streams is an easily identified positive environmental aspect.

Quality control of materials and curing conditions is an advantage to casting the units in a plant environment. The dry mix used in the units yields a denser, less permeable concrete than the cast-in-place option.

End treatments may be precast, cast-in-place, or a combination of both.

If a precast box culvert is selected for a particular design project, the designer shall notify the Bureau of Structures (BOS) early in scoping or design phase. If project is designed by a consultant, preliminary plans and complete final structure plans are required to be sent to BOS for approval. Please refer to 36.12 of the Bridge Manual

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/bridge-manual.aspx

FDM 13-1-25 Fill Height Tables

November 30, 2018

25.1 Design Criteria

The fill height tables included in this procedure are based on the following design criteria:

1. Weight of Embankment: 120 lbs/ft^3
2. Backfill: Good side fill material compacted to 90 percent of standard density based on AASHTO T 99. Modulus of passive soil resistance, E' = 1050 psi. Soil stiffness coefficient, K = 0.33.
3. Installation Type: Class C bedding, in accordance with AASHTO standards at the time of adoption [1]. The only exception to this bedding requirement is shown in Fill Height Table (Attachment 25.3), where a Class B bedding is required for reinforced concrete pipe placed under fill heights in excess of 35 ft. (see Attachment 25.2). Load factors for the zero-projecting embankment condition were used in the fill height determinations.

   For pipe arch structures, the confining backfill must be capable of supporting a corner pressure of two tons per square foot.
4. Safety factors: 4 for longitudinal seams; 2 for buckling.
5. Materials and fabrication: In accordance with the appropriate AASHTO specification as required by the Standard Specifications or special provisions.

25.2 Design Methods

The fill height tables for flexible conduit were developed using the service load design method described in the AASHTO LRFD Bridge Design Specifications. The fill height table for reinforced concrete pipe was developed using the design procedure included in the Concrete Pipe Design Manual prepared by the American Concrete Pipe Association.

25.3 Cut Ends

The ends of metal pipe cut as skews or mitered to slope (or both) are not as strong as square ends. Cut ends should be reinforced with concrete headwalls or collars when the bevel is flatter than 2:1 and the skew is greater than 20 degrees.

25.4 Multiple Structures

Where multiple lines of pipes or pipe arches greater than 48 inches in diameter or span are used, they shall be spaced so that the adjacent sides of the pipe are at least one-half diameter or three feet apart, whichever is less, to permit adequate compaction of backfill material. For diameters up to 48 inches the minimum spacing shall be 24 inches.

When multiple lines of pipe have less than half the diameter of the smallest pipe between them and the out-to-out length along the roadway reference line is greater than 20 feet, the pipe installation shall be assigned a B-number by the Region. Coordination with the Bureau of Structures is required in these situations.

25.5 Abrasive or Corrosive Conditions

Metal thicknesses shown in the fill height tables are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, either greater thicknesses or protective coatings should be provided. For structural plate pipe, greater thicknesses may be specified for the plates in the invert.

LIST OF ATTACHMENTS

[1] Class A, B, C and D Bedding Type has been superseded by Installation Types 1-4. At 90% compaction WisDOT's foundation and trench backfill specifications meets or exceeds a Type 2 installation and subsequently the past B or C bedding class. Future fill height tables will refer to the new installation type nomenclature.

Attachment 25.4	Fill Height Tables: Corrugated Steel Pipe Arch, 2- 2/3in x 1/2in Corrugations; and Corrugated Steel Pipe Arch, 3in x 1in Corrugations
Attachment 25.5	Fill Height Table, Structural Plate Pipe Arch, 6inx2in Corrugations
Attachment 25.6	Fill Height Tables: Corrugated Aluminum Pipe, 3in x 1in Corrugations; and Aluminum Alloy Structural Plate Pipe, 9in x 2 1/2in Corrugations
Attachment 25.7	Fill Height Table, Corrugated Aluminum Pipe Arch, 2 2/3in x 1/2in Corrugations
Attachment 25.8	Fill Height Table, Aluminum Alloy Structural Plate Pipe Arch, 9in x 2- 1/2in Corrugations
Attachment 25.9	Fill Height Table, Reinforced Concrete Arch and Elliptical Pipe (all sizes); and Dimensions for Reinforced Concrete Arch and Elliptical Pipe (English)

FDM 13-1-30 Culvert Replacement and Analysis for Perpetuation & Rehabilitation Projects *August 16, 2022*

30.1 Background

As described in FDM 3-5-1, the Department's preservation focus in the asset management roadway delivery program is a practical design approach to system management that maintains acceptable serviceability using improvement strategies that optimizes to the best possible system-wide service at the lowest practicable cost. Central to the Department's practical design approach is to not degrade safety and operations when applying practical design standards to the roadway. While the practical design approach may seemingly only apply to the geometric elements of the roadway, drainage does have an impact on safety and operations and should be evaluated as well. This section describes practices to evaluate and replace or upgrade existing drainage systems on perpetuation and rehabilitation type projects. Similar to geometric elements of the project, under certain conditions the Department allows the application of practical design approaches to drainage systems. While it is best practice to perform hydrology and hydraulic (H&H) analysis for all drainage structures, this section describes criteria for when engineering judgement can be used in lower risk installations utilizing simplified procedures for the analysis of roadway culverts.

30.2 Applicability

The culvert selection practices of this section apply only to perpetuation and rehabilitation roadway segments. This section does not apply to:

- Modernization projects or sections of a project utilizing modernization standards.
- Spot improvements reconstructed for safety or otherwise [2]
- The analysis and sizing of storm sewer systems regardless of project type.
- Any improvements on Interstates, Expressways and Freeways (Non-Interstate Highways), Connecting Highways or the NHS system.

Standard H&H analysis and materials selection requirements, as described elsewhere in this Chapter, is required for all other culverts not meeting the conditions of this section.

30.3 Guidelines for Culvert Replacement on Perpetuation and Rehabilitation Projects

30.3.1 Evaluation and Identification

Drainage structures will typically remain intact with perpetuation and rehabilitation improvement projects. For these projects evaluate the existing drainage structures along the corridor to identify signs of failure, excessive erosion, or indications of undersizing or recurring flooding. Where local testimony or other evidence indicates a recurring flooding issue, a hydrology and hydraulics (H & H) analysis is required to determine if the structure is appropriately sized. Replacement of drainage structures may be included within a perpetuation or rehabilitation project if the existing drainage structure is determined to be nearing a failure threshold or demonstrated to cause recurring flooding.

Identifying the size, type, and condition of the culverts within the project limits should be performed early in the scoping process. Evaluating the condition of the culvert not only includes the physical condition of the structure itself but also looking for signs that the structure is not adequately sized. It is also recommended to engage WDNR during scoping if Aquatic Organism Passage could be a concern. AOP culvert sizing almost always will significantly increase the size of a culvert when compared to the existing. (see FDM 13-1-30.3.2.2 for additional

[2] Where spot improvements are made on a perpetuation or rehabilitation project due to safety or otherwise, the upgraded portion of the roadway shall include culverts and drainage features designed using WisDOT's standard hydrology and hydraulic (H&H) analysis and materials selection practices.

discussion on AOP coordination).

In most cases, Regional maintenance staff will identify the size, type and physical condition of the culverts within the project limits and identify culverts in need of replacement or repair. This assessment may be performed in cooperation with Bureau of Structures for large culverts (25 sf or >60 inches). When recommending culverts in need of replacement, consideration shall be given to if the remaining service life of the culvert meets or exceeds the pavement treatment surface life for the planned improvement. For example, if the culverts on an individual project appear to have 15 or more years of service life remaining it would be ill advised to replace functioning culverts if the proposed improvement type has a 10-year pavement treatment service life.

Once all the culverts in need of replacement due to physical condition are identified, the regional Stormwater and Erosion Control Engineer, or an engineering working under their direction, shall inspect and further evaluate the culverts, especially those under consideration for 'in-kind' replacement per FDM 13-1-30.3.2 to determine if there are concerns with the size of the existing culverts. Signs of an undersized culvert can include:

- Erosion/scour at the inlet and/or outlet
- Excessive sediment on the upstream side of the crossing
- Frequent accumulation of debris
- Increased depth of flow upstream and downstream of the culvert
- A significant increase in development (impervious area) within the culvert's drainage area
- Evidence of roadway overtopping such as downstream shoulder erosion or washouts
- Plunge pools, scour, culvert perching on the downstream side of the culvert.

Regional maintenance staff or local officials may also be aware of past issues related to flooding or erosion at a culvert site.

***Figure 3.1* Erosion Damage to Downstream Embankment Slopes from Previous Overtopping - Sources FHWA and Utah DOT**

***Figure 3.2* Scour Holes and Perched Culverts – Sources FHWA and WisDOT**

***Figure 3.3* Stable and Unstable Channels Downstream of Culvert – Source FHWA and UDOT**

30.3.2 Simplified Culvert Sizing Evaluation

30.3.2.1 Background

As stated previously, while it is best practice to perform hydrology and hydraulic (H&H) analysis for all drainage structures, this section describes criteria for when engineering judgement can be used in lower risk installations utilizing simplified procedures for the analysis of roadway culverts. For perpetuation and rehabilitation roadway segments this involves “in kind” replacement of drainage structures under limited conditions.

30.3.2.2 Criteria for “In-Kind” Replacement

For a culvert to be eligible for “in kind” replacement on perpetuation and rehabilitation roadway segments, the following additional criteria apply:

- Pavement treatment service life < 18 years.
- No clear signs or evidence of undersizing have been observed or reported.
- The culvert is located in rural or undeveloped areas or otherwise outside municipal boundaries and outside populated areas (when in doubt see Attachment 30.1 for guidance).
- Not located in a TS4 Permitted Area. (TS4 areas are transportation facilities with MS4 areas defined by Wisconsin DNR https://dnr.wi.gov/topic/stormwater/data/Municipal/. Contact WisDOT’s stormwater coordinator in the Bureau of Technical Services, Environmental Services Section when in doubt).
- ADT < 7,000.
- The fill height for the culvert does not exceed 15 feet.
- The proposed culvert slope meets or exceeds the slope of the existing culvert.
- The culvert is not extended more than 10% of its existing length.
- Culvert diameter ≤ 48 inches.
- The total open area of the culverts does not exceed 15 sf for multiple culverts in place at a single crossing.
- The project is not located in rolling terrain (primarily areas of southwest and central Wisconsin - see FDM Chapter 11 and Highway Capacity Manual, 7th Edition (Chapter 12, Section 3).
- No structures (buildings) are located immediately upstream and are at least 2 feet higher than the point of roadway overtopping.
- No valuable properties or unique resources are located immediately upstream.
- The culvert is not located in a floodplain, drainage district (FDM 5-15-1) or mapped perennial or intermittent stream. WDNR’s surface water data viewer can assist in locating these resources (https://dnr.wi.gov/topic/surfacewater/swdv/).
- The Wisconsin Department of Natural Resources has not identified Aquatic Organism Passage Concerns (AOP) for the culvert in question. [Note: The Regional Environmental Coordinator and/or Storm Water and Erosion Control Engineer must agree with WDNR with the need for AOP consideration. A WDNR request for AOP consideration alone does not warrant upsizing structures without regional concurrence.]

Please note that a single culvert with the project limits not meeting these additional criteria does not exclude the remaining culverts from “in-kind” replacement.

30.3.2.3 Confirmation of "In-Kind" Replacement

To confirm field observations, or where evaluation of a culvert is otherwise inconclusive, the tables in Attachment 30.2 offer a check of culvert size for "replace in kind" structures. The tables trend towards being conservative and are intended for small watersheds typical to the maximum "replace in kind" culvert size described in this part. These tables shall not be used to size culverts requiring complete hydrology and hydraulic (H&H) analysis such as those on modernization projects or segments of a project using modernization standards (see FDM 13-1-30.2). In those cases, however, the tables can still be used as part of the QA/QC of the H&H drainage design.

The tables require the user to have a general idea of land cover, soil type, and watershed area. This does not have to be an extensive delineation and characterization of the watershed. Only the basic characteristics of the watershed are required. The tables assume a time of concentration based on the size of the watershed.

This check should also be only part of the evaluation of "in kind" replacement. The tables are not meant to dictate the need to increase or reduce the size of an existing culvert, they are intended as a check. Still, in the event the in-place culvert size and the tabulated size are substantially different, a full H&H analysis may be appropriate.

30.3.2.4 Determining Watershed Area

There are numerous methods and tools available to size and characterize a watershed. GIS tools (ArcGIS online), surface models in design software, digital topographic maps, and many other resources can be used to delineate a watershed. There are also online and software-based tools that can approximate a watershed boundary in a manner of minutes. The user should keep in mind the digital terrain employed by these tools may not be high resolution and/or up to date and results need to be scrutinized. This is especially the case in flat areas where low resolution terrain models may not depict breaks in drainage from roadways, small embankments or depressions, or other localized formations. Some available resources for delineating watersheds include:

- Civil 3D – WisDOT's standard design software, Civil 3D, includes tools for delineating the boundary of a watershed. Results will depend on the quality of the surface model created for the project. WisDOT maintains training videos for various design tasks including inserting aerial images, creating surface models and defining culvert 'catchments'. When in doubt it is good to compare results to USGS maps or similar contour maps to affirm the accuracy of the results.
- General Civil 3D Training:

  https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/civil3d/civil-train.aspx
- H&H in Civil 3D Training:

  https://c3dkb.dot.wi.gov/Content/c3d/hydro-dsn.htm
- USGS Streamstats - Some larger watersheds can be delineated using USGS's streamstats online tools. The user zooms in to the area of interest and the available stream data shows up as pixilated threads. https://streamstats.usgs.gov/ss/

In addition to these tools many counties have LIDAR generated contours available on their GIS sites. This may be the most accurate of the readily accessible public data. In most cases the user will have to delineate the boundary using online measurement tools and characterize the watershed using the aerial photography available on these sites.

30.4 Culvert Materials on Perpetuation and Rehabilitation Projects

Replacement of culverts "in-kind", as described in this chapter, does not require that the new culvert be the same material as the existing culvert. More cost effective or site appropriate materials may be available. For example, it would not make sense to replace a metal culvert that has corroded due to soil conditions with the same type of metal culvert. In this case a coated metal culvert, class III-A or III-B non-metal culvert, or concrete culvert of the same diameter may be more appropriate. In selection of a culvert material for "in-kind" culvert replacement projects the criteria of FDM 13-1-15 apply with exceptions shown in Table 30.1.

***Table 30.1* Culvert Materials for "In-Kind" Replacement on Perpetuation and Rehabilitation Projects**

Existing Material	Replacement Requirements
Corrugated Metal	- Metal, Concrete, Class III-A or III-B, or Class III-A or III-B Non-Metal in conformance with FDM 13-1-15 - Refer to FDM 13-1 Attachment 25.1 - 25.9 for allowable fill heights
Reinforced Concrete (1)	- Concrete or Class III-A or III-B Non-Metal in conformance with FDM 13-1-15 - Refer to FDM 13-1 Attachment 25.1 - 25.9 for allowable fill heights
Thermoplastic (1)	- Concrete or Class III-A or III-B Non-Metal in conformance with FDM 13-1-15 - Refer to FDM 13-1 Attachment 25.1 - 25.9 for allowable fill heights

Note:

1. Reinforced concrete and thermoplastic culvert pipes shall not be replaced 'in-kind' with corrugated metal pipe due to significant differences in manning's roughness.

30.5 Culvert Extensions, Endwalls and Traversable Grates on Perpetuation and Rehabilitation Projects

30.5.1 Culvert Extensions, and Traversable Grates on Perpetuation and Rehabilitation Projects

Lengthening proposed culvert replacements, extending existing culverts beyond the clear zone, or adding traversable grates to replacement or existing culverts (Apron Endwalls for Culvert Pipe Sloped Cross Drains or Apron Endwalls for Culvert Pipe Sloped Side Drains) are not required for Perpetuation and Rehabilitation projects where S-1 standards are applied.

For Rehabilitation projects subject to S-2 standards, or areas of a project employing S-2 standards, consider lengthening replacement culverts or extending existing culverts beyond the clear zone. Remove/remedy adjacent hazardous drainage features identified in the roadside hazard evaluation or analysis process (RHA). Consideration shall be given to areas with identified crash history or areas subject to high "run off the road" crashes such as the outside of sharp horizontal curves. Examples of additional locations with high "run off the road" crashes can be found in FDM 11-45-20.4.2. Note that culvert replacements with improvement projects may be predicated on existing structural, hydraulic capacity or maintenance issues and not on exclusively on existing roadside hazardous conditions.

Some limiting factors for lengthening culverts within S-2 areas could include need for right of way acquisition, environmental concerns (wetlands, floodplains, endangered or threatened species) or cost justifications that consider maintenance and crash cost. In evaluating factors limiting extensions also consider future plans for upgrading the facility. There may be a benefit to lengthening structures to more easily accommodate future improvements.

Where culverts within S-2 areas cannot be extended due to limiting factors consider installing traversable grates. Some limiting factors for traversable grates may include where increased headwater or potential for debris accumulation threaten adjacent properties, environmental concerns (wetlands, floodplains, endangered or threatened species) or cost justifications that consider maintenance and crash cost.

Refer to FDM 11-38 for details on the Safety Certification Process (SCP) and to FDM 11-45-20 for further guidance on the RHA process. For all improvement projects, document final decisions and outcomes with roadside hazard evaluations and treatments Design Study Report (DSR) especially those not identified in the initial Safety Certification Document (CSD).

30.5.2 Endwalls on Perpetuation and Rehabilitation Projects

Culverts replaced on Perpetuation and Rehabilitation projects shall have standard endwalls installed even when endwalls are not in place on the existing culvert. Where an existing culvert does not have an end wall and is not scoped for replacement, there is no requirement to install endwalls on the existing pipe.

30.99 Resources

The following is a brief list of useful resources for learning more about evaluating culverts.

Assessment:

FHWA. (2010). Culvert Assessment and Decision-Making Procedures Manual. Lakewood, CO.

FHWA (2014). Hydraulic Toolbox Version 4.2. [Offers hydraulic tools including a culvert assessment tool based on the 2010 Culvert Assessment and Decision-Making Procedures Manual.]

Design:

Federal Highway Administration. Culvert hydraulic analysis program and supporting documentation, HY-8, Version 7.5. 2016.

Federal Highway Administration. Hydraulic Design of Highway Culverts Hydraulic Design Series Number 5 (HDS 5) Third Edition, FHWA-HIF-12-026. 2012.

FHWA (2014). Hydraulic Toolbox Version 4.2. [Offers hydraulic tools including a culvert assessment tool based on the 2010 Culvert Assessment and Decision-Making Procedures Manual.]

LIST OF ATTACHMENTS

Attachment 30.1	Guidelines for Defining a Rural Area
Attachment 30.2	Culvert Sizing Quick Check

Glossary of Terms

The definitions in this Glossary are for use with this Chapter and the references cited. They are not necessarily definitions as established by case or statutory law.

Acre-Foot: A unit of measurement for volume of water. It is equal to the quantity of water required to cover one acre to a depth of one foot and is equal to 43,560 cubic feet or 325,851 gallons. The term is commonly used in measuring volumes of water used or stored.

Annual Flood: The highest peak discharge in a water year.

Antecedent Precipitation Index: An index of moisture stored within a drainage basin before a storm (Linsley and others, 1949, p. 414).

Area-Capacity Curve: A graph showing the relation between the surface area of the water in a reservoir and the corresponding volume.

Average Discharge: In the annual series of the Geological Survey's reports on surface water supply, the arithmetic average of all complete water years of record, whether or not they are consecutive. Average discharge is not published for less than five years of record. The term "average" is generally reserved for averages of record and "mean" is used for averages of shorter periods, namely, daily mean discharge.

Backwater: An unnaturally high stage in a stream caused by obstruction or confinement of flow, as by a dam, bridge, or levee. Its measure is the excess of unnatural over natural stage, not the difference in stage upstream and downstream from its cause.

Bank: The lateral boundary of a stream confining water flow. The bank on the left side of a channel looking downstream is called the left bank, etc.

Bank Storage: The water absorbed into the banks of a stream channel when the stages rise above the water table in the bank formations, then returns to the channel as effluent seepage when the stages fall below the water table (After Houk, 1951, p. 179.).

Base Flow: See "Base Runoff."

Base Runoff: Sustained or fair-weather runoff. In most streams, base runoff is composed largely of groundwater effluent (Langbein and others, 1947, p. 6). The term "base flow" is often used in the same sense as base runoff. However, the distinction is the same as that between stream flow and runoff. When the concept in the terms "base flow" and base runoff is that of the natural flow in a stream, base runoff is the logical term (also see "Groundwater Runoff" and "Direct Runoff").

Bulking: The increase in volume of flow due to air entrainment, debris, bedload, or sediment in suspension.

Capacity: The effective carrying ability of a drainage structure. Generally measured in cubic feet per second.

Catch Basin: A drainage structure that collects water. May be either a structure where water enters from the side or through a grating.

Cfs: Abbreviation of cubic feet per second.

Cfs-Day: The volume of water represented by a flow of one cubic foot per second for 24 hours. It equals 86,400 cubic feet, 1.983471 acre-feet, or 646,317 gallons.

Cfsm (cubic feet per second per square mile): The average number of cubic feet of water per second flowing from each square mile of area drained by a stream, assuming that the runoff is distributed uniformly in time and area.

Channel Storage: The volume of water at a given time in the channel or over the floodplain of the streams in a drainage basin or river reach. Channel storage is great during the progress of a flood event (see Horton, 1935, p. 3).

Coefficient Runoff: Percentage of gross rainfall that appears as runoff.

Concentrated Flow: Flowing water that has been accumulated into a single, fairly narrow stream.

Concentration: In addition to its general sense, means the unnatural collection or convergence of waters so as to discharge in a narrower width and at greater depth or velocity.

Control: A natural constriction of the channel, a long reach of the channel, a stretch of rapids, or an artificial structure downstream from a gaging station that determines the stage-discharge relation at the gage. That section which determines the stage for a particular reach of a drainage system.

Critical Depth (depth at which specific energy is a minimum):
The depth of water in a conduit at which under certain other conditions the maximum flow will occur. These other conditions are when the conduit is on the critical slope with the water flowing at its critical velocity and when there is an adequate supply of water. The depth of water flowing in an open channel or a conduit partially filled for which the velocity head equals one-half the hydraulic mean depth.

Critical Flow: A condition that exists at the critical depth. Under this condition, the sum of the velocity head and static head is a minimum.

Critical Slope: That slope at which the maximum flow will occur at the minimum velocity. The slope or grade that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; the slope of a conduit that will produce critical flow.

Critical Velocity: Mean velocity of flow when flow is at critical depth.

Cubic Feet Per Second: A unit expressing rates of discharge. One cubic foot per second is equal to the discharge of a stream of rectangular cross section, one foot wide and one foot deep, flowing water an average velocity of one foot per second.

Culvert: A closed conduit, other than a bridge, that allows water to pass under a highway. A culvert has a span of 20 feet or less as measured between the interior walls of the outside bents.

Depression Storage: The volume of water contained in natural depressions in the land surface, such as puddles (After Horton, 1935, p. 2).

Design Discharge: The quantity of flow that is expected at a certain point as a result of a design storm. Usually expressed as a rate of flow in cubic feet per second.

Design Frequency: The recurrence interval for hydrologic events used for design purposes. As an example, a design frequency of 50 years means a storm of a magnitude that would be expected to recur on the average of once every 50 years.

Design Storm: That particular storm that contributes runoff that the drainage facilities were designed to handle. This storm is selected for design on the basis of its probable recurrence; i.e., a 50-year design storm would be a storm for which its maximum runoff would occur on the average of once every 50 years.

Direct Runoff: The runoff entering stream channels promptly after rainfall or snowmelt. Superposed on base runoff, it forms the bulk of the hydrograph of a flood. Also see "Surface Runoff." The terms base runoff and direct runoff are time classifications of runoff. The terms groundwater runoff and surface runoff are classifications according to source.

Discharge: A volume of water flowing out of a drainage structure or facility. Measured in cubic feet per second.

Discharge Rating Curve: See "Stage-Discharge Relation."

Drainage: (1) The process of removing surplus groundwater or surface water by artificial means. (2) The system by which the waters of an area are removed. (3) The area from which waters are drained; a drainage basin.

Drainage Area (Drainage Basin) (Basin):
That portion of the earth's surface upon which falling precipitation flows to a given location. With respect to a highway, this location may be either a culvert, the farthest point of a channel, or an inlet to a roadway drainage system.

Drainage Divide: The rim of a drainage basin. A series of high points from which water flows in two directions, into the basin and away from the basin.

Drainage System: Usually a system of underground conduits and collector structures that flow to a single point of discharge.

Eddy Loss: The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

Effective Precipitation - (rainfall):
(1) That part of the precipitation that produces runoff. (2) A weighted average of current and antecedent precipitation that is "effective" in correlating with runoff. (3) As described by U.S. Bureau of Reclamation (1952, p. 4), that part of the precipitation falling on an irrigated area that is effective in meeting the consumptive use requirements.

Energy Grade Line: A hydraulic term used to define a line representing the total amount of energy available at any point along a watercourse, pipe, or drainage structure. Where the water is motionless, the water surface would coincide with the point or the energy grade line. As the flow of water is accelerated, the water surface drops further away from the energy grade line. If the flow is stopped at any point, the water surface jumps back to the energy grade line.

Energy Head: The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the water in that section.

Entrance Head: The head required to cause flow into a conduit or other structure. It includes both entrance loss and velocity head.

Entrance Loss: The head lost in eddies and friction at the inlet to a conduit or structure.

Equalizer: A drainage structure similar to a culvert but different in that it is not intended to pass a design flow in a given direction. Instead, it is often placed level so as to permit passage of water in either direction. It is generally used where there is no place for the water to go. Its purpose is to maintain the same water surface elevation on both sides of the highway embankment.

Evaporation: A process whereby water as a liquid is changed into water vapor through heat supplied by the sun.

Flood-Frequency Curve:
(1) A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are equaled or exceeded. (2) A similar graph but with recurrence intervals of floods plotted as abscissa (see Dalrymple, 1960).

Flood Peak: The highest value of the stage or discharge attained by a flood, thus peak stage or peak discharge. Flood crest has nearly the same meaning, but since it connotes the top of the flood wave, it is properly used only in referring to stage, thus crest stage but not crest discharge.

Floodplain: Strip of land adjacent to a river or channel that has a history of overflow.

Flood Profile: A graph of elevation of the water surface of a river in flood, plotted as ordinate, against distance, measured in the downstream direction, plotted as abscissa. A flood profile may be drawn to show elevation at a given time, crests during a particular flood, or to show stages of concordant flows.

Flood Routing: The process of determining progressively the timing and shape of a flood wave at successive points along a river (see Carter and Godfrey, 1960).

Flood Stage: The elevation at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured.

Flow Line: A term used to describe the line connecting the low points in a watercourse.

Freeboard : The distance between the normal operating level and the top of the sides of an open conduit; the crest of a dam, etc., designed to allow for wave action, floating debris, or any other condition or emergency, without overtopping the structure.

Flow-Duration Curve: A cumulative frequency curve that shows the percentage of time that specified discharges are equaled or exceeded.

Free Outlet: A condition under which water discharges with no interference such as a pipe discharging into open air.

Gage Height: The water surface elevation referred to some arbitrary gage datum. Gage height is often used interchangeably with the more general term stage, although gage height is more appropriate when used with a reading on a gage.

Gaging Station: A particular site on a stream, canal, lake, or reservoir where systematic observations of gage height or discharge are obtained (also see "Stream Gaging Station").

Grade to Drain: A construction note often inserted on a plan for the purpose of directing the contractor to slope a certain area in a specific direction so that the storm waters will flow to a designated location.

Gradient (Slope):	The rate of ascent or descent, expressed as a percent or as a decimal as determined by the ratio of the change in elevation to the length.
Groundwater Runoff :	That part of the runoff that passed into the ground, has become groundwater, and has been discharged into a stream channel as spring or seepage water (also see "Base Runoff" and "Direct Runoff").
Head:	When used as a hydraulic term, this represents an available force equivalent to a certain depth of water. This is the motivating force in effecting the movement of water. The height of water above any point or plane of reference. Used also in various compound expressions, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc.
Hydraulic Gradient:	A line which represents the relative force available due to the potential energy available. This is a combination of energy due to the height of the water and the internal pressure. In any open channel, this line corresponds to the water surface. In a closed conduit, if several openings were placed along the top of the pipe and open tubes inserted, a line connecting the water surface in each of these tubes would represent the hydraulic grade line.
Hydraulic Jump (or Jump):	Transition of flow from the rapid to the tranquil state. A varied flow phenomenon producing a rise in elevation of water surface. A sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy.
Hydraulic Mean Depth:	The area of the flow cross section divided by the water surface width.
Hydraulic Radius:	The cross-sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter.
Hydrograph:	A graph showing stage, flow, velocity, or other properties of water with respect to time.
Hydrography:	Water surveys. The art of measuring, recording, and analyzing the flow of water, and of measuring and mapping watercourses, shorelines, and navigable waters.
Hydrology:	The science dealing with the occurrence and movement of water upon and beneath the land areas of the earth. Overlaps and includes portions of other sciences such as meteorology and geology. The particular branch of hydrology that a drainage section is generally interested in is surface runoff that is the result of excessive precipitation.
Hyetograph:	Graphical representation of rainfall intensity against time.
Infall:	Point of entrance into a storm sewer system through an apron endwall or pipe opening.
Infiltration:	The passage of water through the soil surface into the ground.
Infiltration Capacity:	The maximum rate at which the soil, when in a given condition, can absorb falling rain or melting snow (After Horton, 1935, p. 2).
Infiltration Index:	An average rate of infiltration, in inches per hour, equal to the average rate of rainfall such that the volume of rainfall at greater rates equals the total direct runoff (Langbein and others, 1947, p. 11).
Inlet Time (i.e., Time of Concentration):	The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point where it enters a drain or culvert.
Interception:	The process and the amount of rain or snow stored on leaves and branches and eventually evaporated back to the air. Interception equals the precipitation on the vegetation minus stemflow and throughfall (after Hoover, 1953, p. 1).
Invert:	The bottom of a drainage facility along which the lowest flows would pass.
Isohyetal Line:	A line drawn on a map or chart joining points that receive the same amount of precipitation.
Isohyetal Map:	A map containing isohyetal lines and showing rainfall intensities.
Isovel:	Line on a diagram of a channel connecting points of equal velocity.
Lag:	Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.
Laminar Flow:	That type of flow in which each particle moves in a direction parallel to every other particle and in which the head loss is approximately proportional to the velocity (as opposed to turbulent flow).
Mass Curve:	A graph of the cumulative values of hydrologic quantity (such as precipitation or runoff), generally as ordinate, plotted against time or date as abscissa (see "Double-Mass Curve" and "Residual-Mass Curve").

Mean Velocity: Average velocity within a cross section.

Meander: The winding of a stream channel.

Normal: A central value (such as arithmetic average or median) of annual quantities for a 30-year period ending with an even 10-year period, thus 1921-50, 1931-60, and so forth. This definition accords with that recommended by the Subcommittee on Hydrology of the Federal Inter-Agency Committee on Water Resources.

Normal Depth: The depth at which flow is steady and hydraulic characteristics are uniform.

Outfall: Discharge or point of discharge of a culvert or other closed conduit.

Partial-Duration Flood Series: A list of all flood peaks that exceed a chosen base stage or discharge, regardless of the number of peaks occurring in a year (also called basic stage flood series or floods above a base).

Peak Flow: Maximum momentary stage or discharge of a stream in flood. Design discharge.

Perched Water: Groundwater located above the level of the water table and separated from it by a zone of impermeable material.

Percolating Waters: Waters that have infiltrated the surface of the land and moved slowly downward and outward through devious channels (aquifers) unrelated to stream waters until they reach an underground lake or regain and spring from the land surface at a lower point.

Permeability: The property of soils that permits the passage of any fluid. Permeability depends on grain size, void ratio, shape, and arrangement of pores.

Point of Concentration: That point at which the water flowing from a given drainage area concentrates. With reference to a highway, this would generally be either a culvert entrance or some point in a roadway drainage system.

Potamology: The hydrology of streams.

Precipitation: Rainfall, snow, sleet, fog, dew, and frost.

Rainfall: Point Precipitation: That which registers at a single gauge. Area Precipitation: Adjusted point rainfall for area size.

Rainfall Excess: The volume of rainfall available for direct runoff. It is equal to the total rainfall minus interception, depression storage and absorption (see American Society of Civil Engineers, 1949, p. 106).

Rainfall, Excessive: Rainfall in which the rate of fall is greater than certain adopted limits, chosen with regard to the normal precipitation (excluding snow) of a given place or area. In the U.S. Weather Bureau, it is defined for states along the southern Atlantic Coast and the Gulf Coast as rainfall in which the depth of precipitation is 0.90 inch at the end of 30 minutes and 1.50 inches at the end of an hour, and for the rest of the country as rainfall in which the depth of precipitation at the end of each of the same periods is 0.50 inch and 0.80 inch, respectively.

Reach: The length of a channel uniform with respect to discharge, depth, area, and slope. More generally, any length of a river or drainage course.

Recession Curve: A hydrograph showing the decreasing rate of runoff following a period of rain or snowmelt. Since direct runoff and base runoff recede at different rates, separate curves, called direct runoff recession curves or base runoff recession curves, are generally drawn. The term "depletion curve" in the sense of base runoff recession is not recommended.

Recurrence Interval (return period): The average interval of time within which the given flood will be equaled or exceeded once (American Society of Civil Engineers, 1953, p. 1221).

Regimen: The characteristic behavior of a stream during ordinary cycles of flow.

Runoff: The portion of precipitation that appears as flow in streams. Drainage or flood discharge which leaves an area as surface flow or as pipeline flow, having reached a channel or pipeline by either surface or subsurface routes, and includes underflow in some cases.

Scour: Wearing of the bed of a stream by entrainment of alluvium and erosion of native rock. Also caused by excessive velocities at the entrance of a concentrated stream of water onto unstable material. Wearing away by abrasive action.

Second-Foot: Same as cfs. This term is no longer used in published reports of the U.S. Geological Survey.

Silt: (1) Water-Borne Sediment: Detritus carried in suspension or deposited by flowing water, ranging in diameter from 0.0002 to 0.002 inch. The term is generally confined to fine earth, sand, or mud, but is sometime's broadened to include all material carried, including both suspended and bed load. (2) Deposits of Water-Borne Material: As in a reservoir, on a delta, or on floodplains.

Skew: When a drainage structure is not normal (perpendicular) to the longitudinal axis of the highway, it is said to be on a skew. The skew angle is the smallest angle between the perpendicular and the axis of the structure.

Slope: (1) Gradient of a stream. (2) Inclination of the face of an embankment, expressed as the ratio of horizontal to vertical projection. (3) The face of an inclined embankment or cut slope. In hydraulics it is expressed as percent or in decimal form.

Slugflow : Flow in culvert or drainage structure that alternates between full and partly full. Pulsating flow--mixed water and air.

Soffit: The bottom of the top - (1) With reference to a bridge, the low point on the underside of the suspended portion of the structure. (2) In a culvert, the uppermost point on the inside of the structure.

Specific Energy: The energy of a stream referred to its bed, namely, depth plus velocity head of mean velocity.

Stage: The elevation of a water surface above its minimum; also above or below an established "low water" plane; hence above or below any datum of reference; gage height.

Stage-Capacity Curve: A graph showing the relation between the surface elevation of the water in a reservoir, usually plotted as ordinate, against the volume below that elevation, plotted as abscissa.

Stage-Discharge Curve (rating curve): A graph showing the relation between the gage height, usually plotted as ordinate, and the amount of water flowing in a channel, expressed as volume per unit of time, plotted as abscissa.

Storage: Detention or retention of water for future flow, naturally in channel and marginal soils or artificially in reservoirs.

Storage Basin: Space for detention or retention of water for future flow, naturally in channel and marginal soils or articifically in reservoirs.

Relation to time:

Perennial: One that flows continuously.

Intermittent or Seasonal: One that flows only at certain times of the year when it receives water from springs or from some surface source, such as melting snow in mountainous areas.

Ephemeral: One that flows only in direct response to precipitation and whose channel is above the water table at all times.

Relation to space:

Continuous: One that does not have interruptions in space.

Interrupted: One that contains alternating reaches that are either perennial, intermittent, or ephemeral.

Relation to groundwater:

Gaining: A stream or reach of a stream that receives water from the zone of saturation.

Losing: A stream or reach of a stream that contributes water to the zone of saturation.

Insulated: A stream of reach of a stream that neither contributes water to the zone of saturation nor receives water from it. It is separated from the zones of saturation by an impermeable bed.

Perched: A perched stream is either a losing stream or an insulated stream that is separated from the underlying groundwater by a zone of saturation.

Stream Gaging: The process and art of measuring the depths, areas, velocities, and rates of flow in natural or artificial channels (see Corbett and others, 1943).

Stream Gaging Station: A gaging station where a record of discharge of a stream is obtained. Within the Geological Survey this term is used only for those gaging stations where a continuous record of discharge is obtained.

Subcritical Flow: Flow with a velocity head less than half the hydraulic mean depth of water.

Supercritical Flow: Flow with a velocity head more than half the hydraulic mean depth of the water.

Surface Runoff: The movement of water on the earth's surface, whether flow is over surface of ground or in channels.

Tapered Inlet: A transition to direct the flow of water into a channel or culvert. A smooth transition to increase hydraulic efficiency of an inlet structure.

Time of Concentration: The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point under consideration. It is usually associated with the design storm (see Inlet Time).

Total Storage: The volume of reservoir below the maximum controllable level, including dead storage (Thomas and Harbeck, 1956, p. 13).

Trunk (or Trunk Line): In a roadway drainage system, the main conduit for transporting the storm waters. This main line is generally quite deep in the ground so that laterals coming from fairly long distances can drain by gravity into the trunk line.

Turbulent Flow: That type of flow in which any particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the square of the velocity.

Unit Hydrograph: The hydrograph of direct runoff from a storm uniformly distributed over the drainage basin during a specified unit of time; the hydrograph is reduced in vertical scale to correspond to a volume of runoff of one inch from the drainage basin (after American Society of Civil Engineer, 1949, p. 105). The hydrograph of surface runoff (not including groundwater runoff) on a given basin due to an effective rain falling for a unit of time (Sherman, 1949, p. 514) (also see Hoyt and others, 1936, p. 124).

Velocity Head: A term used in hydraulics to represent the kinetic energy of flowing water. This "head" is represented by a column of standing water equivalent in potential energy to the kinetic energy of the moving water calculated as V2/2g, where "V" represents velocity in feet per second and "g" represents potential acceleration due to gravity in feet per second per second.

Watershed: The area drained by a stream or stream system.

Water Year: In Geological Survey reports dealing with surface water supply, the 12-month period, October 1 through September 30. The water year is designated by the calendar year in which it ends and which includes nine of the 12 months. Thus, the year ended September 30, 1959, is called the "1959 water year."

SURFACE DRAINAGE STUDIES

Input - Output Data & Design Aids

	INPUT Data	SOURCE
	Drainage Area	USGS Quadrangles, Aerial Photo or other sources
	Land Use	" " "
Watershed	Watershed Steepness	" " "
Information	Soils, Types	" " "
	Covers	" " "
Climate	Rainfall Intensity	Weather Charts
Information	Storm Frequency	Design Criteria
Limiting Design	Allowable High Water	Local Records
Factors	Gradeline Control	Design Criteria
	Description of Existing	Records
Existing	Str. & O. Section	
Facilities	High Water of Exist.	Local Records
	Structure	
	OUTPUT Data	SOURCE
Design	**Design**	**Design computations applying the**
Discharge	Discharge	input to pertinent charts, etc.
Proposed	Type	Design Criteria
Facilities	Size	Design Computations
Drainage Easements	Size	R/W Manual
	Location	Design Computations
Cost	Cost, including	Design Computations
	Channel changes and	
	other related items	
	DESIGN AIDS Data	SOURCE (FILES)
	Rational Method	**Facilities Development Manual**
Estimating	N.R.C.S. Methods - TR55 *	" " "
Run-Off	USGS Flood Frequency	" " "
	Equations for Wisconsin	" " "
	Gaging Station	U.S.G.S
	Published Watershed Studies	Regional Planning Agencies, U.S. C.O.E., U.S. N.R.C.S., U.S.G.S., etc.
	Culvert Capacity	FHWA Hydraulic Engineering Circular
	Inlet Control	" " "
Structure	Outlet Control	" " "
Design	Critical Depth	" " "
	Headwater Depth	" " "

* NRCS is Natural Resources Conservation Service, the new name for the Soil Conservation Service (SCS)

COMPUTER REFERENCES

Public Domain Software	TR55		P.C. program that mirror procedures in Urban Hydrology for Small Watershed Technical Release 55, June 1986
	Hydrain * HYDRA		Storm/sanitary sewers
		WSPRO	Step backwater and bridge hydraulics
		HYDRO	IDF curves, hyetographs, peak flows
		HYCLV	Culvert analysis and Design
HEC-15			HYCHL Lining stability - based on HEC-11 and
on			HY8 Culvert system performance - based HDS-5, HEC-14, HEC-19
		NFF	U.S.G.S. National Flood Frequency Model
		HYEQT	Analyze user supplied Equations
* This software package is available from Mctrans which is at the University of Florida.			
Telephone: 1-(352)-392-0378 email: uftrc@ce.ufl.edu web site: http://mctrans.ce.ufl.edu/			

Alternate ---------------------- **Schedule No.** ---------------------- **District No.** ----------------------

Project No. ---------------------- **County** ---------------------- **Designer** ----------------------

Name of Road ---------------------- **Hwy.** ----------------------

Design Frequency ---------------------- **Date** ----------------------

Major Drainage Summary Sheet

Input						**Output**			**Remarks**
Sat. or Loc.	Drainage Area (Acres)	Chief Land Use or Cover	Description of Terrain	Head-Water Allow-able	Existing Facility Size & Type	Design Dis-charge	Proposed Facility Size & Type	Cost	Remarks Special Limita-tion, Channel Changes

Sample Stormwater-Drainage-WQ Report Spreadsheet: Drainage-Summary Worksheet
Download a zipped working copy of the spreadsheets at:
http://wisconsindot.gov/rdwy/fdm/files/WisDOT-Stormwater-Drainage-WQ-Channel-Spreadsheets.zip

9/12/2012

Basic Project Information

Project ID: XXXX-XX-XX
Title: Example Project
Designer/Checker:
DOT Region/Firm Name:
Date:

HIGHWAY:	
LIMITS:	
COUNTY:	
DESCRIPTION OF WORK:	
PROJECT MANAGER:	
PS&E DATE:	
DESIGN STAGE	☐ Planning ☐ 30% ☐ 60% ☐ 90% ☐ Final

Drainage Summary
IS THERE A SIGNIFICANT FLOW INCREASE OR DECREASE WITHIN ANY SUB BASIN OF THE PROJECT? IF YES, DESCRIBE THE CAUSE OF THE CHANGE AND WHY IT IS NECESSARY.
IS THERE A SIGNIFICANT IMPERVIOUS AREA CHANGE TO ANY SUB BASIN OF THE PROJECT? IF YES, DESCRIBE THE CAUSE OF THE CHANGE AND WHY IT IS NECESSARY.
HAVE THE DRAINAGE SUB BASIN AREAS OR FLOW PATHS CHANGED SIGNIFICANTLY? IF YES, DESCRIBE THE CAUSE OF THE CHANGE AND WHY IT IS NECESSARY.
DESCRIBE THE PROPOSED DRAINAGE CONVEYANCE AND CONTROL SYSTEMS FOR THE PROJECT.
DESCRIBE THE AQUATIC ORGANISM PASSAGE ISSUES FOR THE PROJECT, IF ANY.
IF THE DESIGN DOES NOT MEET THE DOT FDM CHAPTER 13 DRAINAGE REQUIREMENTS, EXPLAIN HOW AND WHY.
DESCRIBE WDNR COORDINATION. PROVIDE NAME OF WDNR CONTACT AND DATE, AND ATTACH ANY CORRESPONDENCE.
IF THE DRAINAGE DESIGN MEETS LOCAL, MUNICIPAL OR REGIONAL GUIDELINES THAT EXCEED FDM CHAPTER 13 DRAINAGE REQUIREMENTS, EXPLAIN HOW AND WHY.
IF A SIGNIFICANT IMPACT TO THE PROJECT OCCURS DUE TO DRAINAGE, PROJECT MANAGER CONCURRENCE IS REQUIRED. (PM SIGN AND DATE)

Page 1

Sample Stormwater-Drainage-WQ Report Spreadsheet: Data Worksheet
(Use link on FDM 13-1 Attachment 10.1 to download a zipped working copy of the spreadsheets.

1	**Drainage Data**
2	**Project ID: XXXX-XX-XX**
3	**Title: Example Project**
4	**Designer/Checker:**
5	**DOT Region/Firm Name:**
6	**Date:**

7	**OUTFALL INFORMATION**						
8	Outfall number	**1**	**2**	**3**	**4**	**5**	**6**
9	Outfall discharges to:						
10	Waterway crossing type	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
11	If discharging to environmentally sensitive area, what kinds of buffers were used at outfall?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
12	Previous flooding issues or flow restrictions?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
13	Is the drainageway in the DOT ROW a navigable waterway?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
14	Classify the drainageway in the DOT ROW	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu

15	**BASIC SUB BASIN DRAINAGE INFORMATION**						
16	Outfall number	**1**	**2**	**3**	**4**	**5**	**6**
17	Stormwater conveyance type	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
18	Outfall station						
19	Subbasin starting station						
20	Subbasin ending station						
21	Proposed roadway length (ft)	0	0	0	0	0	0
22	Flow conveyance change						
23	Flood design frequency (yrs)						
24	Check design frequency (yrs)						
25	Is the check design storm safely passed?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
26	DOT right-of-way area (acres)						
27	Subbasin drainage area (acres)						
28	DOT right-of-way compared to subbasin drainage area (%)	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
29	DOT impervious area - existing (acres)						
30	DOT impervious area - proposed (acres)						
31	Change in impervious area (acres)	0	0	0	0	0	0
32	Percent change in DOT impervious area	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
33	Design software used						
34	Method used to estimate peak flows						
35	*Complete lines 36-46 for culverts only*						
36	Existing peak flow (cfs)						
37	Proposed peak flow (cfs) (before detention)						
38	Proposed peak flow (cfs) (after detention/in-line storage/other)						
39	Change in peak flow (cfs)	0	0	0	0	0	0
40	Percent change in peak flow	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
41	Existing 2-yr peak flow (cfs)						
42	Proposed 2-yr peak flow (cfs) (before detention)						
43	Proposed 2-yr peak flow (cfs) (after detention/in-line storage/other)						
44	Change in 2-yr peak flow (cfs)	0	0	0	0	0	0
45	Percent change in 2-yr peak flow	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
46	Existing Tc (min)						
47	Proposed Tc (min)						
48	C or CN (existing)						
49	C or CN (proposed)						
50	Rainfall intensity (in/hr) (rational method only)						
51	Rainfall depth used for design storm, if applicable (in)						

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52	**CULVERT DESIGN**						
53	**Existing Culvert**						
54	Outfall number	**1**	**2**	**3**	**4**	**5**	**6**
55	Culvert present? (Yes or No)	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
56	Existing culvert shape	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
57	Existing culvert material	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
58	Existing culvert size (ft)						
59	Existing number of culverts						
60	Existing Manning's n						
61	Inlet entrance type	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
62	Inlet loss coefficient (Ke)						
63	Upstream invert (ft)						
64	Downstream invert (ft)						
65	Length (ft)						
66	Slope (%)	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
67	**Floodplain Management**						
68	Is culvert in a mapped floodplain?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
69	Will proposed culvert increase water surface profile?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
70	**Drainage District Issues**						
71	Is culvert in a drainage district?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
72	Drainage District Name						
73	Will proposed culvert raise the culvert invert or increase water surface profile?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
74	Has drainage board approved increases?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
75	**Aquatic Organism Passage**						
76	Is aquatic organism passage a concern?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
77	Does WDNR agree with AOP design?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
78	**Proposed Culvert Design**						
79	Design ADT						
80	Design flow						
81	Design year frequency						
82	Hydrological method used						
83	Assumed tailwater condition						
84	Maximum allowable headwater						
85	Maximum allowable headwater design criteria	DDMenu	DDMenu	DDMenu	DDMenu	DDMenu	DDMenu
86	Proposed culvert shape	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
87	Proposed culvert material	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
88	Proposed culvert size						
89	Proposed number of culverts						
90	Manning's n						
91	Type of endwalls	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
92	Inlet loss coefficient (Ke)						
93	Proposed upstream invert (ft)						
94	Proposed downstream invert (ft)						
95	Proposed length (ft)						
96	Proposed slope (%)	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
97	Embedment depth (ft)						
98	Embedment material						
99	Discharge velocity (ft/s)						
100	Riprap outfall (Size riprap or None)						
101	Station of lowest subgrade shoulder point in subbasin (0+00)						
102	Elevation of lowest subgrade shoulder point in subbasin (ft)						
103	Headwater distance below subgrade shoulder point (ft)						
104	Headwater to pipe diameter ratio						
105	Design software used						
106	Proposed tailwater condition						
107	Discharge pipe end submerged?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
108	Assumed tailwater elevation (ft)						

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109	**CULVERT LINER DESIGN**						
110	**Existing Culvert**						
111	Outfall number	**1**	**2**	**3**	**4**	**5**	**6**
112	Does WDNR agree with use the of a liner?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
113	Existing culvert size (ft)						
114	Pipe material						
115	Pipe condition						
116	Any collapse?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
117	Any deflection?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
118	Are ends crushed?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
119	*n* value existing pipe						
120	Pipe geometry (i.e. circular)	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
121	Pipe inlet invert elevation (ft)						
122	Pipe outlet invert elevation (ft)						
123	Length (ft)						
124	Slope (%)	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
125	Depth of cover over culvert (ft)						
126	Is overtopping an issue?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
127	Are there hydraulically sensitive structures or property up-stream that could be flooded if water surface profile is increased?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
128	If existing culvert diameter > 48", hydraulic design is required. Has this been done?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
129	Are any of the culverts greater than 48" in diameter?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
130	Field verify dimension?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
131	**Floodplain Management**						
132	Is culvert in a mapped floodplain?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
133	Will proposed liner increase water surface profile?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
134	**Drainage District Issues**						
135	Is culvert in a drainage district?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
136	Drainage District Name						
137	Has drainage board approved use of a liner?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
138	**Aquatic Organism Passage**						
139	Is aquatic organism passage a concern?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu
140	Does WDNR agree with AOP design?	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu	DD Menu

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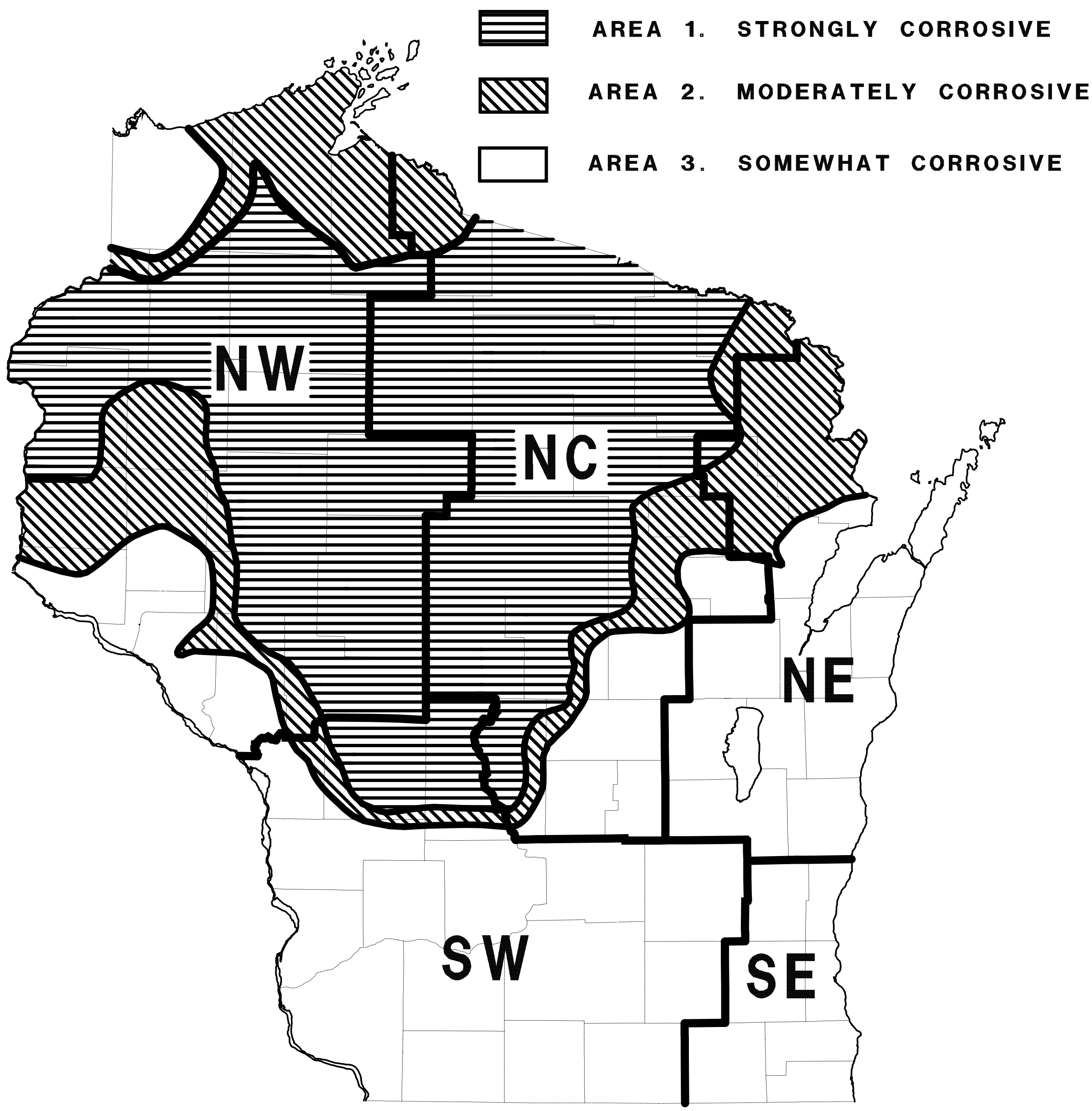
POTENTIAL FOR BACTERIAL CORROSION OF ZINC GALVANIZED STEEL CULVERT PIPE



INDIVIDUAL SITES IN AREA 3 MAY BE STRONGLY TO MODERATELY CORROSIVE DUE TO LOCAL CONDITIONS SUCH AS FARM RUNOFF, ANAEROBIC BACTERIA IN THE SOIL, ETC.

STORM SEWER - FILL HEIGHT TABLE FOR CONCRETE PIPE

Type/Class of Pipe	AASHTO Materials Designation	Pipe Size I.D. (inches)	Maximum Height of Cover Over Top of Pipe (feet)
Reinforced Concrete Class II	M 170	12- 108	11
Reinforced Concrete Class III	M 170	12- 108	15
Reinforced Concrete Class IV	M 170	12- 84	25
Reinforced Concrete Class V	M 170	12- 72	35

Surface Loadings

The minimum concrete pipe class required based on depth to subgrade is as follows:

Depth of Subgrade Cover (feet)	0 to 2	2 to 3	3 to 6
Minimum Class of Concrete Pipe Required	IV	III	II

The desired minimum cover is 2 feet below subgrade. Where less than two feet of cover is provided special measures may be required during construction to minimize equipment loading impacts on the pipe. At a minimum, locations with reduced subgrade cover should be identified on the plans so that the contractor can take precautionary measures.

Design Criteria

The above table refers to Class C bedding using sand/gravel backfill weighing 120 lb/ft^3 with zero projecting embankment condition and trench widths as specified Standard Spec 608.

FILL HEIGHT TABLE 1

Corrugated Steel, Aluminum, Polyethylene, Polypropylene and Reinforced Concrete Pipe

HS20 Loading 2" x 2/3" Corrugations - Standard Specification Bedding Unless Otherwise Noted

		Height of Cover Over Top Pipe in Feet - "H"																	
		Min. to 15' **(2)**			16' to 20'			21' to 25'			26' to 30'			31' to 35'			36' to 40'		
Dia. In. **(5)**	Area S.F.	Thickness		RCCP Class Pipe	Thickness		RCCP Class Pipe	Thickness		RCCP Class Pipe	Thickness		RCCP Class Pipe	Thickness		RCCP Class Pipe	Thickness		RCCP Class Pipe
		Steel	Alum		Steel	Alum		Steel	Alum		Steel	Alum		Steel	Alum		Steel	Alum	
12 *	0.8	0.064	0.060	III	0.064	0.060	IV	0.064	0.060	IV	0.064	0.060	V	0.064	0.060	V	0.064	0.075	V **(4)**
15 *	1.2	0.064	0.060	III	0.064	0.060	IV	0.064	0.060	IV	0.064	0.060	V	0.064	0.075	V	0.064	0.105	V **(4)**
18 *	1.8	0.064	0.060	III	0.064	0.060	IV	0.064	0.060	IV	0.064	0.075	V	0.064	0.105	V	0.064	0.135	V **(4)**
21 *	2.4	0.064	0.060	III	0.064	0.060	IV	0.064	0.075	IV	0.064	0.105	V	0.064	0.135	V	0.079	X	V **(4)**
24 *	3.1	0.064	0.075	III	0.064	0.075	IV	0.079	0.075	IV	0.079	0.105	V	0.079	0.164	V	0.079	X	V **(4)**
30 *	4.9	0.079	0.075	III	0.079	0.075	IV	0.079	0.105	IV	0.079	0.135	V	0.109	X	V	0.109	X	V **(4)**
36 *	7.1	0.079	0.105	III	0.079	0.105	IV	0.109	0.135	IV	0.109	0.164	V	0.138	X	V	0.138	X	V **(4)**
42	9.6	0.109	0.105	III	0.109	0.135	IV	0.109	0.164	IV	0.138	0.164	V	0.138	X	V	0.168	X	V **(4)**
48	12.6	0.109	0.105	III	0.109	0.135	IV	0.138	0.164	IV	0.168	X	V	0.168	X	V	0.138 E	X	V **(4)**
54	15.9	0.109	0.105	III	0.138	0.135	IV	0.168	0.164	IV	0.168	X	V	0.109 E	X	V	0.138 E	X	V **(4)**
60	19.6	0.138	0.164	III	0.138	X	IV	0.168	X	IV	0.138 E	X	V	0.138 E	X	V	0.168 E	X	V **(4)**
66	23.8	0.138	0.164	III	0.168	X	IV	0.168	X	IV	0.138 E	X	V	0.138 E	X	V	0.168 E	X	V **(4)**
72	28.3	0.138**(3)**	0.164	III	0.168	X	IV	0.168	X	IV	0.138 E	X	V						
78	33.2	0.168	X	III	0.168	X	IV	0.168 E	X	IV	**(1)**								
84	38.5	0.168	X	III	0.168	X	IV												

E = Elongated, Vertical Axis 5% greater than Horizontal.
(1) Any pipe under the heavy line will require a special design.
(2) 12" minimum cover, top of pipe to top of subgrade for steel, aluminum and concrete. 24" required minimum cover for Class IIIA and IIIB pipe under Standard Spec 520 or 608, or as polyethylene and polypropylene pipe under Standard Spec 530.
(3) A thickness of 0.138" may be used for fill heights of minimum to 10 Ft. a thickness of 0.168" may be used for fill heights of greater than 10 Ft. but less than 26 feet.
(4) Class "B" Bedding required.
NOTE: For steel and aluminum pipe in the shaded portion of the table (>60 in. dia.), a corrugation size of 3" by 1" is generally more economical than 2 2/3" by 1/2". See Tables 2 and 7.

X = Do not use
(5) For corrugated steel pipe in a 6", 8", or 10" diameter, the minimum thickness is 0.052" and 0.064" respectively.
For corrugated aluminum pipe in 6", 8" or 10" diameter, the minimum thickness is 0.048", 0.048" and 0.06" respectively.
* Corrugated polyethylene and corrugated polypropylene pipe in these diameters are available for use under the Class III-A and Class III-B bid items as specified in FDM 13-1-15 and FDM 13-1-17. Minimum fill height shall be 24 inches and maximum fill height shall be 11 feet for polyethylene (Class III-A) and 15 feet for polypropylene (Class III-B). It is not necessary to specify thickness for polyethylene or polypropylene pipe.

FILL HEIGHT TABLE 2 (1)
Corrugated Steel Pipe - 3" x 1" Corrugations - H20 Live Load

Pipe Dia. In.	Waterway Area Sq. Ft.	Min. Cover In. (3)	Maximum Height of Fill - Ft.				
			Metal Thickness in Inches (2)				
			0.064	0.079	0.109	0.138	0.168
60	19.6	12	24	30	44	53	58
66	23.8	12	22	27	40	48	53
72	28.3	12	20	25	37	44	49
78	33.2	12	18	23	34	40	45
84	38.5	12	17	22	32	37	42
90	44.2	12	16	20	29	35	39
96	50.3	12	X	19	28	33	37
102	56.7	24	X	17	26	31	34
108	63.6	24	X	X	24	29	32
114	70.9	24	X	X	23	27	31
120	78.5	24	X	X	X	26	29

FILL HEIGHT TABLE 3 (1)
Structural Plate Pipe 6" x 2" Corrugations - H20 Live Load

Pipe Dia. In.	Waterway Area Sq. Ft.	Min. Cover In. (3)	Maximum Height of Fill - Ft. (4)						
			Metal Thickness in Inches (2)						
			0.10	0.138	0.168	0.188	0.218	0.249	0.280
60	19.6	12	35	51	67	77	87 (93)	96 (110)	106 (120)
72	28.3	12	29	43	54	57 (64)	62 (77)	67 (91)	73 (100)
84	38.5	12	25	36	44	46 (55)	49 (66)	53 (78)	56 (85)
96	50.3	12	22	32	39	40 (48)	42 (58)	44 (68)	47 (75)
102	56.7	24	20	30	37	38	40 (55)	42 (64)	43 (70)
108	63.6	24	19	28	35	36	38 (52)	39 (61)	41 (66)
120	78.5	24	17	25	33	34	35 (46)	36 (55)	37 (60)
132	95.0	24	16	23	30	32	33 (42)	34 (50)	35 (54)
144	113.1	24	14	21	28	31	32	32 (45)	33 (50)
156	132.7	24	13	19	25	29	31	31 (42)	32 (46)
168	153.9	24	12	18	24	27	30	31	31 (42)
180	176.7	24	11	17	22	25	30	30	30 (40)

(1) Table 2 is valid for 5” x 1” corrugations which may be used in lieu of 3" x 1" corrugations for fill heights to 30 feet.
(2) The steel thicknesses shown are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, greater thicknesses should be specified.
(3) Minimum cover top of pipe to top of subgrade.
(4) Maximum fill heights shown in parentheses are permitted if the pipe is elongated - Vertical axis 5% greater than the horizontal axis.
NOTE: Corrugated steel pipe (CSCP) is normally more economical to use than structural plate pipe (SPP) for installations where either type will satisfy fill height requirements. The potential cost savings of the CSCP is possible because CSCP is factory assembled into transportable lengths whereas SPP must be field assembled from plates.

FILL HEIGHT TABLE 4
Corrugated Steel Pipe Arch - 2" x 1/2" Corrugations - H20 Live Load

Size: Span x Rise (Inches)	Min. Thickness In. (1)	Min. Cover In. (2)	Max. Height of Fill Ft. (3)	Waterway Area Sq. Ft.	Round Pipe of Equal Periphery	
					Waterway Area Sq. Ft.	Dia. Inches
17 X 13	0.064	18	13	1.1	1.23	15
21 x 15	0.064	18	12	1.6	1.77	18
24 x 18	0.064	18	10	2.2	2.41	21
28 x 20	0.064	18	9	2.8	3.14	24
35 x 24	0.079	18	9	4.4	4.91	30
42 x 29	0.079	18	7	6.4	7.07	36
49 x 33	0.109	18	7	8.7	9.62	42
57 x 38	0.109	18	7	11.4	12.57	48
64 x 43	0.109	18	7	14.3	15.90	54
71 x 47	0.138	18	7	17.6	19.64	60
77 x 52	0.168	18	7	21.3	23.76	66
83 x 57	0.168	18	8	25.3	28.27	72

FILL HEIGHT TABLE 5
Corrugated Steel Pipe Arch (4) - 3" x 1" Corrugations - H20 Live Load

Size: Span x Rise (Inches)	Min. Thickness In. (1)	Min. Cover In. (2)	Max. Height of Fill Ft. (3)	Waterway Area Sq. Ft.	Round Pipe of Equal Periphery	
					Waterway Area Sq. Ft.	Dia. Inches
40 x 31	0.064	18	12	6.4	7.07	36
46 x 36	0.064	18	12	8.7	9.62	42
53 x 41	0.064	18	12	11.4	12.57	48
60 x 46	0.064	18	12	14.3	15.90	54
66 x 51	0.064	18	12	17.6	19.64	60
73 x 55	0.064	18	15	22.0	23.76	66
81 x 59	0.079	18	15	26.0	28.27	72
87 x 63	0.079	18	14	31.0	33.18	78
95 x 67	0.109	18	12	35.0	38.48	84
103 x 71	0.109	24	11	40.0	44.18	90
112 x 75	0.109	24	10	46.0	50.27	96
117 x 79	0.109	24	10	52.0	56.74	102
128 x 83	0.138	24	9	58.0	63.62	108

(1) The steel thicknesses shown are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, greater thicknesses should be specified.
(2) Minimum cover top of pipe to top of subgrade.
(3) Allowable fill heights are computed on the basis that corner bearing pressure will not exceed two tons per square foot.
(4) Table 5 is also valid for the metric 125 mm x 25 mm corrugation which may be used in lieu of the 3" x 1" corrugations.

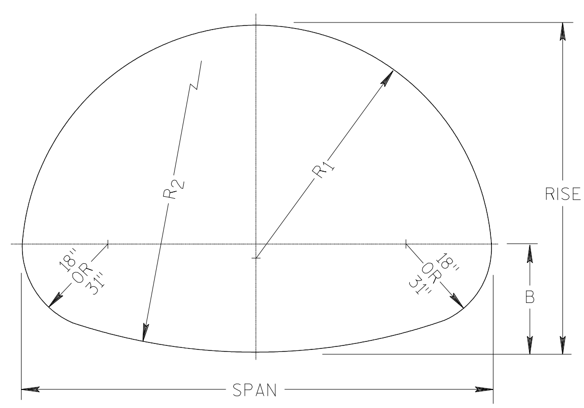
Fill Height Table 6
Structural Plate Pipe Arch - 6" x 2" Corrugations - H20 Live Load

<table>
<tr><th rowspan="2">Bid Item Number</th><th>Size</th><th rowspan="2">Waterway Area Sq. Ft.</th><th rowspan="2">Min. Thickness Inches (1)</th><th rowspan="2">Min. Cover Inches (2)</th><th rowspan="2">Max. Height of Fill Ft. (3)</th><th rowspan="2">Corner Radius Inches</th><th colspan="3">Lay out Dimensions</th></tr>
<tr><th>Span x Rise (Ft. - Inches)</th><th>B Inches</th><th>R1 Feet</th><th>R2 Feet</th></tr>
<tr><td>527.0305</td><td>6-1 x 4-7</td><td>22</td><td rowspan="10">0.109</td><td rowspan="2">18</td><td>15</td><td rowspan="8">18</td><td>21.0</td><td>3.07</td><td>6.36</td></tr>
<tr><td>527.0310</td><td>7-0 x 5-1</td><td>28</td><td>15</td><td>21.4</td><td>3.53</td><td>8.68</td></tr>
<tr><td>527.0315</td><td>8-2 x 5-9</td><td>38</td><td rowspan="6">24</td><td>12</td><td>20.9</td><td>4.08</td><td>15.24</td></tr>
<tr><td>527.0320</td><td>8-10 x 6-1</td><td>43</td><td>11</td><td>21.8</td><td>4.24</td><td>14.89</td></tr>
<tr><td>527.0325</td><td>9-9 x 6-7</td><td>52</td><td>10</td><td>21.9</td><td>4.86</td><td>18.98</td></tr>
<tr><td>527.0330</td><td>11-5 x 7-3</td><td>64</td><td>8</td><td>27.4</td><td>5.78</td><td>13.16</td></tr>
<tr><td>527.0335</td><td>11-10 x 7-7</td><td>71</td><td>7</td><td>25.2</td><td>5.93</td><td>18.03</td></tr>
<tr><td>527.0340</td><td>12-10 x 8-4</td><td>85</td><td>6</td><td>24.0</td><td>6.44</td><td>26.23</td></tr>
<tr><td>SPV.0090</td><td>13-3 x 9-4</td><td>97</td><td rowspan="9">36</td><td>13</td><td rowspan="9">31</td><td>38.5</td><td>6.68</td><td>16.05</td></tr>
<tr><td>SPV.0090</td><td>14-2 x 9-10</td><td>109</td><td>12</td><td>38.8</td><td>7.13</td><td>18.55</td></tr>
<tr><td>SPV.0090</td><td>15-4 x 10-4</td><td>123</td><td rowspan="3">0.138</td><td>11</td><td>41.8</td><td>7.76</td><td>17.38</td></tr>
<tr><td>SPV.0090</td><td>16-3 x 10-10</td><td>137</td><td>10</td><td>42.1</td><td>8.21</td><td>19.67</td></tr>
<tr><td>SPV.0090</td><td>17-2 x 11-4</td><td>151</td><td>10</td><td>42.3</td><td>8.65</td><td>22.23</td></tr>
<tr><td>SPV.0090</td><td>18-1 x 11-10</td><td>167</td><td rowspan="3">0.168</td><td>9</td><td>42.4</td><td>9.09</td><td>24.98</td></tr>
<tr><td>SPV.0090</td><td>19-3 x 12-4</td><td>182</td><td>8</td><td>45.9</td><td>9.75</td><td>23.22</td></tr>
<tr><td>SPV.0090</td><td>19-11 x 12-10</td><td>200</td><td>7</td><td>42.5</td><td>9.98</td><td>31.19</td></tr>
<tr><td>SPV.0090</td><td>20-7 x 13-2</td><td>211</td><td>0.188</td><td>6</td><td>43.7</td><td>10.33</td><td>31.13</td></tr>
</table>

(1) The metal thickness shown are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, greater thicknesses should be specified at least for the bottom plates.

(2) Minimum cover top of pipe to top of subgrade.

(3) Allowable fill heights are computed on the basis that corner bearing pressure will not exceed two tons per square foot.



LAYOUT DIMENSIONS

FILL HEIGHT TABLE 7
Corrugated Aluminum Pipe 3" x 1" Corrugations - H20 Live Load

Pipe Dia. In.	Waterway Area Sq. Ft.	Min. Cover In. (2)	Maximum Height of Fill - Ft. Metal Thickness in Inches (1)				
			0.060	0.075	0.10	0.13	0.164
60	19.6	12	12	17	23	31	32
66	23.8	12	13	16	21	31	31
72	28.3	12	12	14	19	30	30
78	33.2	18	X	13	18	30	30
84	38.5	18	X	X	17	29	30
90	44.2	18	X	X	16	29	29
96	50.3	18	X	X	16	29	29
102	56.7	18	X	X	X	27	29
108	63.6	18	X	X	X	25	28
114	70.9	18	X	X	X	X	28
120	78.5	18	X	X	X	X	28

FILL HEIGHT TABLE 8
Aluminum Alloy, Structural Plate Pipe 9" x 2 1/2" Corrugations - H20 Live Load

Pipe Dia. In.	Waterway Area Sq. Ft.	Minimum Cover In. (2)	Maximum Height of Fill - Ft. Metal Thickness in Inches (1)						
			0.10	0.12	0.15	0.17	0.20	0.22	0.250
60	19.6	15	22	29	37	44	55	59	61
72	28.3	21	18	24	31	37	44	46	48
84	38.5	21	15	21	26	31	37	39	40
96	50.3	24	14	19	23	28	35	35	36
102	56.7	24	13	17	22	26	34	34	35
108	63.6	27	12	16	21	24	33	33	34
120	78.5	27	11	14	19	22	31	32	32
132	95.0	30	X	13	17	20	28	31	31
144	113.1	30	X	12	15	18	25	29	30
156	132.7	30	X	11	14	17	24	27	30
168	153.9	30	X	X	13	16	22	25	28
180	176.7	30	X	X	X	15	20	23	26

Note: X = Do not use - design strengths exceeded.

(1) The metal thicknesses shown are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, greater thickness should be specified.

(2) Minimum cover top of pipe to top of subgrade.

FILL HEIGHT TABLE 9
Corrugated Aluminum Pipe Arch, 2 - 2/3" X 1/2" Corrugations - H20 Live Load

Size					Round Pipe of Equal Periphery	
Span x Rise Inches	Min. Thickness In. (1)	Min. Cover In. (2)	Max. Height of Fill ft. (3)	Waterway Area Sq. Ft.	Waterway Area Sq. Ft.	Dia. Inches
17 x 13	0.060	18	12	1.1	1.23	15
21 x 15	0.060		10	1.6	1.77	18
24 x 18	0.060		8	2.2	2.41	21
28 x 20	0.075		7	2.8	3.14	24
35 x 24	0.075		6	4.4	4.91	30
42 x 29	0.105		6	6.4	7.07	36
49 x 33	0.105		5	8.7	9.62	42
57 x 38	0.135		6	11.4	12.57	48
64 x 43	0.135		6	14.3	15.90	54
71 x 47	0.164		7	17.6	19.64	60

(1) The metal thicknesses shown are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, greater thicknesses should be specified.

(2) Minimum cover top of pipe to top of subgrade.

(3) Allowable fill heights are computed on the basis that corner bearing pressure will not exceed two tons per square foot.

FILL HEIGHT TABLE 10

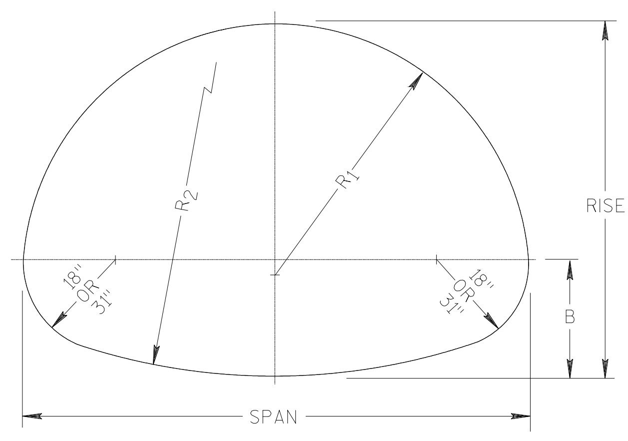
Aluminum Alloy Structural Plate Pipe Arch - 9" X 2 1/2" Corrugations - H20 Live Load

Size Span x Rise Ft-In	Waterway Area Sq. Ft.	Min. Thickness, In. (1)	Min. Cover (2)	Max. Height of Fill (3)	Corner Radius	Layout Dimensions B Inches	Layout Dimensions R_1 Feet	Layout Dimensions R_2 Feet
6-2 x 5-0	25	0.100	24 Inches	18	27 Inches	27.2	3.25	24.93
6-7 x 5-8	30			16	31.8 Inches	32.5	3.46	5.82
8-1 x 6-1	39			13		33.5	4.44	9.00
8-10 x 6-4	44		30 Inches	11		35.6	5.27	7.75
9-11 x 6-8	53			10		34.2	5.53	15.72
11-5 x 7-1	64			9		35.3	6.51	18.50
12-3 x 7-3	70		36 Inches	8		38.4	7.57	13.77
13-1 x 8-4	87			8		42.0	7.40	11.97
14-0 x 8-7	94	0.125		10		39.4	7.52	17.92
14-8 x 9-8	110	0.125		10		44.0	7.57	13.85
15-7 x 10-2	123	0.150		10		44.4	8.03	15.80
16-9 x 10-8	137	0.150		10		47.9	8.75	15.52
17-9 x 11-2	152	0.175		9		48.2	9.20	17.40
18-8 x 11-8	167	0.175		8		48.5	9.65	19.44
19-10 x 12-1	183	0.225		8		52.3	10.39	18.97
20-10 x 12-7	200	0.250		8		52.5	10.83	20.93
21-6 x 12-11	211	0.250		7		53.9	11.23	21.43

(1) The metal thicknesses shown are adequate for structural requirements only. Where corrosive and/or abrasive conditions exist, greater thicknesses should be specified at least for the bottom plates.

(2) Minimum cover top of pipe to top of subgrade.

(3) Allowable fill heights are computed on the basis that corner bearing pressure will not exceed two tons per square foot.



LAYOUT DIMENSIONS

Dimensions for Reinforced Concrete Arch and Elliptical Pipe

	Arch			Vertical Elliptical			Horizontal Elliptical		
Equivalent Round Size (Inches)	**Rise x Span (Inches)**	**Waterway Area (Sq. Ft).**	**Minimum Wall Thickness (Inches)**	**Rise x Span (Inches)**	**Waterway Area (Sq. Ft.)**	**Minimum Wall Thickness (Inches)**	**Rise x Span (Inches)**	**Waterway Area (Sq. Ft.)**	**Minimum Wall Thickness (Inches)**
15	11 x 18	1.1	2.25						
18	13 x 22	1.6	2.5				14 x 23	1.8	2.75
21	15 x 26	2.2	2.75						
24	18 x 28	2.8	3.0				19 x 30	3.3	3.25
27							22 x 34	4.1	3.5
30	22 x 36	4.4	3.5				24 x 38	5.1	3.75
33							27 x 42	6.3	3.75
36	27 x 44	6.4	4.0	45 x 29	7.4	4.5	29 x 45	7.4	4.5
39				49 x 32	8.8	4.75	32 x 49	8.8	4.75
42	31 x 51	8.8	4.5	53 x 34	10.2	5.0	34 x 53	10.2	5.0
48	36 x 58	11.4	5.0	60 x 38	12.9	5.5	38 x 60	12.9	5.5
54	40 x 65	14.3	5.5	68 x 43	16.6	6.0	43 x 68	16.6	6.0
60	45 x 73	17.7	6.0	76 x 48	20.5	6.5	48 x 76	20.5	6.5
66				83 x 53	24.8	7.0	53 x 83	24.8	7.0
72	54 x 88	25.6	7.0	91 x 58	29.5	7.5	58 x 91	29.5	7.5
78				98 x 63	34.6	8.0	63 x 98	34.6	8.0
84	62 x 102	34.6	8.0	106 x 68	40.1	8.5	68 x 106	40.1	8.5
90	72 x 115	44.5	8.5	113 x 72	46.1	9.0	72 x 113	46.1	9.0
96	77 x 122	51.7	9.0	121 x 77	52.4	9.5	77 x 121	52.4	9.5
102				128 x 82	59.2	9.75	82 x 128	59.2	9.75
108	87 x 138	66.0	10.0	136 x 87	66.4	10.0	87 x 136	66.4	10.0
114				143 x 92	74.0	10.5	92 x 143	74.0	10.5
120	97 x 154	81.8	11.0	151 x 97	82.0	11.0	97 x 151	82.0	11.0
132	106 x 169	99.1	10.0	166 x 106	99.2	12.0	106 x 166	99.2	12.0
144				180 x 116	118.6	13.0	116 x 180	118.6	13.0

Fill Height Table 11

Reinforced Concrete Arch and Elliptical Pipe (All Sizes)

Type of Pipe	**Maximum Height of Fill - Ft.**			
	Class of Pipe (0.01" Crack D-Load)			
	Class A-III Class VE-III Class HE-III (1350 D)	**Class A-IV Class VE-IV Class HE-IV (2000 D)**	**Class VE-V (3000 D)**	*Class VE-VI* **(4000 D)**
Arch	15	25		
Vertical Elliptical	15	25	35	45
Horizontal Elliptical	15	25		

NOTES:

(1) Minimum cover excluding pavement shall be 1 ft.

(2) Fill Heights were computed assuming Class "C" bedding. If Class "B" bedding is used, increase maximum height of fill by 20%.

Materials shall conform to AASHTO designation M206 for reinforced concrete arch pipe and AASHTO designation M207 for reinforced concrete elliptical pipe. Requires special provision. Use SPV.0090 Bid Item.

Guidelines for Determining a Rural Area

The following is meant to assist in the defining a "rural area" for the purposes of "in-kind" culvert replacement. This guidance is not all inclusive. Good engineering judgement should be employed in determining rural versus urban or urbanizing areas of a project.

A Rural Area is:

A project area that is not within a defined municipal boundary, or an area where the population density averages 1000 or more persons per square mile of urban area.

- The population density must correlate to the project area. If the project area covers only part of a populated area or municipal boundary, only those culverts within those areas require full H&H analysis.
- For annually revised population estimates, refer to the Wis. Department of Administration, Division of Inter-Governmental Relation's Website at: https://doa.wi.gov/demographics and reference the applicable population or population estimates. Other population projections may be obtained from the applicable Regional Planning Commission.

An area of the project in which the adjacent land is not used for commercial or industrial land uses.

- This includes a variety of commercial land uses such as strip commercial, office parks, shopping centers and downtown commercial.
- This classification also includes governmental, institutional, transportation and recreational uses that contain source areas (such as parking lots, streets, storage areas, large landscaped areas) generating an above average amount of rainfall runoff volumes and/or pollutant loads.

An area that is not surrounded by an area described above. Island parcels of land that are completely surrounded by urban land covers may also be considered urban, even though the existing land cover may be something else.

Culvert Sizing Quick Check

To confirm field observations, or where visual observation of a culvert is inconclusive, these tables in offer a check of culvert size for "replace in kind" structures. The tables trend towards being conservative and are intended for small watersheds typical to the maximum "replace in kind" culvert size described in this part. These tables shall not be used to size culverts requiring complete hydrology and hydraulic analysis. The tables can be used however as part of the QA/QC of the H&H drainage design.

The tables require the user to have a general idea of land cover, soil type, and watershed area. This does not have to be an extensive delineation and characterization of the watershed. Only the basic characteristics of the watershed are required. The tables assume a time of concentration based on the size of the watershed. For additional information on selection of a curve number ("C") refer to FDM 13-10-5.3 and FDM 13-10 Attachment 5.2 Runoff Coefficients (C), Rational Formula; and Runoff Coefficients for Specific Land Uses.

This check should also be only part of the evaluation of "in kind" replacement. The tables are not meant to dictate the need to increase or reduce the size of an existing culvert, they are intended as a check. Still, in the event the in-place culvert size and the tabulated size are substantially different, a full H&H analysis may be appropriate.

Typical Culvert Sizing – Western and Southwestern Wisconsin – Corrugated Metal Culverts

Drainage Area (acres)	Diameter of Culvert (inches)			
	Wooded/ Gentle Slope (C=0.2)	Mixed Wooded/Open Space. Low to Medium Density Development (C=0.4)	Steeper Slopes with limited vegetative cover, Commercial Areas (C=0.7)	Impervious (C=0.9)
0-2	24	24	24	24
2-5	24	30	36	36
5-10	30	36	42	48
10-15	30	36	42	48
15-20	30	42	48	Perform H&H
20-30	36	48	Perform H&H	Perform H&H
30-40	36	48	Perform H&H	Perform H&H
40-50	42	Perform H&H	Perform H&H	Perform H&H
50-75	48	Perform H&H	Perform H&H	Perform H&H
75-100	Perform H&H	Perform H&H	Perform H&H	Perform H&H

Additional Notes:

1. Assumes 25-year storm for rural class roadway with ADT <7,000.
2. 25-year rainfall was derived from typical volumes in updated IDF curves, NOAA Atlas 14, Volume 8.
3. Time of concentration is assumed to increase and therefore design rainfall intensity decreases with drainage area size.
4. The pipes are assumed to not be completely submerged by backwater.
5. A maximum HW/D of 1.5 is assumed per FDM 13-15-5.5.
6. For culverts up to 100 feet.

Typical Culvert Sizing – Far Northwestern and Southeastern Wisconsin – Corrugated Metal Culverts

Drainage Area (acres)	Diameter of Culvert (inches)			
	Wooded/ Gentle Slope (C=0.2)	Mixed Wooded/Open Space. Low to Medium Density Development (C=0.4)	Steeper Slopes with limited vegetative cover, Commercial Areas (C=0.7)	Impervious (C=0.9)
0-2	24	24	24	24
2-5	24	30	30	36
5-10	30	36	42	48
10-15	30	36	42	48
15-20	30	42	48	Perform H&H
20-30	36	48	Perform H&H	Perform H&H
30-40	36	48	Perform H&H	Perform H&H
40-50	42	48	Perform H&H	Perform H&H
50-75	48	Perform H&H	Perform H&H	Perform H&H
75-100	48	Perform H&H	Perform H&H	Perform H&H

Typical Culvert Sizing – Northeast Wisconsin – Corrugated Metal Culverts

Drainage Area (acres)	Diameter of Culvert (inches)			
	Wooded/ Gentle Slope (C=0.2)	Mixed Wooded/Open Space. Low to Medium Density Development (C=0.4)	Steeper Slopes with limited vegetative cover, Commercial Areas (C=0.7)	Impervious (C=0.9)
0-2	24	24	24	24
2-5	24	24	30	36
5-10	24	36	42	42
10-15	30	36	42	48
15-20	30	36	48	48
20-30	36	42	Perform H&H	Perform H&H
30-40	36	42	Perform H&H	Perform H&H
40-50	36	48	Perform H&H	Perform H&H
50-75	42	Perform H&H	Perform H&H	Perform H&H
75-100	48	Perform H&H	Perform H&H	Perform H&H

Additional Notes:

1. Assumes 25-year storm for rural class roadway with ADT <7,000.
2. 25-year rainfall was derived from typical volumes in updated IDF curves, NOAA Atlas 14, Volume 8.
3. Time of concentration is assumed to increase and therefore design rainfall intensity decreases with drainage area size.
4. The pipes are assumed to not be completely submerged by backwater.
5. A maximum HW/D of 1.5 is assumed per FDM 13-15-5.5.
6. For culverts up to 100 feet.

Typical Culvert Sizing – Western and Southwestern Wisconsin – Concrete and Thermoplastic Culverts

Drainage Area (acres)	Diameter of Culvert (inches)			
	Wooded/ Gentle Slope (C=0.2)	Mixed Wooded/Open Space. Low to Medium Density Development (C=0.4)	Steeper Slopes with limited vegetative cover, Commercial Areas (C=0.7)	Impervious (C=0.9)
0-2	24	24	24	24
2-5	24	24	30	36
5-10	24	36	42	48
10-15	30	36	42	48
15-20	30	36	48	Perform H&H
20-30	36	48	Perform H&H	Perform H&H
30-40	36	48	Perform H&H	Perform H&H
40-50	42	48	Perform H&H	Perform H&H
50-75	48	Perform H&H	Perform H&H	Perform H&H
75-100	48	Perform H&H	Perform H&H	Perform H&H

Typical Culvert Sizing – Far Northwestern and Southeastern Wisconsin – Concrete and Thermoplastic Culverts

Drainage Area (acres)	Diameter of Culvert (inches)			
	Wooded/ Gentle Slope (C=0.2)	Mixed Wooded/Open Space. Low to Medium Density Development (C=0.4)	Steeper Slopes with limited vegetative cover, Commercial Areas (C=0.7)	Impervious (C=0.9)
0-2	24	24	24	24
2-5	24	24	30	36
5-10	24	30	42	42
10-15	24	36	42	48
15-20	30	36	48	48
20-30	36	42	Perform H&H	Perform H&H
30-40	36	48	Perform H&H	Perform H&H
40-50	36	48	Perform H&H	Perform H&H
50-75	42	Perform H&H	Perform H&H	Perform H&H
75-100	48	Perform H&H	Perform H&H	Perform H&H

Additional Notes:

1. Assumes 25-year storm for rural class roadway with ADT <7,000.
2. 25-year rainfall was derived from typical volumes in updated IDF curves, NOAA Atlas 14, Volume 8.
3. Time of concentration is assumed to increase and therefore design rainfall intensity decreases with drainage area size.

4. The pipes are assumed to not be completely submerged by backwater.
5. A maximum HW/D of 1.5 is assumed per FDM 13-15-5.5.
6. For culverts up to 100 feet.

Typical Culvert Sizing – Northeast Wisconsin – Concrete & Thermoplastic Culverts

Drainage Area (acres)	Diameter of Culvert (inches)			
	Wooded/ Gentle Slope (C=0.2)	Mixed Wooded/Open Space. Low to Medium Density Development (C=0.4)	Steeper Slopes with limited vegetative cover, Commercial Areas (C=0.7)	Impervious (C=0.9)
0-2	24	24	24	24
2-5	24	24	30	30
5-10	24	30	36	42
10-15	24	30	42	42
15-20	30	36	42	48
20-30	30	42	48	Perform H&H
30-40	36	42	Perform H&H	Perform H&H
40-50	36	48	Perform H&H	Perform H&H
50-75	42	Perform H&H	Perform H&H	Perform H&H
75-100	48	Perform H&H	Perform H&H	Perform H&H

Additional Notes:

1. Assumes 25-year storm for rural class roadway with ADT <7,000.
2. 25-year rainfall was derived from typical volumes in updated IDF curves, NOAA Atlas 14, Volume 8.
3. Time of concentration is assumed to increase and therefore design rainfall intensity decreases with drainage area size.
4. The pipes are assumed to not be completely submerged by backwater.
5. A maximum HW/D of 1.5 is assumed per FDM 13-15-5.5.
6. For culverts up to 100 feet.

FDM 13-5-1 Introduction
August 8, 1997

1.1 Introduction
Field surveys are routine and are intended to give the designer a clear picture of existing conditions at any location where water comes to and/or leaves a proposed project.

This procedure contains a general description of the field survey data required for the hydraulic design of small culverts, large culverts, and bridges. For a more detailed description of drainage survey requirements and procedures, the structural hydraulic engineer, roadway drainage engineer and survey chief are referred to FDM 9-55-1 through FDM 9-55-15 of this manual.

FDM 13-5-5 Survey Data
August 8, 1997

5.1 Drainage Cross Section for Small Culverts
Show clearly the traverse and stream profile. With this information the designer can:

1. Determine if reduction in water elevation is practical.
2. Determine if realignment or relocation can better serve the overall design.
3. Proportion the structure dimensions.

Show topography and elevations of existing ditches and natural streams in detail. Usually, infall information is necessary only for 100 to 300 feet or a sufficient distance upstream to indicate the degree of channelization and direction of flow.

Information on the outfall portion should extend far enough to determine the direction and degree of channelization, the rate of fall in water surface, and the effect of channelization on downstream structures and development, etc. Any apparent constriction in outfall portions should be noted, located, and cross-sectioned.

If ditch or channel work is desired, topography will be necessary downstream to a point at which damage to adjacent property need no longer be considered.

The field party will have to exercise its own judgment in many cases where the slope along a line being considered as an outfall is very flat. Use of such an outfall is in most cases a matter of economics.

5.2 Drainage Surveys for Large Culverts and Bridges
The meander of both banks of a stream for sufficient distance upstream and downstream to determine the approximate extent of any probable channel relocation should be shown in the field survey. This ordinarily can be shown within 500 to 1,000 feet laterally from the structure site.

Any major overflow channels should also be indicated within approximately the same limits or within the limits that these channels leave and return to the main channel. Meandering channels close to and approximately parallel to the project center line should be located carefully and cross-sectioned.

If the proposed project follows an existing fill that crosses a floodplain, cross sections should extend laterally far enough to provide a record of natural ground profiles right and left of the project. Any washouts or significant swales, runs, or sloughs should be noted clearly in the topography.

For bridges it is necessary to obtain certain existing high-water elevation data. If reliable data are not available, that fact should be noted by the field party.

Record, if available, the extreme high water within the proposed, or existing structure location and give the approximate date of occurrence. If other high waters can be dated, supply as many as practical, showing dates of occurrence.

If possible, determine a "normal" high-water elevation or one that can be expected to recur about every two to three years. Record a normal water elevation that would be expected to prevail through seasons of average rainfall. High-water locations within one-quarter mile are also of value.

5.3 Preliminary Field Review

A field review of the proposed structure site is important to the designer in terms of a physical determination of structure size and the legal responsibility of the Department of Transportation. Information gathered from this review should include existing and past flood conditions, special controls on flood rates, proposed changes to existing conditions, and possible tail-water controls. Prior to the field review a preliminary flow rate and structure size should be determined from an office review of the soil types, land usage, and from as-built plans.

5.3.1 Preliminary Flow Rate

After the drainage pattern and the drainage areas have been outlined on a contour map, a preliminary peak flow rate should be calculated to give the engineer an idea of the proposed size of the structure (see FDM 13-10-5 for methods of computing peak flow rates). Comparing this preliminary work-up with existing conditions or structures may point out the accuracy of the work-up. The determination of the preliminary flow rate will require that a field review be made of the drainage areas with regard to land usage. It should be noted whether the land is wooded, fallow, plowed and planted with crops, or urban development containing a high percentage of roof areas, paved parking lots, grass lawns, etc. All of these factors will affect the flow rate computations, especially with the use of the Rational Method (Q = CIA).

1. Soil Types: Soil characteristics of the various drainage areas for the project should be considered prior to a field review. The engineer must be aware of the runoff characteristics of the soils, the capabilities of the soil to resist erosion, locations where soil erosion may be a problem, and locations where channel banks may need riprap for protection against erosion. This information may be obtained from consultation with the region soils engineer or from soils maps. Design engineers should also obtain their own firsthand knowledge of the various soils throughout the region.

2. As-Built Plans: If the project consists of reconstruction of an existing highway, the as-built plans of the highway can be used as a guide for the sizing of the proposed structures and the determination of drainage patterns. As-built plans should be used during the field review to aid in the determination of the drainage patterns and drainage areas.

5.3.2 Existing and Past Flood Conditions

After the information noted above has been accumulated and reviewed, the design engineer is prepared for a field trip to review the existing hydrology and hydraulics of the drainage patterns and drainage areas. The field trip is required to allow the engineer to become aware of the effects caused by the new highway on the existing drainage patterns. The things to look for in the field may not be easily discernible, since time has a way of healing and disguising flood damage. The following information should be gathered:

1. High-Water Elevations for Channels: Existing stream channels (when there is a definite channel) show flooding effects in various ways. The effects can be shown by different degrees of erosion in the stream bank or the stream bottom. Sometimes scouring can occur during low-frequency storms as well high-frequency storms. The degree of scouring will depend upon the type of soil material that exists in the stream bottom and sides. Other indications that may be evident are high-water marks shown as mud lines on concrete surfaces or rock faces along stream embankments and debris that has been deposited along the channel slopes. These high-water marks are only to be used as indications that serve to confirm or validate design flow rate computations.

2. Personal Interviews: Another method of determining previous flooding problems is by personal interview of area residents. This information may not be 100 percent correct, because some people's recollections become exaggerated over time. It has been found that information obtained from interviews of local residents is helpful and can be used as an indication of high-water elevations in conjunction with the proper computations and field observations.

3. Approximate Flow in Channels: When high-water elevations are determined in a channel, a rough approximation of the flow rate can later be determined by estimating an average velocity that is compatible with the type and slope of the channel and then multiplying this velocity by the estimated waterway area determined from rough field measurements.

4. High Water Elevations for Existing Structures: High-water elevations determined for an existing drainage structure will enable the engineer to calculate a more definite flow rate that might have existed at that structure. The information needed for an existing structure is the approximate depth of the headwater, the approximate depth of the tail water, the slope of the existing structure, and the type and size of the structure. This information can then be analyzed through the use of FDM 13-15-10 and FDM 13-20-1.

5. Deposition and Scour at Existing Structures: Signs of flooding at existing culverts are scour marks around the inlets of the structures. Scour marks around the outlets of structures are not as indicative of

flood conditions, since normal flows through a structure can cause scour and erosion at the outlet. Deposition of stream load, which will consist of sand, gravel, or other debris, can occur within the upper portions of a drainage structure that does not have any significant outlet control. Deposition and scour marks are indications of possible build-up of headwater at the inlet, which shows that a culvert may be too small.

6. Debris and Velocity: The size and weight of deposited material is also a general indication of the velocity of the stream during maximum flow conditions. As the stream channel grade steepens and as the flow rate increases, the velocity will be faster and the heavier debris is more easily moved by the flowing water. By observing these general characteristics of a stream and culvert, the engineer can make a better determination of the past velocity and flow rate to confirm the design flow rate.

5.3.3 Special Controls on Flow Rates in Drainage Areas

Special conditions should be considered during the field review that will affect the time of concentration and the flow rate for a stream channel or structure. These special controls are existing swamps, ponded areas, flood control dams, reservoirs, and lakes. These all have the effect of increasing the time of concentration, which reduces the flow rate and the size of the structure at the point under consideration. If the control is high in the headwaters of the drainage area, its effects can usually be ignored, since the delay in the time of concentration will not be as great as the effect would be if the control were immediately upstream of the point of highway crossing. However, if the storage feature is close to the point of crossing, a special analysis will have to be performed to determine the proper flow rate. This analysis, referred to as flood routing for reservoirs and ponding areas, is discussed in FDM 13-10-10. In some cases storage basins, which have a retarding effect on the flow of a stream, can reduce the size of a required structure appreciably.

When reviewing a drainage area that does contain a flow rate control system, consideration will have to be given to the future existence of this flow control feature. For example, if the feature is a city reservoir, a large natural lake, or a permanent flood control system, it is very doubtful that the effect from the feature will be altered in any way in the near future that will require a larger pipe structure for the highway crossing. However, if there is a swamp area or natural low spots in the surrounding terrain that tend to hold water during high-water stages, these features can very easily become obliterated as commercial and urban development progresses in the area. If the latter situation prevails, it is best to consider designing the culvert size under the highway to accommodate the flow rate that is uninhibited by the existing storage areas. This would be more practical than designing a small pipe, with the possibility of having to increase the culvert capacity at a greater cost in the future.

5.3.4 General Guides

During the field review, special consideration should be given to the effects of the proposed drainage patterns on any future flood conditions upstream or downstream. The following guides should be followed when designing a drainage system for a highway:

1. Every effort is to be made in the design of a highway to perpetuate the drainage pattern that existed prior to the construction of the highway project. Collection and diversion of flows should be avoided whenever possible.
2. When existing drainage patterns are disturbed by a highway project (by collection, diversion, elimination of ponding areas, or an increase in stream velocities), provisions are to be made to return the drainage pattern downstream of the highway to approximately the conditions existing prior to the highway project. Whenever possible, the natural drainage pattern is to be reestablished within the highway right-of-way.
3. Drainage easements (usually permanent easements) may be purchased from private property owners. This should be done only in special situations (refer to FDM 13-1-5).
4. Under special circumstances, overflow sections may be considered.

The purpose of all highway drainage design is to plan for the removal of water from the highway, prevent surface runoff from reaching the highway, and pass existing streams under the highway economically while disturbing the surrounding environment as little as possible.

5.4 Changes in Existing Flow Conditions

The proposed highway alignment can force changes of existing flow conditions in various ways. The first is the collection and concentration of water through a structure under the proposed highway. This water would normally flow in an overland sheet pattern. The second is the change of depth of floodwaters immediately above the proposed highway.

When the drainage area in the first case noted above has existing conditions that do not allow the accumulation of the runoff, the amount of water contained in the stream channel under flood conditions may not be excessive. However, if after the highway is constructed the water is concentrated through a single structure, the Department may be responsible for causing a flood condition that does not exist under present conditions. Therefore, a careful field review should be made of possible concentrations of water through a structure and a careful note of the provisions for a clear and definite outlet.

The second of the changes of flow conditions noted above can occur when an existing stream channel is inhibited by the proposed culvert and highway embankment. The stream channel under existing conditions would allow the water to flow unobstructed with an acceptable depth, but under the proposed conditions the water would be forced to develop a greater depth at the inlet of the structure. The engineer should be aware of the approximate depth of headwater required for the new conditions and review the drainage area in the field for possible locations of flood damage as a result of increased depth of floodwaters. A stream channel that is too shallow to contain an increased depth of water will allow the water to flow over its banks and cause flooding in another drainage area, flooding of buildings, or flooding onto or across a highway embankment.

5.5 Tail-Water Controls

The tail-water depth at the outlet of a structure may directly affect the headwater depth at the inlet of the structure. Tail-water depth, as defined for the purposes of designing culverts, is the depth of water at the outlet of a structure that will affect the flow of water through a structure. The depth of tail water other than that determined by the use of critical depth can normally be calculated by the use of Manning's equation, with either a field cross section of the existing channel or a new cross section of the proposed channel at the outlet end of the culvert.

There are conditions in the field that may give a variable effect on the tail water at the outlet of a structure other than that computed from Manning's equation. These conditions can be water surface variations in lakes controlled by power authorities or recreation authorities, barge canals, flood stages in large streams and rivers, etc. These variable restrictions on the tail water may force the culvert to act in inlet control when the tail-water surface is down to a low elevation or may force the culvert to act in outlet control when the tail water is at its maximum elevation. The maximum and minimum elevations of the tail water should be recorded during the field review. The culvert should then be designed for the condition that yields the largest culvert size. The engineer may be able to eliminate the effect of the variable tail water by placing the outlet of the culvert above the maximum high-water elevation of the control feature.

The field review should also include notations on features that will impose a fixed control on the tail water. This tail-water control may be from existing drainage structures downstream of the proposed structure. Information should be obtained to perform an analysis on the existing downstream structures to determine what effect it has on the proposed culvert.

5.6 Final Field Review

Sometimes all of the problems mentioned in the foregoing paragraphs are not immediately evident during a preliminary field review but may have to be determined by a more accurate field survey and by taking additional cross sections. After the preliminary field trip is completed and the follow-up design progresses, another field trip will have to be made to confirm that the structure designs proposed for each structure site are appropriate.

FDM 13-10-1 Design Criteria

August 8, 1997

1.1 Introduction

To the highway engineer, hydrology includes the analysis of precipitation and runoff, and the determination of a flood flow rate for a given stream or channel. It also addresses the frequency of flood occurrence.

1.2 Flood Frequency

Flood frequency or recurrence interval is defined as the average interval in years between the actual occurrence of a hydrological event of a given or greater magnitude. For example, a flood frequency of 50 years means that a storm of that magnitude or greater would be expected to occur on the average of once every 50 years. It also can be stated that a 50-year flood would have a 2% chance of occurring in any one year.

Flood frequencies for various classes of highways and types of drainage structures have been selected to produce a balance between the cost of a drainage facility and the cost of potential flood damage - including risk to the traveling public. These selected frequencies are referred to as design flood frequencies or design frequencies, and are used in determining the magnitude of the design flood - which the drainage structure must accommodate with low probability of risk to the traveling public, minimum damage to the roadway, and minimum flood damage to adjacent property. By common definition, the design flood does not inundate the roadway. In many instances, the design flood will not approach overtopping of the roadway, but will be limited to a maximum backwater elevation so as not to create unreasonable flood damage to either the roadway or adjacent property.

1.3 Design Frequency

The hydraulic design of drainage structures shall use the flood design frequencies given in Attachment 1.1 of this procedure. Design frequencies for bridges and box culverts are not included in this attachment, but the procedure for their sizing is discussed in the Bridge Manual Chapter 8.

1.3.1 Major Drainage Structures

Watercourses of sufficient magnitude to potentially produce significant flood damage (to the roadway, drainage structure, or abutting property) are most frequently crossed using a major drainage structure (a bridge, box culvert, or their replacement with large drainage conduits). Therefore, when a major drainage structure is required, the process of selecting a design frequency which best produces a balance between structure costs and the cost of potential flood related damages or risks, requires a detailed analysis of each situation. It also requires that the designer be knowledgeable of FAPG Part 650A, "Location and Hydraulic Design of Encroachments on Flood Plains;" NR 116, "Wisconsin's Floodplain Management Program;" NR 320, "Bridges an Culverts in or Over Navigable Waterways;" and the "Cooperative Agreement Between the Wisconsin Department of Transportation and Department of Natural Resources" (refer to https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/environment/formsandtools.aspx).

Therefore, the following method should be used when designing a major drainage structure:

The hydraulic design of major drainage structures is to be addressed in terms of either a replacement structure condition, or a structure associated with a highway on new location.

Replacement structures should typically be sized to develop headwater elevations not greater than that experienced with the existing structure in place. This presumes that extensive experience at the existing structure site has indicated acceptable backwater elevations, permissible stream velocities, and adequate protection for the roadway and motorist. When this is the case, the headwater elevation for the regional flood (100 year-flood) with the existing structure in place should be computed and used as a controlling hydraulic factor in the design of the replacement structure.

Occasionally a reasonable increase in headwater depth would lead to a material savings in structure costs that would obviously outweigh backwater related impacts or risks. In these situations, the acceptable headwater elevation under regional flood conditions should be determined and then used as a controlling hydraulic factor in the structure sizing. The "acceptable" headwater elevation must also take into consideration the floodplain management standards of NR 116, relevant local floodplain zoning ordinances, and the potential need for drainage easements.

Upon completion of the structure design, predicted water surface elevations shall be made available to the applicable local zoning authorities. When a structure is located on a stream that has an established water surface profile for the regional flood incorporated into the local zoning ordinance, the region shall provide the local agency with the predicted water surface elevations. It is then incumbent upon the local agency to amend their zoning ordinance, as outlined in NR 116, whenever the headwater elevation would be increased over that contained in the zoning ordinance.

Structures for highways on new locations should generally be designed to accommodate the regional flood without increasing the backwater (0.01') over that of existing conditions. However, if reduced structure costs significantly outweigh any backwater related impacts, the procedures required for its accomplishment are the same as previously described for replacement structures.

Requirements for documentation of structure sizing are contained in FDM 13-1-10, "Documentation of Hydrologic/Hydraulic Design."

Plan survey datum must conform to datum in use by local zoning authorities. The datum in almost all cases are USGS or USC and GS datum.

1.4 Freeboard Considerations

The provisions for freeboard in the design of bridges is desirable and should be achieved whenever practicable. While sound engineering judgment must be used in this determination, experience has shown that 2 ft of freeboard for the 100-year flood provides a reasonable allowance for the passage of debris, ice flow, etc. under extreme flood conditions. If other factors outweigh the achievement of a 2 ft freeboard (e.g. high cost, undesirable profile, etc.), this should be documented in the "Discussion of Structure Sizing" which is addressed in FDM 13-1-10.

Freeboard may also be necessary to provide reasonable clearances for navigation purposes. Section NR 320 of the Wisconsin Administrative Code makes reference to a 5-foot clearance over navigable waterways, which is measured from a waterway's "ordinary high-water mark" as would be evident from observation of the stream bank. The need to provide freeboard for this purpose should be investigated whenever existing usage of the waterway would indicate that this is a relevant consideration.

A discussion on the hydraulic design of culverts and associated freeboard considerations is given in FDM 13-15-5.

1.5 Use and Design of Overflow Sections

Normally, hydraulic structures on arterials should be designed to convey an appropriate frequency of flood without inundation of the highway. However, under special circumstances on collectors and local roads, a specified flood (i.e. overtopping flood) may be conveyed by the structure and an overflow section, both acting together as a hydraulic system. This type of design should be undertaken only after considering an incremental analysis of estimated construction costs; probable property damage, including damage to the highway; traffic volumes and the cost of traffic delays; duration and depth of inundation; frequency of occurrence; length of roadway to be flooded; availability of alternate routes, emergency supply, and evacuation routes; and considering the potential for loss of life and budgetary constraints.

Where possible, the roadway approach embankments for an overflow section should be constructed slightly above the design flood elevation while the low point of the superstructure should be constructed with an appropriate amount of freeboard. With this type of design, the structure would convey the design flood while the overflow section would convey the "super flood" (or unusually large flood). Thus, large floods would cause minimal damage to the structure itself. If during flood stage the overflow section operates as a weir having no downstream tail water, the downstream roadway embankment may erode. Under these circumstances the downstream roadway embankment of the overflow section should be protected with riprap or some other erosion-resistant material if significant damage is likely to occur.

References detailing the hydraulic design of overflow sections are contained in FDM 13-20-1.

For projects on collectors or local roads that are being designed in anticipation of roadway overflow, the designer should consider specifying "HIGH-WATER" advance warning signs. In general, this sign should be used when all three of the following conditions exist:

1. The current ADT is greater than 300 AND
2. The operating speed exceeds 35 mph AND
3. The expected overflow frequency is more often than once every ten years, i.e. the ten-year storm is expected to cause overflow.

1.6 Probability of Flood Occurrence

The probability (P) that an event with a recurrence interval t_p will be equaled or exceeded in any one year is:

$$P = 1/t_p$$

For example, floods with recurrence intervals of 10 years, 50 years, and 100 years are also called a "10 percent flood," a "two percent flood," and a "one percent flood," respectively. In other words, in any one-year period the probability of getting a 10-year, 50-year, or 100-year flood is 10 percent, two percent, and one percent, respectively.

When communicating with the public about specific floods, it is probably more effective to talk about a percent flood instead of recurrence interval. The use of recurrence interval may give the false impression that a specific flood will occur only at those intervals, whereas in fact there is a specific constant probability that it will be equaled or exceeded in any one year.

The risk of flooding is the probability that a flood with a given probability will be equaled or exceeded at least once in a specified number of years. Attachment 1.2, which lists risks of flooding for various design periods and recurrence intervals, shows that there is a 64 percent chance that the 50-year flood (or greater) will occur in any 50-year period, and even a 40 percent chance that the 100-year flood (or greater) might occur in the 50-year period.

The risk of floodland occupancy can be determined from Attachment 1.3. For example, if the "100-year flood stage" is coincident with the first floor of a building, the probability of first floor (or more) damage before the 25-year mortgage is paid is 22 percent.

For values not listed in Attachment 1.2, the risk of flooding R that at least one event that equals or exceeds the t_p year event will occur in any series of N years is:

$$R = 1-(1-P)^{N1.}$$

1.7 Future Development Effects

Future land development and urbanization will greatly affect the anticipated runoff peaks and volumes for some drainage structures and ditches. In most cases it is very difficult to predict the type and extent of future urbanization. Despite this, it is suggested that the following methods be used as guides in this regard:

1. Areas of 200 acres or less should have a runoff coefficient C for the rational formula determined on the basis of future anticipated conditions. If the majority of the drainage area will be urbanized, the Rational Method may be used on areas up to five square miles.
2. For drainage basins less than five square miles with scattered urban development and for urban drainage basins over five square miles, comprehensive studies of the watershed must be undertaken. These comprehensive studies entail using synthetic hydrographs, which are combined and routed through the drainage basin to the design structure and/or drainage channel.

   Use a runoff figure based on land development expected in the watershed 20 years in the future. Data on existing and future land use can be obtained from regional planning commissions. In addition, these regional planning commissions have published comprehensive plans for various watersheds, which give flood flows for present and/or future (20 years hence) land-use conditions.

1.8 Hydraulic Information on Plans

The hydraulic data that must be shown on structure plans is given in WisDOT's Bridge Manual. This includes providing the flood magnitude and water surface elevation (headwater) associated with the 100-year flood. If the roadway will be overtopped by a flood of lesser magnitude than the 100-year flood, the recurrence interval of the overtopping flood and its magnitude should also be given. When the overtopping flood is greater than the 100-year flood a note should be included with the hydraulic data that states, "Overtopping Road Not Applicable."

Whenever it is determined to use large drainage conduit to replace a major drainage structure, the hydraulic data noted above shall be provided on an appropriate roadway plan sheet.

LIST OF ATTACHMENTS

FDM 13-10-5 Methods of Determining Peak Runoff

November 15, 2019

5.1 Design Discharge

The first step in designing a hydraulic structure is to determine the amount of water to be carried - the design discharge. The problem is particularly difficult for small watersheds, say, under five square miles, because the smaller the area, the more sensitive the design discharge is to conditions that affect runoff and the less likely there are runoff records for the area.

The design discharge is related to the effective rainfall, which is that portion of precipitation that produces direct runoff. Losses or abstractions are that portion of precipitation that is removed from direct runoff through detention, infiltration, evapotranspiration, etc. The best method of determining a design discharge is to use site specific runoff records; but, since these are often non-existent, estimates of runoff must be based on frequency of rainfall by assuming the runoff to have the same frequency as the rainfall of the design storm or on flood-frequency equations developed from regional gauging stations.

There are many methods used to determine discharge values. The methods presented in this chapter may be classified as being based on rainfall frequency (first two methods), runoff records (next two methods), a combination of rainfall frequency and runoff records (next method), and historic data (the last method).

The runoff methods presented in this chapter are:

1. Rational method
2. Hydrology for small watersheds, NRCS - Urban Hydrology for Small Watersheds, (TR-55)
3. USGS flood frequency equations for Wisconsin
   - Flood Frequency Characteristics of Wisconsin Streams, 1992
   - Techniques for Estimating Magnitude and Frequency of Floods in Wisconsin - 1981
   - Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams - 1986
4. Gauging station
   - Log-Pearson Type III distribution
   - Transferring gaged discharges
5. Published watershed studies
6. Field review notes, interviews, and historic data

Due to inherent differences in the methods, it is recommended that the designer compute runoff by at least two of these methods. The results serve as a comparison check and may be averaged or weighted according to the most applicable method to arrive at a design discharge. Attachment 5.1 is a guideline for area limits of various methods.

5.2 Discharge Frequency Graph

A discharge-frequency graph should be constructed for each of the runoff methods presented in this chapter. For an example of the construction and use of a discharge-frequency graph, the designer is referred to the design methods entitled "Flood Frequency Characteristics of Wisconsin Streams" and "Techniques for Estimating Magnitude and Frequency of Floods for Wisconsin," which are contained in this procedure.

5.3 Rational Method

The Rational Method has been the most common approach used to design storm sewers since the publication of a paper by Kuichling in 1889 [1]. The rational formula has the advantage that its physical meaning is reasonably clear. However, it should be used with caution, because it can overestimate peak flows for large drainage basins. As stated previously, comparing multiple methods of determining peak flow is always advised.

The Rational Method is recommended for use in estimating design discharges for urban areas or potential urban areas of five square miles or less. In addition, it may also be used for small rural basins 200 acres or less having similar or non-similar ground cover, e.g., combinations of woodlands, pastureland, and cropland.

The basic assumptions for the Rational Method are:

1. Peak flow occurs when the entire watershed is contributing to the flow.
2. Rainfall intensity is the same over the entire drainage area.
3. Rainfall intensity is uniform over time duration equal to the time of concentration, t_c. The time of concentration is the time required for water to travel from the hydraulically most remote point of the

basin to the point of interest.

4. Frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 10-year rainfall intensity is assumed to produce the 10-year peak flow.
5. Coefficient of runoff is the same for all storms of all recurrence probabilities.

The rational formula is:

$$Q = CIA$$

Where:

Q = peak runoff rate in cubic feet per second (cfs)

C = runoff coefficient, which is the ratio of the peak runoff rate to the average rainfall rate for a duration equal to the time of concentration

I = intensity of rainfall for a duration equal to the time of concentration in inches per hour

A = drainage area in acres

Note that the rational formula is not dimensionally correct, but 1.008 cfs = 1 ac-in/hr

Runoff Coefficient: A matrix of runoff coefficients (C) for various types of land use, hydrologic soil groups, and land slopes is shown in Attachment 5.2, Details A and B. FHWA policy is to use a consistent value for the runoff coefficient, C, over all storm recurrence intervals. The composite runoff coefficient is the weighted average C value of the various surface types.

Time of Concentration: The time of concentration t_c is defined as the flow time from the most remote point (point from which the time of flow is greatest) of the drainage area to the design point. In practice, it is considered to be composed of an overland flow time (called inlet time in urban areas) plus a channel flow time. The time of concentration for small drainage basins can be obtained from the nomograph in Attachment 5.3. The channel flow time may also be determined by dividing the longest channel by the average velocity of flow in the channel at about bank-full stage. For most basins WisDOT's preferred method to compute the time of concentration is using TR-55 methodology, which is detailed in Reference 5: "Urban Hydrology for Small Watersheds, TR-55 by NRCS. A computation tool to determine time of concentration can be found in FHWA's Hydraulic Toolbox software.

In rare instances, partial basin contributions may produce higher peak flows than full basin contributions. This usually occurs when the area near the discharge point has runoff coefficients higher than the rest of the basin.

For example, the area could be a parking lot for the small basins or a large subdivision for the large basins. The combination of higher runoff coefficients and higher rainfall intensity caused by the shorter t_c results in higher peak flows.

Rainfall Intensity: The value of rainfall intensity for various rainfall durations (times of concentration) and recurrence intervals is obtained from the intensity-duration-frequency curves in Attachment 5.4 which are derived from NOAA Atlas 14, Vol. 8: Precipitation Frequency Atlas of the United States[2]. NOAA Atlas 14, Vol.8 was released in June 2013 updating rainfall data in TP No. 25[3] and TP No. 40[4].

Drainage Area: The drainage area, A, can be determined using Geographic Information System (GIS) or civil engineering design software. Drainage area maps should be retained as design documentation.

5.3.1 Rational Method - Example Problem

Refer to Attachment 5.5. Note that the area of this example is out of the normal range of 0 to 200 acres of the Rational Method. However, this drainage basin is used in the example problems throughout this procedure. Therefore, a good comparison of the application of different runoff methods to the same drainage basin is produced.

Drainage Area = 1,067 acres

To find the time of concentration, divide length AC into two lengths of different characteristics.

1. Well-defined channel with heavy grass:
   - $Length_{(1)}$ = 2,500 feet
   - $Fall_{(1)}$ = 200 feet
2. Well-defined channel:
   - $Length_{(2)}$ = 8,300 feet
   - $Fall_{(2)}$ = 62 feet

From Attachment 5.3, read:

t_{cl} =8.5 minutes and modify to 10 minutes

t_{c2} =54 minutes

Time of Concentration = 10 + 54 = 64 minutes

3. From Attachment 5.6, the hydrologic soils group is determined to be B-C.

   Design for a 50-year recurrence interval.

   Enter the La Crosse intensity-duration-frequency curve (see Attachment 5.4, 16 of 36) at 64 minutes and 50 years and find the rainfall intensity I as 2.90 inches per hour.

Table 5.1 Composite Runoff Coefficient

Land Use	C *	Percent	Products
Woods	0.25	40	10
Mixed Cover	0.30	60	18
Thus;			
Weighted C = 28/100 = 0.28			
Q = C*I*A = (0.28)(2.90)(1067) = 866 cfs.			

* = refer to Attachment 5.2.

5.4 Urban Hydrology for Small Watersheds (TR-55)

The Natural Resources Conservation Service (NRCS) created TR-55 for estimating the volume and rate of runoff in watersheds that range in size from 1 to 2000 acres. It provides two methods for doing this, the Graphical Peak Discharge method and the Tabular Hydrograph method. Both methods are derived from TR-20 (NRCS 1983) output. For a description of the hydrograph development method used by NRCS, see chapter 16 of the National Engineering Handbook, Section 4 - Hydrology (NEH-4) (NRCS 1985) [6]. The routing method (Modified Alt -Kin) is explained in appendixes G and H of the draft Technical Release 20 (TR-20) (NRCS 1983). TR-55 software can be downloaded from the following site:

http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?&cid=stelprdb1042925

At this time the windows version (WinTR-55) should not be used. Instead, use Version 2.1 of TR-55 (simplified flood peak and hydrograph development for small watersheds). Despite its title, TR-55 is applicable to rural as well as urban drainage basins. The Graphical Peak Discharge method is outlined below.

Basically, the runoff volumes are determined by using the following parameters:

1. Soil type (see Appendix A); TR-55, for definitions of hydrologic soil types A, B, C, and D. Also refer to Attachment 5.6.
2. Cover type.
3. Rainfall depths (24-hour duration) for selected recurrence intervals.

In addition to the above parameters, peak rates of discharge are related to:

1. Rainfall distribution type.
2. Flow length.
3. Land slope, watercourse slope, channel slope.
4. Drainage area.
5. Percent ponding and swampy areas.

All of these parameters may be converted to numerical figures by using the design figures in TR-55. Hydrologic results by this procedure are for a Type II rainfall distribution (standard NRCS design rainfall distribution applicable to Wisconsin). This method, unlike most other methods, does provide a means to include the effects of ponding and swampy areas, thus lowering the peak runoff.

Procedure: The design figures used in this procedure are located in the TR-55 publication. A list of soil names and their hydrologic classification is located in TR-55, Appendix A.

5.4.1 TR-55 - Example Problem

See Attachment 5.5. This drainage basin is in a rural area of Jackson County with no foreseeable urbanization.

- Drainage Area (A_m) = 1,067 acres = 1.67 mi^2
- Composite Runoff Curve Number (CN):

Table 5.2 Composite Runoff Coefficient

Land Use	HGS	CN *	Percent	Products
Woods (good cover)	B-C	62.5	40	2500
Mixed Cover	B-C	75.5	60	4530
(conservative treatment)			Sum =	7030
Composite CN = (7030/100) = 70				

*Refer to TR-55, Table 2-2a - 2-2d or Attachment 5.6

Design for a 50-year recurrence interval.

- From TR-55, Chapter 3, T_c = 1.43 hr
- From TR-55, Chapter 4, table 4-1, Initial abstraction I_a = .857
- From TR-55, Appendix B, page B-8, 50 year, 24 hour rainfall P = 5.3 inches
- Compute I_a/P = .16
- From TR-55, Table 2-1, Runoff Depth Q=2.29 inches (by interpolation)
- From TR-55, exhibit 4-II, q_u = 264
- No ponding or swamp areas, F_p = 1.0
- $Q_{50} = q_p = q_u A_m Q F_p$ = (264)(1.67)(2.29)(1)
- Q_{50} = 1010 cfs

For an urban drainage basin, use the same general procedure as used in the above sample problem. However, the curve numbers must reflect an urban land use.

Refer to Attachment 5.7 for an example using NRCS TR-55 Urban Hydrology for Small Watersheds version 1.11.

5.5 USGS Flood Frequency Equations for Wisconsin

The U.S. Geological Survey and WisDOT have an ongoing cooperative agreement for analyzing gaging station data to develop general flood-frequency relationships for streams with any size drainage basin. To date, the USGS has published seven reports containing methods for estimating specific flood-frequency relationships (Q_2, Q_5, Q_{10}, Q_{25}, Q_{50}, and Q_{100}).

Flood-frequency equations and comparison methods acceptable to WisDOT for the design of culverts, bridges, and flood protection structures are contained in the three reports entitled:

1. Flood-Frequency Characteristics of Wisconsin Streams, 1992 [8]
2. Techniques for Estimating Magnitude and Frequency of Floods for Wisconsin, 1981 [9]
3. Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams, 1986 [11]

Since the 1992 flood-frequency equations were developed from more years of record, they are statistically more accurate than the 1981 flood-frequency equations. This is evident by the decrease in the standard error of prediction in many of the equations. The 1981 publication is still widely used for the method of transferring discharges at gaged sites to ungaged locations using regional drainage-area exponents.

These flood-frequency equations are applicable to all drainage areas in Wisconsin, EXCEPT for highly regulated streams, some urban developments, and certain areas of the state, as noted in the reports.

The three methods show the standard error of estimate (SE) for each equation so that the user can evaluate the accuracy of the results. The standard error of estimate is defined as "a range of error such that the value estimated by the regression equation is within this range at about two out of three sites and is within twice this range at about 19 out of 20 sites" (Thomas, C.M., and Benson, M.A. 1969, "Generalization of Stream Flow Characteristics", U.S. Geological Survey, Open-File Report, 45 pp.).

The most recent version of Natural Resource Rule Chapter 116 (NR 116), effective March 1, 1986, states that the current USGS empirical equations (see reference 8) may be used in the estimate of the Regional Flood Discharges [10].

The computed discharge by the USGS empirical equations should be used for design proposes after verification by other methods and/or discharge-frequency curves of stream gaging stations of comparable drainage basins. Methodologies for comparisons are described below.

5.5.1 Flood-Frequency Characteristics of Wisconsin Streams (8)

Flood-Frequency characteristics for gaged sites on Wisconsin Streams are presented for recurrence intervals of 2 to 100 years (Q_2 to Q_{100}). This publication also presents the equations of the relations between flood-frequency and drainage-basin characteristics that were developed by multiple-regression analysis of the gage data. The most significant characteristics considered in this analysis were drainage area, stream slope, storage, forest cover, mean annual snowfall, precipitation intensity, and soil permeability. Flood-Frequency characteristics (Q_2 through Q_{100}) for ungaged sites on unregulated, rural streams can be estimated by use of these equations. This publication divides the state into five regions and lists a set of flood-frequency equations for each area. Each set of equations is correlated with three or more basin characteristics.

5.5.2 Flood-Frequency Characteristics - Example Problem

Using the same example problem data as for the previous examples, determine the Flood-Frequency characteristics for this basin.

1. From Reference Number 8, Figure 3, the basin is in Area 2. From Table 1, the required parameters for the Area 2 equations are Area (A), Soil Permeability (SP), and Main Channel Slope (S).
2. From the USGS quadrangle map in Figure 5, the area = 1067 acres = 1.67 square miles. Drainage area data for Wisconsin streams may also be obtained from Drainage Area Data from Wisconsin Streams [13].
3. From Reference No. 8, Plate 2 the Soil Permeability for this site is 1.65 inches per hour. It is recommended that Plate 2 of reference No.8 be the source for soil permeability for use in the USGS regression equations.
4. The altitude at the 10 percent point (0.20 mile) is 965 feet and the altitude at the 85 percent point (1.74 miles) is 1055 feet. The average slope (S) is:

   $$\frac{1065 - 965}{1.74 - 0.20} = 58.4 \text{ feet per mile}$$

5. Compute the 100 year (Q_{100}) recurrence interval runoff.
   - $Q_{100} = 17.7\text{x}(A)^{0.947}\text{x}(SP) -0.713\text{x}(S)^{0.682}$
   - $Q_{100} = 17.7 \text{ x } (1.67)^{0.947} \text{ x } (1.65)^{-0.713} \text{ x } (58.4)^{0.682}$
   - Q_{100} = 322 cfs
6. The peak runoffs for the 2-, 5-, 10-, 25-, and 50-year recurrence intervals are computed with the remaining Area 2 regression equations and yield the following results:
   - Q_2 = 75 cfs
   - Q_5 = 134 cfs
   - Q_{10} = 178 cfs
   - Q_{25} = 236 cfs
   - Q_{50} = 279 cfs
7. The discharge-frequency curve is constructed by plotting the computed discharges against their respective frequencies on log probability paper and fitting a smooth curvilinear line to those points. This discharge-frequency curve is used in picking (interpolating) a new discharge(s) for the selected design frequency(ies) (see Attachment 5.8).

5.5.3 Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams (11)

This report provides a method for estimating the frequencies and magnitudes of floods of ungaged urban streams in Wisconsin. Multiple regression techniques were used to develop flood-frequency equations by relating flood frequency and magnitude characteristics for 32 sites (gages) to basin characteristics, such as drainage area and impervious area. Two sets of equations were developed one set applicable to urban drainage areas in all parts of Wisconsin without significant regulation or diversion and another set applicable only to Milwaukee County. These equations utilize only Drainage Area (A) and Impervious Area (I) and the independent variables. Estimated flood discharges by regression equations should be compared to flood discharges determined from gaged basins with similar types of development whenever possible.

5.5.4 Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams - Example Problem

Use of the Flood-Frequency equation is illustrated by the following problem in which the magnitude of the 100-year flood (Q_{100}) for the urban gaging station 05430403, Fisher Creek Tributary at Janesville, WI, is determined. The applicable equation from Table 2 of Reference 11 is:

$$Q_{100} = 32.8(A)^{0.704} \times (I)^{0.770} \text{ (cfs)}$$

1. Determine the size or the contribution drainage area (A) in square miles from the best available topographic city maps.

   A = 1.88 square miles

2. Compute the percentage of total impervious area (I). See reference 11 pages 9 and 17 for discussion on technique that includes single-family residential, multifamily residential, commercial, industrial, and public facilities.

   I = 19.0%

3. Determine the flood discharge using the selected 100-year flood equation from Table 5.
   - $Q_{100} = 32.8\ (1.88)^{0.704} \times (19.0)^{0.770}$
   - $Q_{100} = 32.8 \times 1.56 \times 9.65$
   - $Q_{100} = 494$ cfs

5.6 Gaging Station Data

In addition to computing discharges by the aforementioned methods, a comparison should be made with steam gaging data from similar drainage basins in the locality. Records of stream flow at gaging stations, partial record stations, and miscellaneous sites are collected as part of the National Water Data System operated by the U.S. Geological Survey and cooperating state and federal agencies in Wisconsin.

Through water year 1960, these records were published in an annual series of U.S. Geological Survey water supply papers entitled "Surface-Water Supply of the United States." Beginning with the 1961 water year, stream flow data have been released in a state boundary basis by the Geological Survey in annual reports entitled "Water Resources data for Wisconsin, Water Year ____."

A search for a stream gaging station must include perusing all available USGS published reports for stream flow data, because the data for some gages are not published every year. Moreover, the data for discontinued gages will only be found in the editions published during the years the gage was operating. If this search fails, the USGS office located in Madison, Wisconsin, may be able to furnish unpublished stream flow data. In any case, they will be able to furnish a complete set of annual flood peak flows for any specific gaging station.

Annual flood peak flows through water year 1988 for most Wisconsin gaging stations, with 10 or more years of records, have been published by the USGS in Flood Frequency Characteristics of Wisconsin Streams [8] Table 6.

Additional annual flood peak flows for years after 1988 may be obtained from the annual Water Resource Data Wisconsin Water Year [14] published yearly by the U.S. Geological Survey.

5.7 Log Pewarson Type III Distribution

This technique is used to construct flood-frequency curves where systematic stream gaging records of sufficient length (at least 10 years) to warrant statistical analysis are available as the basis for the determination. A thorough description of this method is located in Bulletin #17B of the Hydrology Committee, U.S. Water Resources Council entitled "Guidelines for Determining Flood Flow Frequency," September 1981 [12].

One exception to the procedure in Bulletin #17B (update of #17 and #17A) is listed in the Wisconsin Administrative Code, Chapter NR 116.07(1)(a): "When determining skew, a log normal analysis (zero skew) shall be used instead of the generalized skew map found in Bulletin #17" [10].

USGS Published Solutions: The USGS has performed Log Pearson Type III flood-frequency analyses (Bulletin #17A Procedures) at most Wisconsin gaging stations having 10 or more years of record (through 1978 water year) to determine flood-frequency characteristics. Estimates of the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval floods for each of these stations has been published by the USGS in Flood Frequency Characteristics for Wisconsin Steams [8], Table 4.

As more years of data are collected, these published flood-frequency characteristics will become obsolete, and additional gaging stations will meet the 10 or more years of record criterion. Therefore, the published flood-frequency characteristics should only be used for preliminary design. For final design, the designer should collect the additional years of peak data and determine new flood-frequency characteristics with a Log Pearson Type III analysis.

5.8 Transferring Gaged Discharges

In most design problems, there is no gage station located at the project site. The nearest comparison gage or study may be located some distance upstream or downstream. A reasonable comparison gage or study may even be outside of the project basin. The design discharge developed for a gaging station or study site may be transferred to the design site by an equation that relates the discharges and drainage areas of two distinct drainage areas with similar drainage basin characteristics. There are a number of methodologies for transferring gaged and other accepted studied discharges to a project site. The three methods presented here will be the 1992 USGS Adjustment Method, the 1981 USGS transfer method, and Comparison of Similar Drainage Basins at Gaged Sites.

1992 USGS Adjustment Method: This method uses the combination of data for the nearest similar gaging station and data determined by use of the USGS multiple-regression equations [8]. The procedure is applicable for sites that have a drainage area within 50 percent of the drainage area of the gaging station. This procedure was used by Curtis (1987) for streams in Illinois. The procedure is defined on pages 13 and 14 of reference (8) and as follows:

First the regression equation correction or adjustment ratio r is defined by:

$$r = Q_a / Q_r$$

Where:

- Q_a is the accepted (log Pearson III) flood-frequency characteristic at the gaging station.
- Q_r is the flood-frequency characteristic determined for the gaged station by use of the multiple regression equation.

The adjustment for difference in drainage area is determined by r' such that:

$$r' = r - [(A / (0.5 \times A_g)) \times (r - 1.0)]$$

Where:

r = defined above

A = is absolute value of the difference in the drainage area between the ungaged site and the gaged site.

A_g = is the drainage area of the gaged site.

The adjusted flood-frequency characteristic for the project site Q_w is computed by the equation:

$$Q_w = Q_{rug} * r'$$

Where Q_{rug} is the flood-frequency characteristic determined for the ungaged site by the multiple regression equation.

1981 USGS Transfer Method: This method accounts for difference in the drainage area between the gaged site and ungaged upstream or downstream project site. Basically this technique computes a weighted design discharge at the up- or downstream site by weighting the transferred discharge with the flood-frequency (multiple-regression equation) discharge. As the project drainage area approaches that of the gage drainage area the weighted transferred flow at the project site approaches that of the gage. Also, as the difference in drainage area between the gage and the project approaches 50% of the area of the gage, the transferred weighted flow at the project approaches the flow value determined by the regression equation. A thorough discussion of this method is contained in reference (9), pages 11-14.

The transferred discharge, Q_{ud}, is determined by the following formula:

$$Q_{ud} = Q_g \times (A_{ud} / A_g)^{n}$$

Where:

Q_{ud} = is the discharge at the project site transferred from the gage site by drainage-area ratio.

Q_g = Discharge at the gage site for selected recurrence interval.

N = 1981 USGS regional drainage-area exponent (reference 9, page 12).

- area 1 = 0.59
- area 2 = 0.68
- area 3 = 0.76

- area 4 = 0.60
- area 5 = 0.63

A_{ud} is the drainage area at the project site.

A_g is the drainage area at the gage station.

Then weight this discharge (Q_{ud}) with the discharge (Q_r) determined at the project site by the regression equation with the following equation:

$Q_w = Q_{rud} \times (2A/A_g)$

$= Q_{ud} \times (1-(2A/A_g))$

Where:

Q_{rud} = discharge at project site determined by the regression equation.

Q_w = the weighted discharge for the project site.

A = is the absolute value of the difference between the drainage area at the project site and the gage station.

5.8.1 Transferring Gaged Discharges - Example Problems

This example problem will illustrate both the USGS 1992 and the USGS 1981 Transfer methods.

Problem:

Determine the best estimate of the Q_{100} design discharge for Rowen Creek at Main Street in the Village of Poynette, Columbia County.

Given:

- Drainage Area (A) = 10.6 square miles
- Main-Channel Slope (S) = 30.4 feet per mile
- Storage (ST) = 0.3% + 1.0% = 1.3%
- Precipitation Intensity Index (I24-2) = 2.75
- Intens = (I24-2) - 2.3 = 0.45"
- Soil Permeability (SP) = 1.42 inches per hour.

The Q_{100} flow for the Main Street site by regression equation is determined by Equation #30, Table 1 of Reference (8) as follows:

$Q_{rud}(100) = 64.8\ (A)^{0.863} \times (S)^{0.460} \times (ST)^{-0.299} \times (SP)^{-0.302} \times (Intens)^{0.808}$

$Q_{rud}(100) = 64.8(10.6)^{0.863} \times (30.4)^{0.460} \times (1.3)^{-0.299} \times (1.42)^{-0.302} \times (0.45)^{0.808}$

$Q_{rud}(100)$ = 1043 cfs

Gage Station 5405600 Rowen Creek at Poynette Wis.:

- Drainage Area (A_g) = 10.4 square miles
- Storage (ST) = 0.0% + 1.0% = 1.0%
- Precipitation Intensity Index (I_{24}-2) = 2.75"
- Intens = (I_{24} -2) - 2.3 = 0.45"
- Soil permeability (SP) = 1.42 inches per hour
- Log Pearson Q_{100} at Gage = 2180 cfs (ref. 10)
- 1992 100-year Regression Equation Q_{100} = 1030 cfs (ref. 8)

1992 USGS Transfer Method:

First, $r = Q_g / Q_r$

Q_g = 2180 cfs

Q_r = 1030 cfs

r = 2180/1030 = 2.12

Next, $r' = r - (A /(0.5 \times A_g)) \times (r - 1.0)$

A = | 10.6 - 10.4| = 0.2

r' = 2.12 - (0.2 / (0.5 x 10.4)) x (2.12 - 1.0)

r' = 2.08

The transferred flow $Q_w = Q_{ud} \times r'$ (Q_{ud} = 1043 from regression equation at project site)

Transferred Flow = $Q_w = 1043 \times 2.08 =$ 2169 cfs

1981 USGS Transfer Method.

First: $Q_{ud} = Q_g \times (A_{ud} / A_g)^{n}$

(n = 0.63 Area 5)

$Q_{ud} = 2180 \times (10.6 / 10.4)^{0.63}$

Q_{ud} = 2206 cfs

Next: $Q_w = Q_{rud} \times (2 \times A / A_g) + Q_{ud} \times (1 - 2 \times A / A_g)$

$Q_w = 1043 \times (2 \times 0.2/10.4) + 2206 \times (1 - 2 \times 0.2/10.4)$

Transferred Flow Q_w = 2161 cfs

5.9 Comparison of Similar Drainage Basin at Gaged Sites

This method can be used as a check of the regression equations when there are no gaging stations up- or downstream of the project site. This method uses the same drainage area discharge transfer equation as the 1981 USGS Transfer Method to calculate Q_{ud}. However, the calculated transferred flow Q_{ud} may then be further adjusted to account for dissimilar basin parameters between the comparison gage and the project site. The other dissimilar basin parameters are then adjusted in the same manner as the drainage area with the parameters of the project site prorated to the gage site and raised to the appropriate exponent. This factor is then multiplied by Q_{ud}. As many basin parameters can be adjusted as needed, however, the best comparison gages tend to be in the same region with similar basin parameters. Therefore, good comparison gages tend to need few basin parameters adjusted.

Each dissimilar basin parameter that is to be adjusted is prorated to the related gage parameter, then this ratio is raised to the 1992 USGS Regression equation exponent for the subject parameter. The basin parameter exponent should correspond to the regression equation used to estimate the discharged at the project site.

The Transfer Equation takes the form:

$$Q_w = Q_{ud} \times (S_s / S_g)^{Ns} \times (ST_s/ST_g)^{Nst} \times (SP_s / SP_g)^{Nsp}$$

Where:

- Q_w = the Transferred Flow
- Q_{ud} = defined in 1981 USGS Transfer Method ref. (9)
- S_s, ST_s, SP_s, ... etc. = basin parameters at the project site.
- S_g, ST_g, SP_g, ... etc. = basin parameters at the comparison gage site.
- N_s, N_{st}, N_{sp}, ... etc. = basin parameter exponents from 1992 regression equation used to estimate Q_{rug}.

5.9.1 Comparision of Similar Drainage Basin at Gaged Sites - Example Problems

This example problem will illustrate the use of the Comparison of Similar Drainage Basins at gages Method.

Problem: Compare or "Transfer" a 100-year flow for a similar gaged basin to McAdam Branch at Morgan Road in Grant County.

Given: McAdam Branch Drain Area (A) = (A_{ud}) = 6.63 sq. mi.

Precipitation Intensity Index (I24-2) = 3.03"

Intens = (I24-2) - 2.3 = 0.73"

Main Channel Slope (S) = 58.0 feet per mile.

From Table 1 Equation 6 of reference (8)

$Q_{rud}(100) = 342 \times (A)^{0.848} \times (Intens)^{4.06} \times (S)^{0.512}$

$Q_{rud}(100) = 342 \times (6.63)^{0.848} \times (0.73)^{4.06} \times (58.0)^{0.512}$

$Q_{rud}(100)$ = 3790 cfs

Gage 05413400, Pigeon Creek near Lancaster Wis.

Drainage Area (A_g) = 6.93 sq. mi.

Precipitation Intensity Index (I24-2) = 3.02"

Intens = (I_{24}-2) - 2.3 = 0.72"

Main Channel Slope (S) = 49.8 feet per mile

$Q_g(100)$ = 3620 cfs (102.5 m^3/s) (Table 4 ref. (8))

$Q_r(100)$ = <u>3440 cfs</u> (97.4 m^3/s) (Table 5 ref. (8))

<u>First:</u> Basin parameter exponents for transfer method:

Exponent for drainage Area (A) = n = 0.59 (9)

Exponent for Intens (Intens)= N_i = 4.06 (8)

Exponent for Slope (S) = N_s = 0.512 (8)

$Q_{ud} = Q_g \times (A_{ud} / A_g)^{n}$

$Q_{ud} = 3620 \times (6.63/6.93)^{0.59}$

Q_{ud} = <u>3526 cfs</u> (99.8 m^3/s)

<u>Next:</u> the Transferred Flow Q_w is found by further adjustment of basin parameters,

$Q_w = Q_{ud} \times (INTENS_s / INTENS_g)^{Ni} \times (S_s / S_g)^{Ns}$

$Q_w = 3526 \times (0.73 / 0.72)^{4.06} \times (58.0 / 49.8)^{0.512}$

Q_w = <u>4031 cfs</u> (114.1 m^3/s)

This transferred flow may indicate that the regression equations are under estimating flows for basins with these characteristics. This also may be evident from a comparison of the regression results (Q_r) and Log Pearson results (Q_g) at the gage.

5.10 Published Watershed Studies

Pertinent hydrologic and hydraulic information for a specific watershed may be obtained from these studies, thus saving many hours of tedious work.

In years past, watershed studies have been prepared and published by many communities because of local flooding problems. Many additional studies have been prompted by the Department of Housing and Urban Development's (HUD) Flood Insurance Program, which was established by the Congress in the National Flood Insurance Act of 1968 and expanded in the Flood Disaster Protection Act of 1973. These studies are now published by the Federal Emergency Management Agency (FEMA).

These watershed studies have been prepared and published by the following agencies:

1. Regional Planning Agencies.
2. U. S. Army Corps of Engineers.
3. U. S. Natural Resources Conservation Service.
4. U. S. Geological Survey.
5. Consulting engineering companies.

A list of these studies, entitled "Floodplain Management Community Status Report," may be obtained from the Wisconsin Department of Natural Resources. This report lists the rivers by county and community with the following information:

1. DNR district.
2. Ordinance dates (adopted and approved).
3. Insurance information (date, type of map).
4. Report publication date.
5. Class of study (flood insurance study, floodplain management, etc.).
6. Source of information (DNR, HUD, NRCS, etc.).
7. Type of district (general, floodplain, etc.).

5.11 Field Review Notes, Interviews, and Historical Data

Field review notes of stream channels and existing structures can indicate high-water elevations that have occurred in the past.

Field interviews of local residents can be very important in determining past flow rates. The high-water elevations pointed out by local residents can be used to compute a flow rate. This can be done by determining a cross-sectional area of the water and an average velocity with Manning's formula and multiplying the two together.

By making a hydraulic analysis of an existing structure with those field-determined headwater depths and tailwater depths, a past flow rate can be determined.

Historic flood information of extreme high-water elevations can often be used to make estimates of peak discharges. The USGS includes some historic flood information in its published reports and computer files. Additional information can sometimes be obtained from the files of other agencies or extracted from newspaper files. If such records are located, a search of the National Weather Service records should be made to determine the corresponding rainfall intensity in the immediate drainage area.

There is one flaw in the above-mentioned flow rates, namely, the lack of knowledge of corresponding recurrence intervals. Therefore, the determined flow rates can only be used as a comparison to confirm and justify the finalized design flow rate. Moreover, it can also be used to show that the design flow rate determined by various mathematical methods is erroneous.

5.12 References

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3. U.S. Department of Commerce, Weather Bureau, "Rainfall Intensity-Duration-Frequency Curves for Selected Stations in the United States and Puerto Rico, Technical Paper No. 25, 1955."

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6. U.S. Department of Agriculture, Natural Resources Conservation Service, "National Engineering Handbook," Section 4, Hydrology (NEH-4), 1985.

7. U.S. Department of Transportation, Federal Highway Administration, "Hydrology," Hydraulic Engineering Circular No. 19, McLean, Virginia, 1984, 343 pp.

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10. Wisconsin Administrative Code, Department of Natural Resources, "Wisconsin's Floodplain Management Program," Chapter NR 116, March 1, 1986.

11. Conger, Duane H., "Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams," U.S. Department of the Interior, Geological Survey Water Resource Investigation Report 86-4005, Madison Wis. 1986.

12. United States Water Resources Council, Hydrology Committee, "Guidelines for Determining Flood Flow Frequency," Bulletin #17, March 1976, Bulletin #17A, June 1977, Bulletin #17B, September 1981.

13.* Water Resource Data Wisconsin Water Year, U.S. Department of Interior, Geological Survey Water Resource Division, open file report WI-89-1 to current year 1994 WI-94-1.

* *Henrich, E.W., Daniel, D.N., Drainage Area Data for Wisconsin Streams, U.S. Department of Interior, Geological Survey Water Resource Division, Open-file Report 83 - 933.*

NOTE: Documents prepared by the USGS - Water Resources Division can be obtained by calling (608) 828-9901.

LIST OF ATTACHMENTS

FDM 13-10-10 Hydrograph Development and Routing *August 8, 1997*

10.1 Development

The first step in designing a hydraulic structure is to determine the amount of water to be carried also called the design discharge. The problem is particularly difficult for small watersheds, say, under five square miles, because the smaller the area, the more sensitive the design discharge is to conditions that affect runoff and the less likely there are runoff records for the area.

A hydrograph is defined as the graph of flow (rate versus time) at a stream section. The four basic hydrograph types are:

1. Natural Hydrographs: Obtained directly from the flow records of a gaged stream.
2. Synthetic Hydrographs: Obtained by using watershed parameters and storm characteristics to simulate a natural hydrograph.
3. Unit Hydrographs: A natural or synthetic hydrograph for one inch of direct runoff. The runoff occurs uniformly over the watershed in a specified time.
4. Dimensionless Hydrographs: Made to represent many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units. Also called the "Index Hydrograph."

Hydrographs are used in the planning and design of water control structures, especially detention basins, which are used to minimize downstream flooding by attenuating the peak flows of storms with specific duration frequencies. They are also used to show the hydrologic effects of existing or proposed projects.

The urbanization of rural areas increases peak flows, which has and will continue to overtax existing downstream structures such as highway drainage facilities. Replacing such overtaxed facilities with larger or additional structures is one option, but designers should also investigate adding a detention basin(s) upstream of the problem structure.

For both large and small watersheds, the hydrograph development methods discussed in this section are:

1. HEC-1
2. The Natural Resources Conservation Service (NRCS) Tabular Method, TR-55
3. The Unit Hydrograph Method
4. The NRCS Triangular Dimensionless Unit Hydrograph Method
5. The NRCS Curvilinear Dimensionless Unit Hydrograph Method

These methods can be easily applied through manual computations to small watersheds, but not large watersheds, hence, it is necessary to use a computer program in these cases. The computer program selected for inclusion here is the NRCS TR-55, "Urban Hydrology for Small Watersheds" which makes use of the NRCS curvilinear unit hydrograph.

10.1.1 HEC-1

HEC-1 was developed by the U.S. Army Corps of Engineers, Hydraulic Engineering Center. It is designed to simulate surface runoff from various duration storms over a watershed. The conversion of precipitation to direct runoff can be simulated by HEC-1 for both small and large watersheds. Hydrograph combining, channel and reservoir routing and sub-basin runoff are some of the basic components that HEC-1 uses for a simple or complex watershed study.

The HEC-1 computer package has the following capabilities:

1. Simulates watershed runoff and stream flow from design or historical rainfall.
2. Uses unit hydrograph, loss rate and stream flow routing procedures from measured data.

3. Simulates reservoir and channelization flood controls.

10.1.2 NRCS Tabular Method, TR-55

The Tabular Method is an approximation of the more detailed hydrograph analysis contained in Section 4-Hydrology of the NEH-4 (4). Composite hydrographs can be developed for any point within a watershed by dividing the watershed into subareas, developing simple hydrographs for each subarea, routing the simple hydrographs to the point in question, and adding the routed simple hydrographs. The factors required to determine these hydrographs are:

- 24-hour rainfall amount,
- a given rainfall distribution (Type II in Wisconsin),
- hydrologic soil cover complexes (runoff numbers),
- time of concentration,
- travel time, and
- drainage area.

This method should not be used when the runoff curve numbers of the subareas vary appreciably and when runoff volumes are less than 1.5 inches for curve numbers less than 60. Moreover, for most watershed conditions (urban or rural), this procedure can be used to determine hydrographs for subareas up to approximately 2000 acres.

For a thorough discussion of the Tabular Method, with an accompanying example problem, see routing section.

10.1.3 Unit Hydrograph

The unit hydrograph is a very important tool for estimating runoff amounts for various frequencies that may occur at a specific point of a stream. The use of this method requires continuous records of runoff and precipitation for the specific drainage basin.

Sherman(6) defined the unit hydrograph as a hydrograph with a one-inch volume of runoff from a rainstorm of specified duration, time-intensity pattern, and areal pattern. Increasing the duration of the rainfall increases the unit hydrograph time base and peak, because the unit hydrograph contains only one inch of runoff.

In practice, unit hydrographs are generally based on an assumption of uniform intensity of rainfall. Usually the Unit Hydrograph Method is applied to basins small enough so that the areal pattern is rather uniform. The acceptable drainage basin size is equal to or less than 200 square miles.

Theoretically, a given drainage basin will exhibit an infinite number of unit hydrographs, one for every possible duration of rainfall, every possible time-intensity pattern, and every possible areal pattern. In design practice, only the duration of the rainfall is allowed to vary, while variations in areal patterns are ignored. Moreover, unit hydrographs are developed from rainstorms that exhibit basically a rainfall pattern of uniform intensity. Short-duration unit hydrographs can be used to develop a unit hydrograph resulting from a long rain of varying intensity.

10.2 Procedure

The basic steps in the development of a unit hydrograph are:

1. Analyze the stream-flow hydrograph separating the surface runoff from the groundwater flow.
2. Determine the total volume of direct runoff from the storm that produced the original hydrograph. This volume is equal to the area under the original hydrograph minus the groundwater flow area.
3. Divide each ordinate of the direct runoff hydrograph by the total direct runoff volume in inches. The unit hydrograph is the plot of these answers against time.
4. Finally, determine the effective duration of the rainfall that produced this unit hydrograph. This can be obtained by studying the hyetograph of the rainfall.

Generally, the hydrograph for a given drainage basin for a specified design storm (duration, effective rainfall, or total runoff) may be constructed by multiplying each ordinate of the specified duration unit hydrograph by the total runoff (inches).

10.3 NRCS Triangular and Curvilinear Dimensionless Unit Hydrograph Methods

Basically, the Triangular and Curvilinear Methods are the same, except the Triangular Method, as its name implies, substitutes a dimensionless unit hydrograph for the more accurate curvilinear dimensionless unit hydrograph. This method develops synthetic hydrographs for a specific watershed by using watershed parameters, storm characteristics, and a dimensionless unit hydrograph. The dimensionless unit hydrograph was developed from a large number of natural unit hydrographs from watersheds varying widely in size and geographical location.

The shape of the dimensionless unit hydrograph is determined by the drainage area and time of concentration, hence, the watershed should be divided into hydrologic units of uniformly shaped areas. If possible, these subareas should be less than 20 square miles and exhibit a homogeneous drainage pattern.

The basic data required to develop synthetic hydrographs are:

1. Twenty-four-hour and/or six-hour rainfall amount for a specific rainfall frequency.
2. Rainfall distribution.
3. Hydrologic soil cover complexes (runoff numbers).
4. Times of concentration for the subareas.
5. Travel times through reaches.
6. Drainage areas for each sub-area.

For a thorough discussion of this method, with accompanying example problems, see Chapter 16 of NEH-4 (4). In addition, these synthetic hydrographs can also be generated by computer through the use of version 2.1 of NRCS-TR-55 (5).

10.4 Routing

Hydrograph development and hydrograph routing are closely interrelated. A simple hydrograph for a subarea of a watershed can and is developed without routing, but the downstream, more complex hydrographs must be developed through routing and/or combining the simple upstream hydrographs.

In the American Society of Civil Engineers Manual, "Nomenclature for Hydraulics," flood routing is variously defined as follows:

routing, (hydraulics) (1): The derivation of an outflow hydrograph of a stream from known values of upstream inflow. The procedure utilizes wave velocity and the storage equation; sometimes both (2). Computing the flood at a downstream point from the flood inflow at an upstream point, and taking channel storage into account.

routing, flood: The process of determining progressively the timing and shape of a flood wave at successive points along a river.

routing, stream flow: The procedure used to derive a downstream hydrograph from an upstream hydrograph, or tributary hydrographs, and from considerations of local inflow by solving the storage equation.

The purpose of flood routing is to mathematically determine from the inflow hydrograph the shape of the outflow hydrograph at specific locations in streams or structures during passages of floods. These outflow hydrographs are used in designing a water control structure or project.

Detention and retention basins have been used to control the effects and results of urbanization and urban runoff hydrology.

Urbanization Can Cause:

1. Reduction in natural storage capacity.
2. Increase in impervious area.
3. Greater direction and conveyance of runoff.

Urban Runoff Hydrology Results In:

1. Higher peak discharge (2 to 5 times).
2. Shorter time to peak, as high as 50 percent.
3. Higher velocity of storm runoff.
4. As much as 50 percent increased volume of storm runoff.
5. Reduction of infiltration, inflow and base stream flows.

To help alleviate these problems it may be necessary to design a retention/ detention facility. This facility may be designed as a pond, underground tank or parking lot as well as other types of facilities.

The following steps should be performed to assure a proper design.

1. Determine the purposes for which the basin will be used.
2. Determine the design storm inflow hydrographs before and after development.
3. Estimate the volume of storage needed.
4. Determine the depth-storage curve for the basin.
5. Select the outlet structure types compatible with the uses outlined in step 1 and determine the depth-outflow curve.
6. Determine the routing curve.
7. Perform the routing.
8. Add additional outlet features to ensure that the peak outflow rate is reduced to at least the pre-development rate for the more frequent storms.
9. Perform the routings for these smaller storms to ensure compliance.
10. Check the length of time needed to empty the basin for the various storms to determine if the other uses of the basin will be unduly delayed and/or if water quality detention times are met.

For the example shown in this procedure, a detention pond (122' x 122') will be designed.

Note: The NRCS publication (reference 5) is needed to fully understand the following example.

10.5 Detention Pond Example

(NRCS TR-55 Tabular Hydrograph Method)

Given:

1. Area of Watershed = 10 Acres
2. Curve Number = 75 *
3. 50 Year 24 hour Rainfall = 5"
4. Time of Concentration (t_c)=18 minutes
5. Type II Rainfall Distribution
6. Maximum Post Q £ Pre Q of 8 cfs.

Note: Items 2 - 5 can be determined by using Chapters 2 and 3 of NRCS TR-55(5) and associated exhibits and figures.

Procedure:

The procedure shown below is based on the steps described above.

1. The basin is to be used as a detention pond.
2. Determine storm inflow hydrograph:
   A. Determine runoff from Table 2-1, NRCS TR-55 (5).

      Rainfall = 5"

      CN = 75

      Runoff = 2.45"
   B. Complete work sheet (Attachment 10.1 B)
   C. Complete work sheet (Attachment 10.2 B) using a Type II rainfall distribution to develop a hydrograph. See reference 5. The NRCS TR-55 computer program (version 2.1 non-Windows) may be used instead of manually calculating the results of steps A - C.
   D. Plot the tabulated hydrograph. See Attachment 10.3.
3. Determine volume of storage required to detain a 50 year storm with Q = 8 cfs.

   Required storage can be determined by assuming an outflow curve (see reference 5 and Attachment 10.3 for details) and determining the area between the inflow curve and the outflow curve. For this example, using a planimeter on the area between the curves in Attachment 10.3 yields a required volume of approximately 54,000 ft^3.

4. Depth Storage Relationship:

   We will first evaluate a trapezoidal storage pond with the following dimensions. (see Attachment 10.11)

   Square, L = W = 122 ft. (bottom of pond)

   Side slope (Z) = 4:1

The equation below can be used to find the volume of a trapezoidal pond. Use it to determine the depth needed to provide adequate storage for the detention pond.

Volume = LWD + (L + W) ZD^2 + 4/3 Z^2D^3	
Depth	**Volume**
0.5	7688
1.0	15881
1.5	24594
2.0	333842
2.5	43643
3.0	54011
3.5	64963

From this table a depth of 3.5 ft is chosen to provide ample storage plus some freeboard. See Attachment 10.8 for a plot of this data. This is the depth-storage relationship.

5. Determine outlet pipe size by using Attachment 10.4, with concrete pipe/grooved end with head wall, determine pipe size that can handle 8 cfs w/3.5 ft of head.

   For Attachment 10.4, use a 12" concrete pipe with a grooved end with head wall.

6. The depth/outflow relationship can be determined by multiple applications of Attachment 10.4 with a constant pipe diameter (D) of 1 ft. See the table below.

Depth (ft)	HW/D (ft)	Outflow (cfs)
.5	.5	.75
1.0	1.0	2.5
1.5	1.5	4.0
2.0	2.0	5.4
2.5	2.5	6.3
3.0	3.0	7.2
3.5	3.5	8.0

Plot the information as shown on Attachment 10.5.

7. Construct a storage indicator table, Attachment 10.6 B, plot column 2 vs column 6 to create a storage indicator curve as shown on Attachment 10.7. The curve is used to complete Attachment 10.9 B. When the storage indicator number (column 6 of Attachment 10.9 B) reaches a maximum, then peak discharge occurs.

Attachment 10.10 shows the actual inflow and outflow hydrographs. The peak outflow is 5.49 cfs. Since the maximum Q post = 8 cfs, this solution is acceptable. Therefore, steps 8-10 of the process need not be done. If a design with a Q post closer to 8 cfs is desired then the problem should be re-examined.

10.6 References

1. Poertner, Herbert G., "Practices in Detention of Urban Storm Water Runoff," American Public Works Association, Special Report No. 43, 1974.

2. Terstriep, Michael L., and Stall, John B., "Urban Runoff by Road Research Laboratory Method," Journal of the Hydraulics Division-ASCE, November 1969, pp. 1809-1834.

3. Stall, J.B., and Terstriep, M.L., "Storm Sewer Design--An Evaluation of the RRL Method," prepared for the Office of Research and Monitoring of the USEPA, October 1972.

4. U.S. Department of Agriculture, National Resources Conservation Service, National Engineering Handbook, Section 4, Hydrology (NEH-4), August 1972.

5. National Resources Conservation Service, Engineering Division, "Urban Hydrology for Small Watersheds," Technical Release 55, June 1986.

6. Sherman, L.K., "Stream flow from Rainfall by the Unit-Graph Method," Eng. News-Rec., Volume 108, pp. 501-505, 1932.

LIST OF ATTACHMENTS

Flood Design Frequency Selection Chart

Design Frequency - Years					Drainage Structures
Rural Class			Urban Class		
STH/CTH (1) T1, T2, T3, T4, T5, T6	**STH/CTH (2) T7**	**STH (3)**	**Urban Streets 1, 2, 3, 4, 5,**	**Urban Expressways & Freeways 1, 2, 3**	
See FDM 13-10-1, discussion of Major Drainage Structures					Bridges & Box Culverts
50	50	50	50	50	Underpass Storm Sewers
25	50	50	50	50	Main or Primary Channels
25	25	50	50	50	Cross Drain Pipe Culverts
25	25	25	25	25	Side Drain Pipe Culverts
25	25	25	25	25	Side Ditches and Channels
X	X	25	X	25	Median Ditches and Channels
X	X	X	10 **(4)** Check 25 Yr.	25	Urban Gutters, Inlets and Storm Sewers

(1) All state trunk highways with design ADT under 1500 and all county trunk highways with design ADT under 4000.

(2) All state trunk highways with design ADT of 1500-7000 and all county trunk highways with design ADT over 4000.

(3) All state trunk highways with design ADT of over 7000.

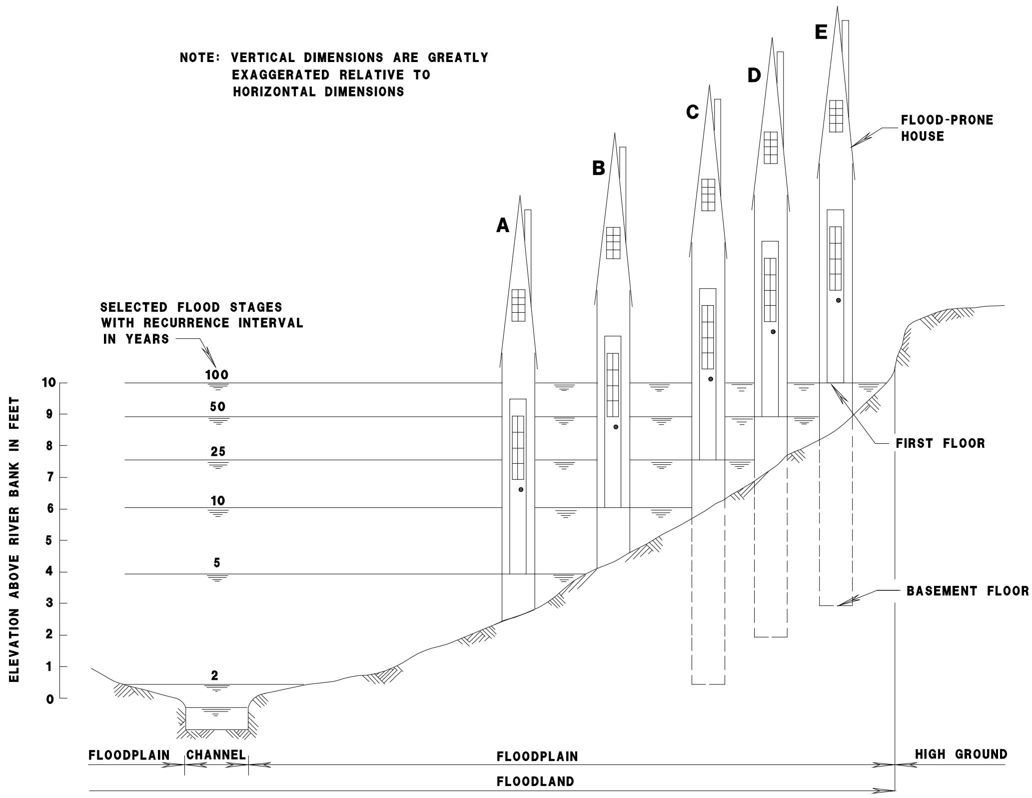
(4) See FDM 13-25-20

PROBABILITY OF FLOOD OCCURRENCE

Percent Chance of Equaling or Exceeding such a Flood at Least Once in This Many Years					Recurrence Interval (Years)
100 Years	50 Years	25 Years	10 Years	Any 1 Year	
--	99	93	65	10	10
98	87	64	34	4	25
87	64	40	18	2	50
64	40	22	10	1	100

PROBABILITY OF FLOOD DAMAGE BEFORE PAYMENT OF 25 YEAR MORTGAGE

CHANNEL, FLOODPLAIN, AND FLOOD-PRONE HOUSES

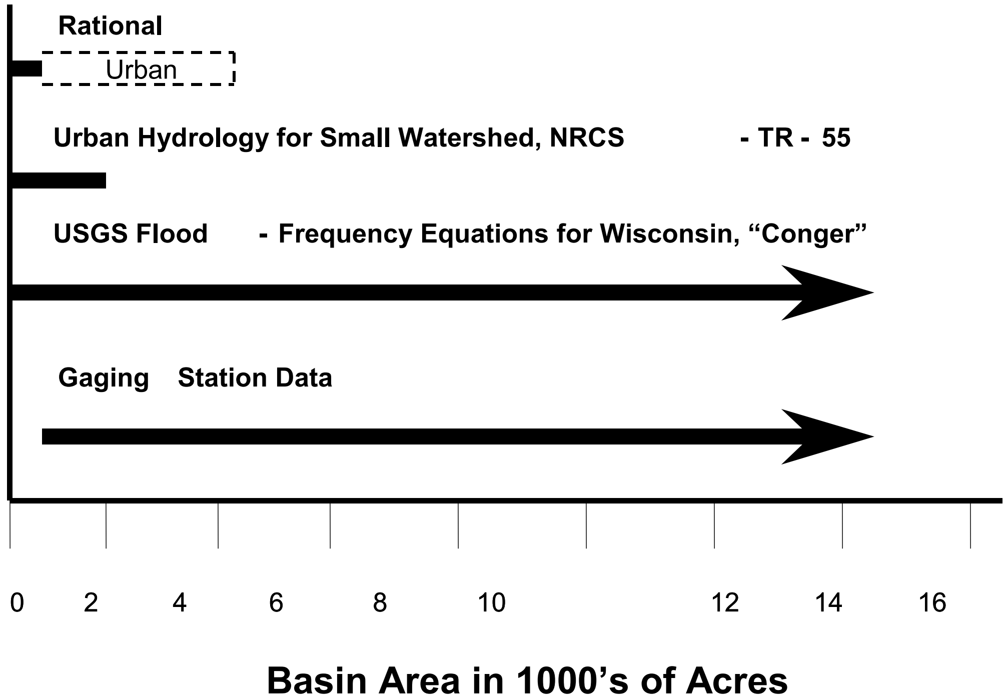


RISK OF FLOODLAND OCCUPANCY

House		Percent Probability of First Floor or More Damage in Any Year	Percent Probability of First Floor or More Damage Before 25–Year Mortgage is Paid
Identification	Position in Floodplain– First Floor Coincident With:		
A	5–Year Recurrence Interval Flood Stage	20.0	99.6
B	10–Year Recurrence Interval Flood Stage	10.0	92.8
C	25–Year Recurrence Interval Flood Stage	4.0	64.0
D	50–Year Recurrence Interval Flood Stage	2.0	40.0
E	100–Year Recurrence Interval Flood Stage	1.0	22.2

Source: SEWRPC.

Area Limits for Peak Discharge Methods



Detail A - Runoff Coefficients (C), Rational Formula

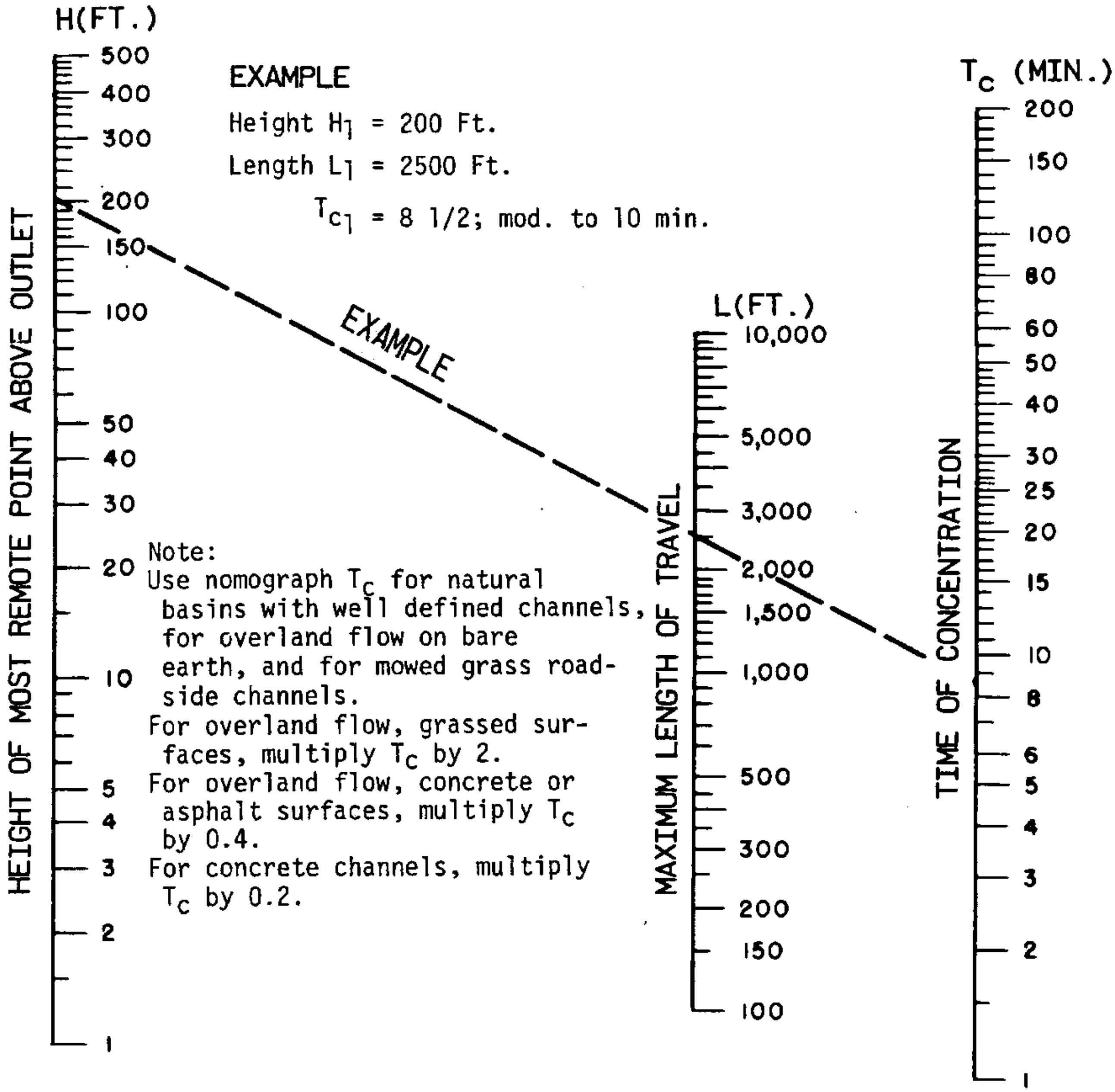
Land Use	Percent Impervious Area	Hydrologic Soil Group A, Slope Range Percent 0-2	A 2-6	A 6 & over	B 0-2	B 2-6	B 6 & over	C 0-2	C 2-6	C 6 & over	D 0-2	D 2-6	D 6 & over
Industrial	90	0.67 0.85	0.68 0.85	0.68 0.86	0.68 0.85	0.68 0.86	0.69 0.86	0.68 0.86	0.69 0.86	0.69 0.87	0.69 0.86	0.69 0.86	0.70 0.88
Commercial	95	0.71 0.88	0.71 0.89	0.72 0.89	0.71 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.90	0.72 0.89	0.72 0.89	0.72 0.90
High Density Residential	60	0.47 0.58	0.49 0.60	0.50 0.61	0.48 0.59	0.50 0.61	0.52 0.64	0.49 0.60	0.51 0.62	0.54 0.66	0.51 0.62	0.53 0.64	0.56 0.69
Med. Density Residential	30	0.25 0.33	0.28 0.37	0.31 0.40	0.27 0.35	0.30 0.39	0.35 0.44	0.30 0.38	0.33 0.42	0.38 0.49	0.33 0.41	0.36 0.45	0.42 0.54
Low Density Residential	15	0.14 0.22	0.19 0.26	0.22 0.29	0.17 0.24	0.21 0.28	0.26 0.34	0.20 0.28	0.25 0.32	0.31 0.40	0.24 0.31	0.28 0.35	0.35 0.46
Agriculture	5	0.08 0.14	0.13 0.18	0.16 0.22	0.11 0.16	0.15 0.21	0.21 0.28	0.14 0.20	0.19 0.25	0.26 0.34	0.18 0.24	0.23 0.29	0.31 0.41
Open Space	2	0.05 0.11	0.10 0.16	0.14 0.20	0.08 0.14	0.13 0.19	0.19 0.26	0.12 0.18	0.17 0.23	0.24 0.32	0.16 0.22	0.21 0.27	0.28 0.39
Freeways & Expressways	70	0.57 0.70	0.59 0.71	0.60 0.72	0.58 0.71	0.60 0.72	0.61 0.74	0.59 0.72	0.61 0.73	0.63 0.76	0.60 0.73	0.62 0.75	0.64 0.78

Detail B - Runoff Coefficients for Specific Land Use

Land Use	Hydrologic Soil Group A, Slope Range Percent 0-2	A 2-6	A 6 & over	B 0-2	B 2-6	B 6 & over	C 0-2	C 2-6	C 6 & over	D 0-2	D 2-6	D 6 & over
Row Crops	.08 .22	.16 .30	.22 .38	.12 .26	.20 .34	.27 .44	.15 .30	.24 .37	.33 .50	.19 .34	.28 .41	.38 .56
Median Stripturf	.19 .24	.20 .26	.24 .30	.19 .25	.22 .28	.26 .33	.20 .26	.23 .30	.30 .37	.20 .27	.25 .32	.30 .40
Side Slopeturf			.25 .32			.27 .34			.28 .36			.30 .38
PAVEMENT												
Asphalt	.70 - .95											
Concrete	.80 - .95											
Brick	.70 - .80											
Drives, Walks	.75 - .85											
Roofs	.75 - .95											
Gravel Roads Shoulders	.40 - .60											

NOTE: The lower C values in each range should be used with the relatively low intensities associated with 2 to 10-year design recurrence intervals whereas the higher C values should be used for intensities associated with the longer 25 to 100 year deign recurrence intervals.

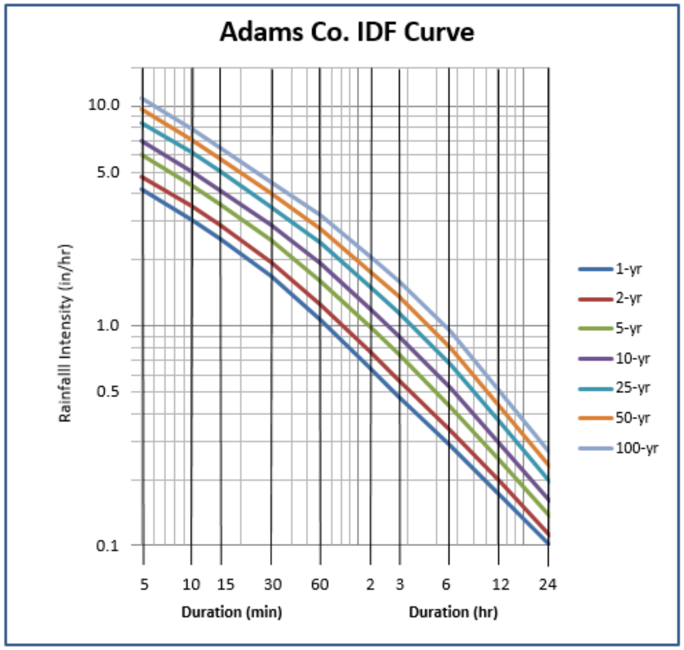
TIME OF CONCENTRATION OF SMALL
T_C
DRAINAGE BASINS



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p.362

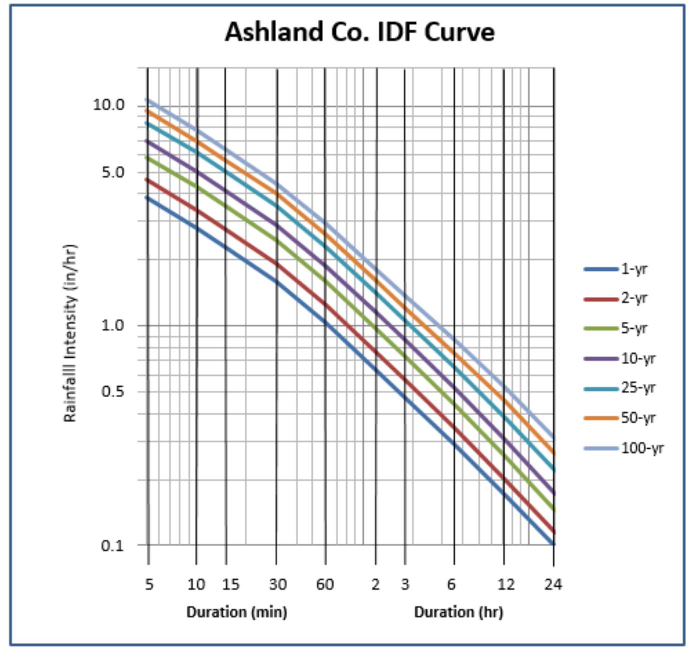
Adams County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.20	3.06	2.48	1.68	1.06	0.64	0.47	0.29	0.17	0.10
2-yr	4.80	3.54	2.88	1.96	1.25	0.77	0.57	0.34	0.20	0.11
5-yr	6.00	4.38	3.56	2.46	1.60	0.99	0.74	0.43	0.24	0.14
10-yr	6.96	5.10	4.16	2.88	1.92	1.20	0.90	0.52	0.29	0.16
25-yr	8.40	6.18	5.04	3.50	2.39	1.51	1.15	0.67	0.36	0.20
50-yr	9.72	7.08	5.76	4.02	2.78	1.78	1.37	0.80	0.43	0.23
100-yr	10.92	7.98	6.52	4.54	3.20	2.07	1.60	0.94	0.50	0.27



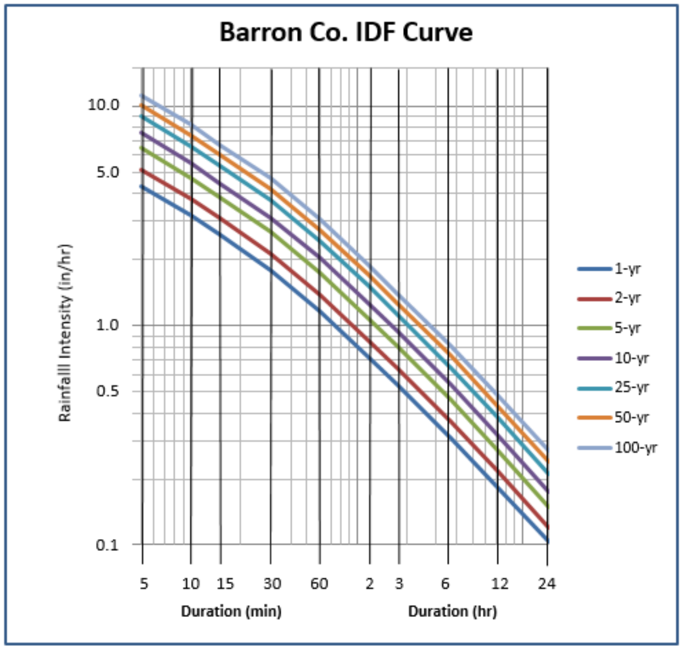
Ashland County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.84	2.82	2.28	1.60	1.03	0.64	0.48	0.29	0.17	0.10
2-yr	4.68	3.36	2.76	1.92	1.24	0.77	0.57	0.34	0.20	0.11
5-yr	5.88	4.32	3.52	2.46	1.59	0.98	0.73	0.43	0.25	0.14
10-yr	6.96	5.10	4.16	2.90	1.88	1.16	0.87	0.52	0.30	0.17
25-yr	8.40	6.18	5.04	3.52	2.29	1.42	1.06	0.64	0.38	0.22
50-yr	9.60	7.02	5.72	4.00	2.61	1.62	1.22	0.75	0.45	0.26
100-yr	10.80	7.86	6.40	4.48	2.94	1.82	1.38	0.86	0.52	0.31



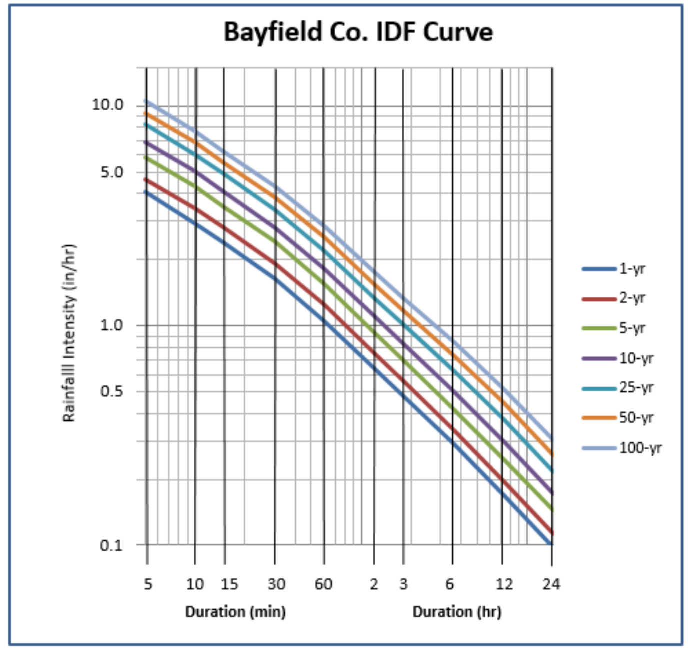
Barron County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.18	2.60	1.80	1.16	0.71	0.53	0.31	0.18	0.10
2-yr	5.16	3.78	3.08	2.12	1.38	0.85	0.63	0.37	0.21	0.12
5-yr	6.48	4.74	3.84	2.68	1.74	1.07	0.80	0.47	0.27	0.15
10-yr	7.56	5.52	4.48	3.12	2.03	1.25	0.93	0.55	0.31	0.17
25-yr	9.00	6.60	5.36	3.74	2.43	1.50	1.12	0.66	0.37	0.21
50-yr	10.20	7.44	6.04	4.22	2.74	1.69	1.25	0.74	0.42	0.24
100-yr	11.28	8.28	6.72	4.70	3.04	1.87	1.39	0.82	0.48	0.27



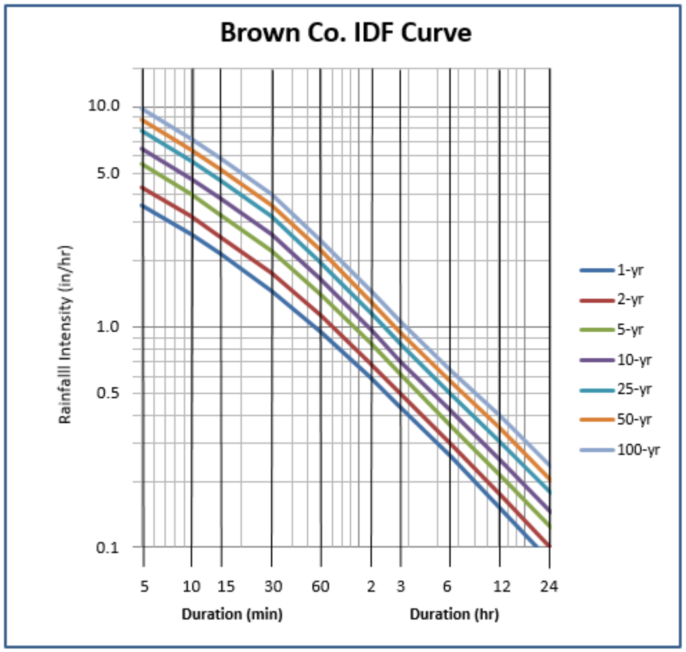
Bayfield County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.08	2.94	2.40	1.64	1.05	0.65	0.48	0.29	0.17	0.10
2-yr	4.68	3.42	2.80	1.92	1.24	0.76	0.56	0.33	0.20	0.11
5-yr	5.88	4.32	3.48	2.42	1.54	0.94	0.70	0.42	0.25	0.14
10-yr	6.84	5.04	4.08	2.82	1.81	1.11	0.83	0.50	0.30	0.17
25-yr	8.28	6.06	4.92	3.40	2.20	1.35	1.02	0.62	0.38	0.22
50-yr	9.36	6.84	5.56	3.86	2.52	1.55	1.18	0.73	0.44	0.26
100-yr	10.56	7.68	6.24	4.34	2.85	1.77	1.35	0.84	0.52	0.30



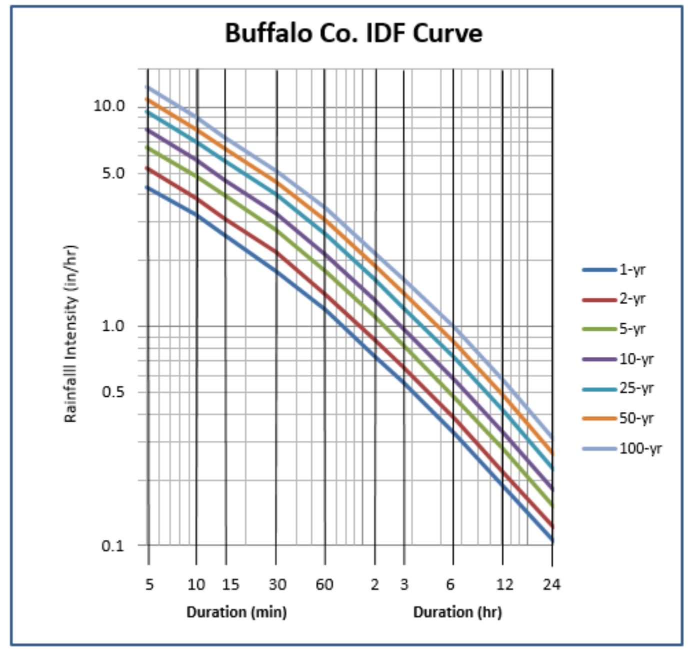
Brown County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.60	2.64	2.16	1.46	0.95	0.59	0.43	0.26	0.15	0.09
2-yr	4.32	3.18	2.56	1.76	1.12	0.68	0.50	0.29	0.17	0.10
5-yr	5.52	4.02	3.24	2.24	1.40	0.84	0.61	0.36	0.21	0.12
10-yr	6.48	4.74	3.84	2.64	1.63	0.98	0.70	0.41	0.25	0.14
25-yr	7.80	5.70	4.64	3.18	1.96	1.17	0.84	0.50	0.30	0.18
50-yr	8.76	6.42	5.24	3.60	2.21	1.31	0.95	0.56	0.34	0.20
100-yr	9.84	7.20	5.88	4.02	2.47	1.46	1.06	0.63	0.39	0.23



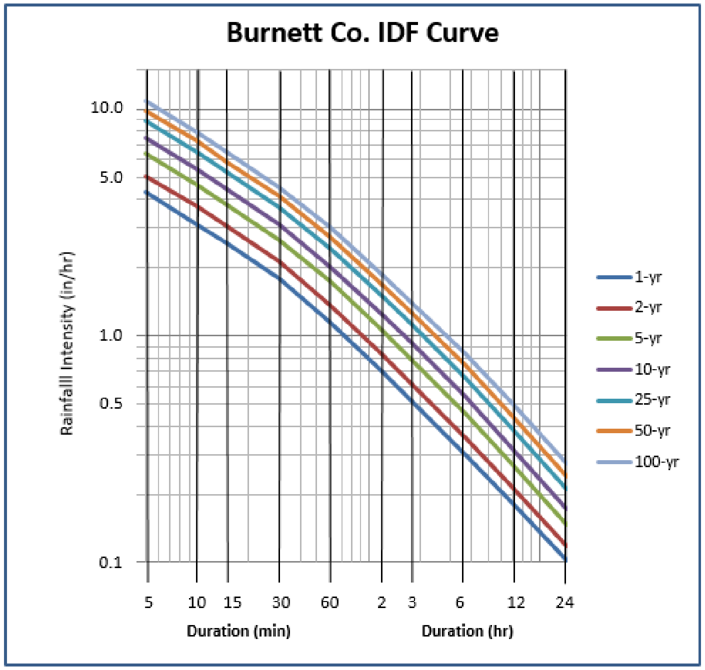
Buffalo County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.24	2.60	1.80	1.19	0.74	0.55	0.33	0.18	0.10
2-yr	5.28	3.84	3.12	2.18	1.41	0.87	0.65	0.38	0.22	0.12
5-yr	6.60	4.86	3.96	2.78	1.80	1.11	0.82	0.48	0.27	0.15
10-yr	7.92	5.76	4.68	3.30	2.14	1.32	0.97	0.57	0.33	0.18
25-yr	9.60	7.02	5.72	4.02	2.63	1.63	1.22	0.72	0.41	0.22
50-yr	10.92	7.98	6.52	4.58	3.04	1.90	1.43	0.85	0.48	0.26
100-yr	12.36	9.00	7.32	5.16	3.47	2.18	1.65	0.99	0.56	0.31



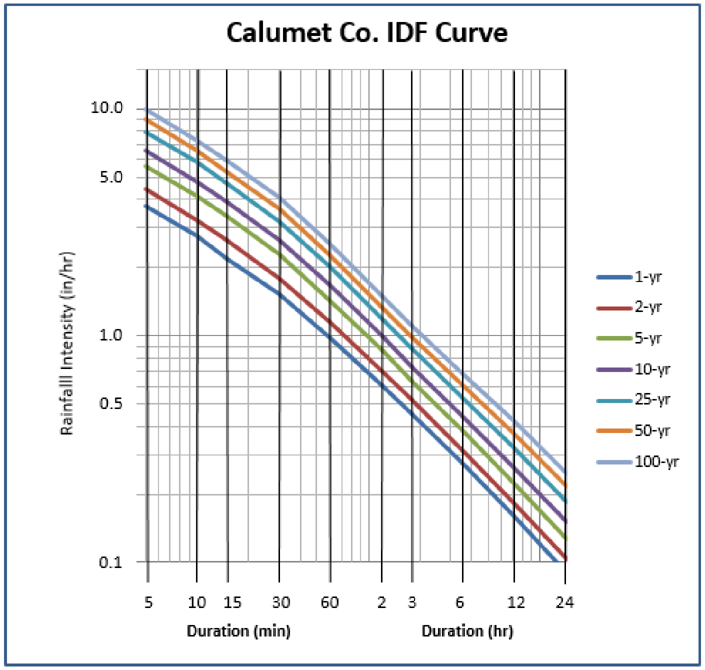
Burnett County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.12	2.56	1.78	1.14	0.70	0.51	0.30	0.18	0.10
2-yr	5.04	3.72	3.04	2.12	1.36	0.84	0.62	0.36	0.21	0.12
5-yr	6.36	4.68	3.80	2.66	1.73	1.06	0.79	0.46	0.26	0.15
10-yr	7.44	5.46	4.44	3.10	2.02	1.25	0.93	0.54	0.31	0.17
25-yr	8.88	6.48	5.28	3.68	2.42	1.50	1.12	0.66	0.38	0.21
50-yr	9.84	7.26	5.88	4.12	2.71	1.69	1.26	0.75	0.43	0.24
100-yr	10.92	7.98	6.52	4.54	3.00	1.87	1.41	0.84	0.49	0.27



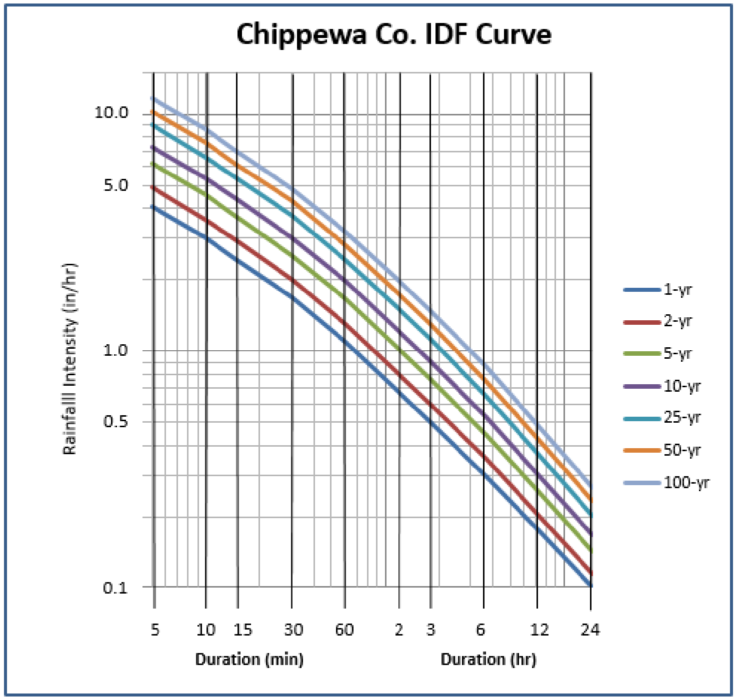
Calumet County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.72	2.76	2.20	1.52	0.98	0.61	0.45	0.27	0.16	0.09
2-yr	4.44	3.24	2.64	1.80	1.15	0.70	0.52	0.31	0.18	0.10
5-yr	5.64	4.14	3.36	2.28	1.43	0.87	0.63	0.38	0.22	0.13
10-yr	6.60	4.80	3.92	2.66	1.67	1.00	0.73	0.43	0.26	0.15
25-yr	7.92	5.82	4.72	3.20	2.00	1.20	0.88	0.52	0.32	0.19
50-yr	9.00	6.54	5.32	3.62	2.26	1.35	0.99	0.60	0.36	0.22
100-yr	9.96	7.32	5.96	4.06	2.52	1.51	1.11	0.67	0.42	0.25



Chippewa County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

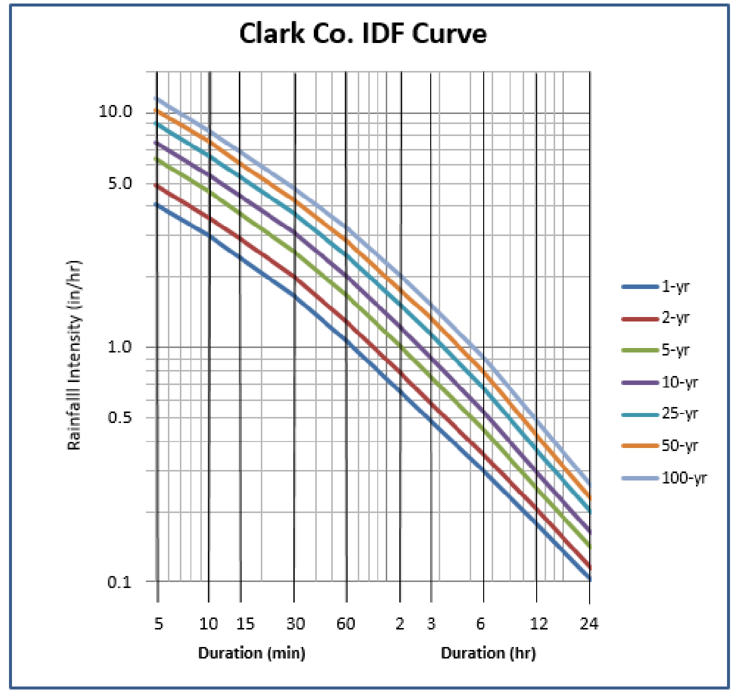
	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.08	3.00	2.44	1.68	1.09	0.67	0.50	0.30	0.18	0.10
2-yr	4.92	3.60	2.92	2.00	1.30	0.80	0.60	0.35	0.20	0.12
5-yr	6.24	4.56	3.68	2.54	1.66	1.02	0.76	0.45	0.25	0.14
10-yr	7.32	5.40	4.36	3.02	1.97	1.22	0.91	0.53	0.30	0.17
25-yr	9.00	6.60	5.36	3.72	2.43	1.51	1.12	0.65	0.37	0.20
50-yr	10.32	7.56	6.16	4.30	2.81	1.74	1.30	0.76	0.42	0.23
100-yr	11.76	8.64	7.00	4.88	3.21	1.99	1.48	0.87	0.48	0.27



Chippewa Co. IDF Curve

Clark County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

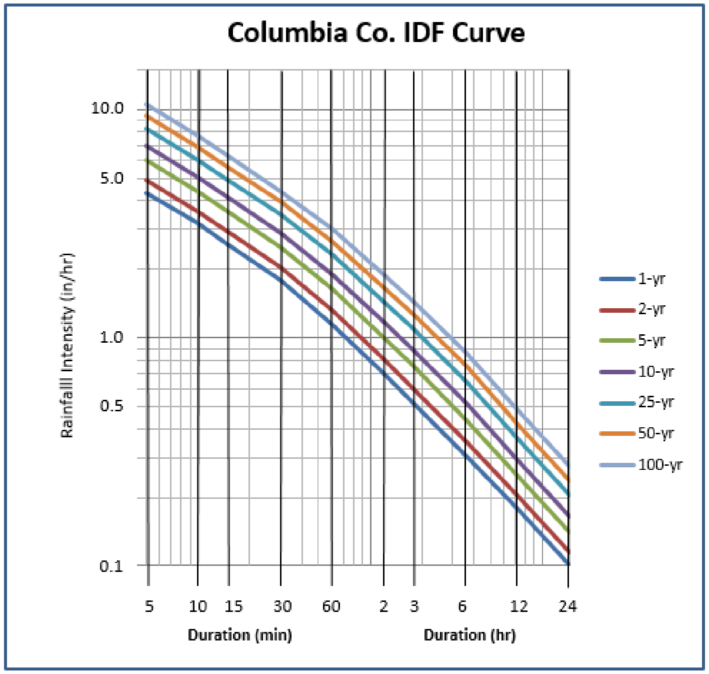
	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.08	3.00	2.44	1.66	1.06	0.65	0.49	0.29	0.18	0.10
2-yr	4.92	3.60	2.92	2.02	1.29	0.79	0.58	0.35	0.20	0.12
5-yr	6.36	4.62	3.76	2.58	1.67	1.02	0.76	0.44	0.25	0.14
10-yr	7.44	5.46	4.44	3.08	2.00	1.23	0.91	0.53	0.29	0.16
25-yr	9.00	6.60	5.40	3.74	2.47	1.53	1.14	0.66	0.36	0.20
50-yr	10.32	7.56	6.12	4.26	2.84	1.78	1.33	0.77	0.42	0.23
100-yr	11.52	8.46	6.88	4.80	3.23	2.03	1.53	0.89	0.48	0.26



Clark Co. IDF Curve

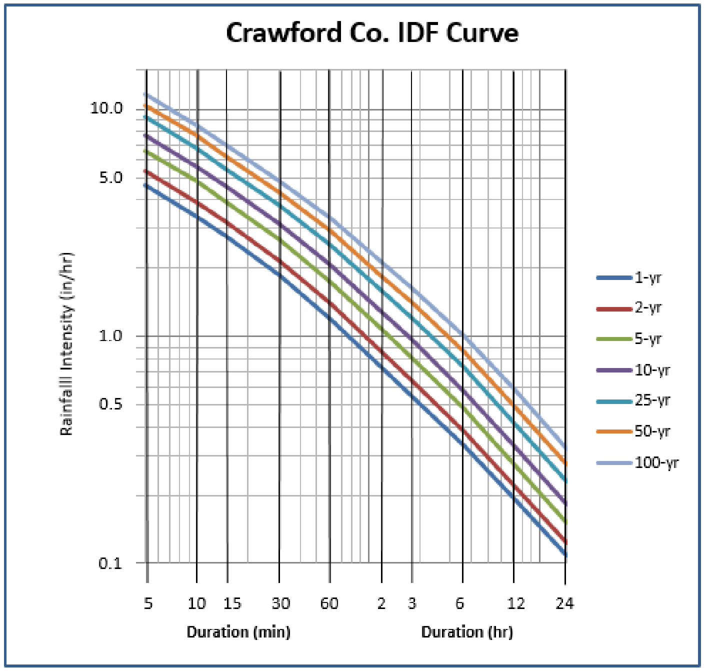
Columbia County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.18	2.56	1.80	1.15	0.70	0.51	0.30	0.18	0.10
2-yr	4.92	3.60	2.92	2.04	1.32	0.81	0.60	0.35	0.20	0.12
5-yr	6.00	4.38	3.56	2.50	1.63	1.01	0.75	0.44	0.25	0.14
10-yr	6.96	5.10	4.12	2.88	1.91	1.19	0.89	0.52	0.29	0.17
25-yr	8.28	6.06	4.96	3.46	2.31	1.45	1.09	0.64	0.36	0.20
50-yr	9.48	6.90	5.60	3.94	2.65	1.67	1.27	0.75	0.42	0.24
100-yr	10.56	7.74	6.32	4.42	3.00	1.90	1.45	0.86	0.48	0.27



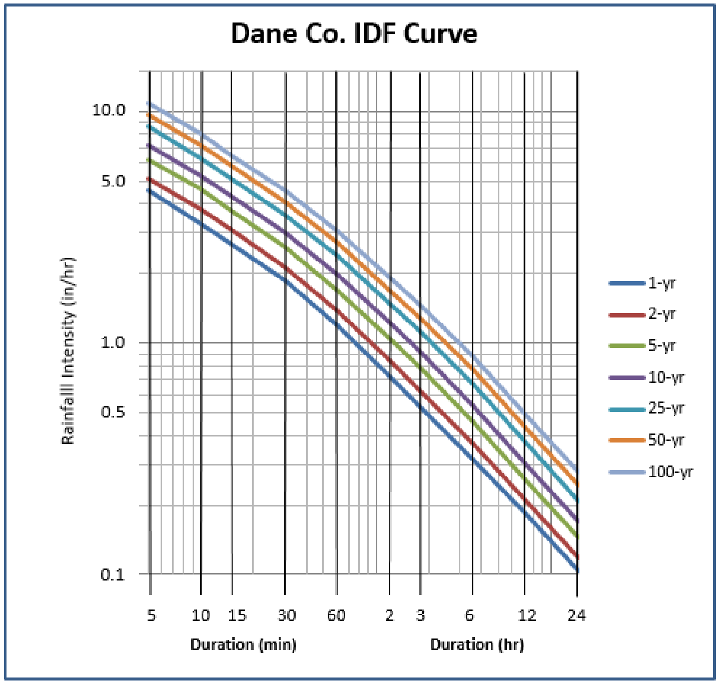
Crawford County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.68	3.36	2.76	1.86	1.19	0.73	0.55	0.33	0.19	0.11
2-yr	5.40	3.90	3.20	2.16	1.40	0.86	0.64	0.38	0.22	0.12
5-yr	6.60	4.86	3.92	2.68	1.75	1.08	0.81	0.48	0.27	0.15
10-yr	7.68	5.64	4.60	3.14	2.07	1.29	0.97	0.57	0.32	0.18
25-yr	9.24	6.78	5.48	3.78	2.54	1.59	1.21	0.73	0.41	0.23
50-yr	10.44	7.68	6.24	4.30	2.92	1.85	1.42	0.86	0.49	0.27
100-yr	11.76	8.58	7.00	4.84	3.33	2.12	1.65	1.01	0.58	0.32



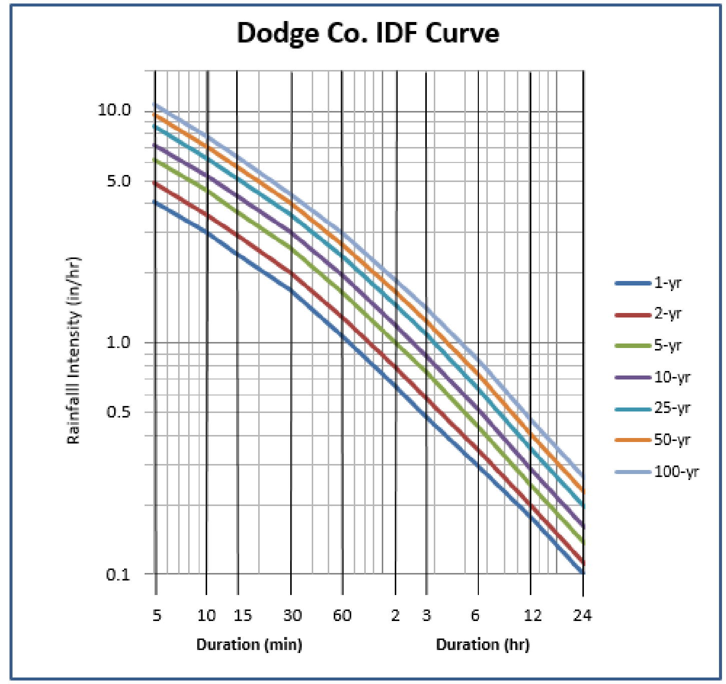
Dane County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.56	3.30	2.68	1.88	1.19	0.72	0.53	0.31	0.18	0.10
2-yr	5.16	3.78	3.08	2.14	1.38	0.84	0.62	0.36	0.21	0.12
5-yr	6.24	4.62	3.72	2.60	1.70	1.05	0.78	0.45	0.26	0.15
10-yr	7.20	5.28	4.32	3.00	1.98	1.23	0.92	0.54	0.30	0.17
25-yr	8.64	6.30	5.16	3.60	2.38	1.49	1.12	0.66	0.37	0.21
50-yr	9.72	7.14	5.84	4.06	2.71	1.70	1.29	0.76	0.43	0.24
100-yr	10.92	8.04	6.52	4.56	3.06	1.92	1.46	0.87	0.49	0.28



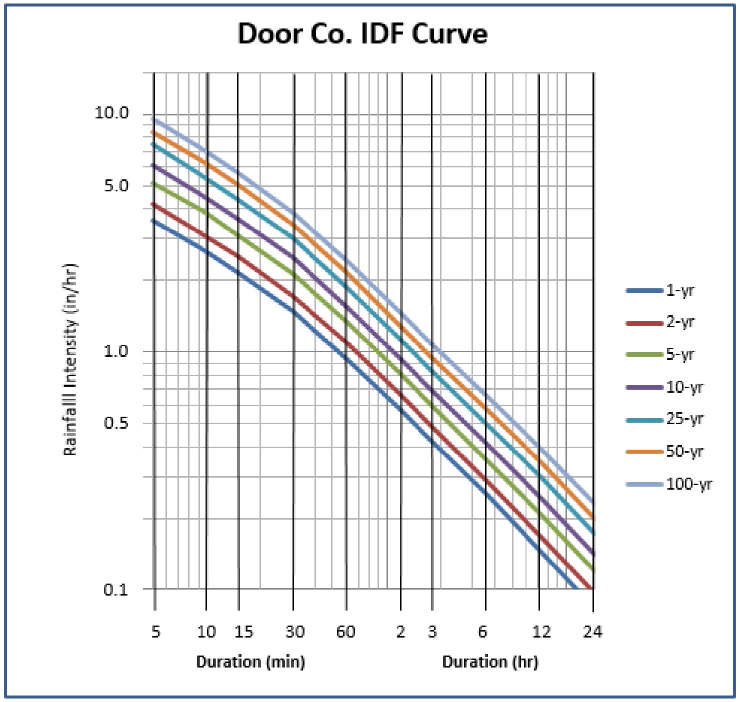
Dodge County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.08	3.00	2.44	1.68	1.07	0.65	0.48	0.29	0.17	0.10
2-yr	4.92	3.60	2.92	2.02	1.29	0.79	0.58	0.34	0.20	0.11
5-yr	6.24	4.56	3.68	2.56	1.65	1.01	0.75	0.43	0.24	0.14
10-yr	7.20	5.28	4.32	3.00	1.95	1.20	0.89	0.51	0.28	0.16
25-yr	8.64	6.30	5.12	3.58	2.35	1.46	1.09	0.63	0.34	0.19
50-yr	9.72	7.08	5.76	4.00	2.66	1.66	1.25	0.73	0.40	0.23
100-yr	10.68	7.80	6.36	4.42	2.97	1.87	1.41	0.83	0.46	0.26



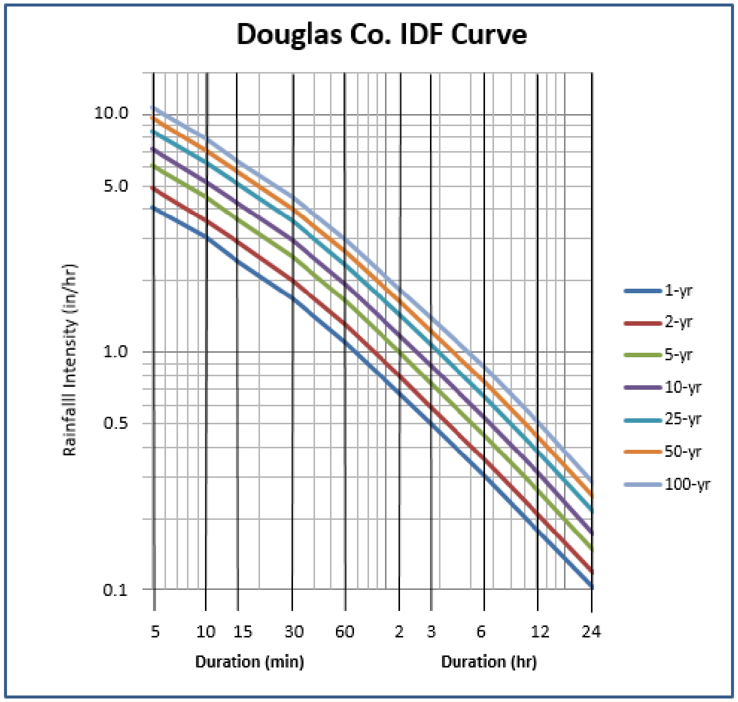
Door County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.60	2.64	2.16	1.48	0.94	0.58	0.42	0.25	0.14	0.08
2-yr	4.20	3.06	2.52	1.72	1.09	0.66	0.49	0.29	0.17	0.10
5-yr	5.16	3.84	3.12	2.12	1.34	0.81	0.59	0.35	0.21	0.12
10-yr	6.12	4.44	3.64	2.48	1.56	0.94	0.69	0.41	0.24	0.14
25-yr	7.44	5.40	4.40	3.00	1.88	1.13	0.83	0.50	0.30	0.17
50-yr	8.40	6.18	5.04	3.44	2.15	1.29	0.95	0.57	0.34	0.20
100-yr	9.60	6.96	5.68	3.88	2.43	1.46	1.08	0.65	0.39	0.23



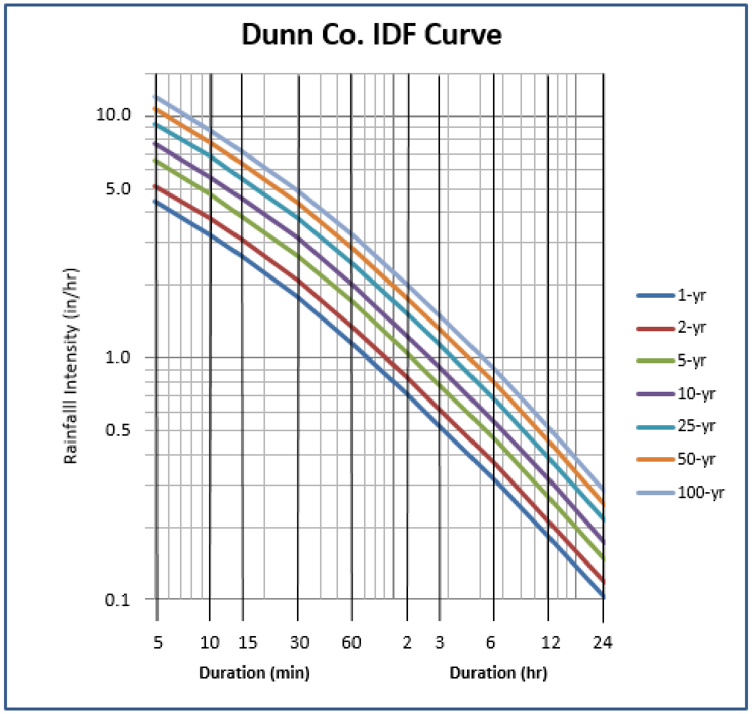
Douglas County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.08	3.06	2.44	1.70	1.09	0.67	0.50	0.30	0.17	0.10
2-yr	4.92	3.60	2.92	2.02	1.30	0.80	0.59	0.35	0.20	0.12
5-yr	6.12	4.50	3.64	2.54	1.64	1.00	0.74	0.44	0.26	0.15
10-yr	7.20	5.22	4.24	2.98	1.93	1.18	0.88	0.52	0.31	0.17
25-yr	8.52	6.30	5.12	3.58	2.33	1.44	1.08	0.65	0.38	0.21
50-yr	9.72	7.08	5.76	4.04	2.65	1.65	1.24	0.75	0.44	0.25
100-yr	10.80	7.92	6.40	4.50	2.98	1.86	1.40	0.85	0.50	0.28



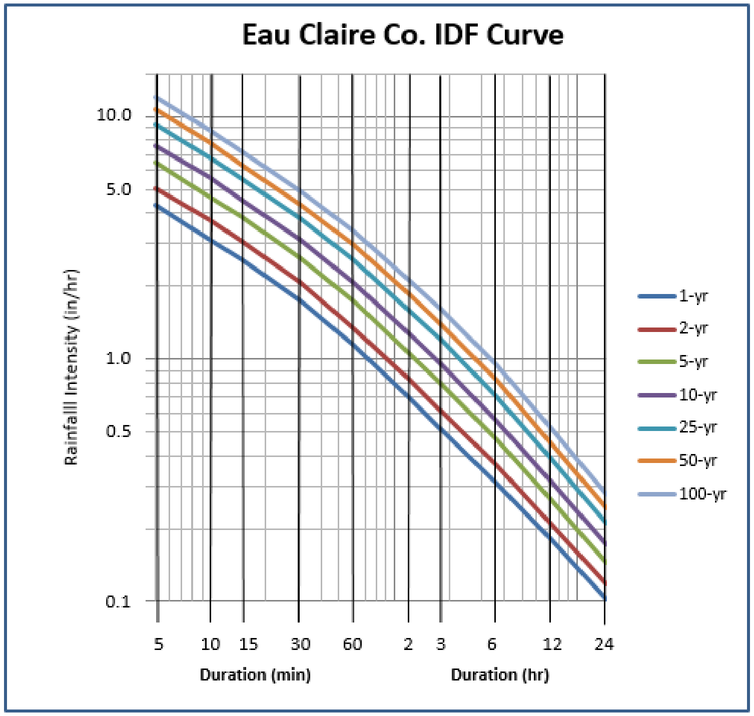
Dunn County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.44	3.24	2.64	1.78	1.15	0.71	0.53	0.31	0.18	0.10
2-yr	5.16	3.78	3.08	2.10	1.35	0.83	0.62	0.36	0.21	0.12
5-yr	6.60	4.80	3.88	2.66	1.71	1.05	0.78	0.46	0.26	0.15
10-yr	7.68	5.64	4.60	3.14	2.02	1.24	0.92	0.54	0.31	0.17
25-yr	9.36	6.84	5.56	3.82	2.47	1.52	1.14	0.67	0.38	0.21
50-yr	10.68	7.80	6.36	4.36	2.85	1.76	1.32	0.78	0.45	0.25
100-yr	12.00	8.82	7.16	4.94	3.24	2.01	1.51	0.90	0.51	0.28



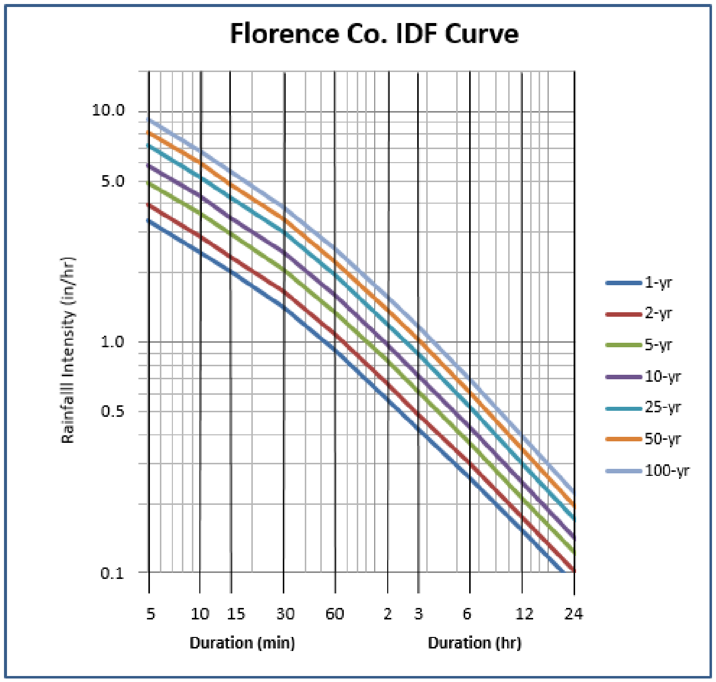
Eau Claire County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.12	2.56	1.76	1.14	0.70	0.52	0.31	0.18	0.10
2-yr	5.04	3.72	3.04	2.10	1.35	0.83	0.62	0.36	0.21	0.12
5-yr	6.48	4.68	3.84	2.66	1.74	1.07	0.80	0.47	0.26	0.15
10-yr	7.56	5.58	4.52	3.14	2.07	1.29	0.96	0.56	0.31	0.17
25-yr	9.24	6.78	5.52	3.86	2.56	1.60	1.20	0.70	0.39	0.21
50-yr	10.68	7.80	6.32	4.42	2.97	1.86	1.40	0.82	0.45	0.24
100-yr	12.00	8.82	7.16	5.02	3.39	2.14	1.62	0.95	0.52	0.28



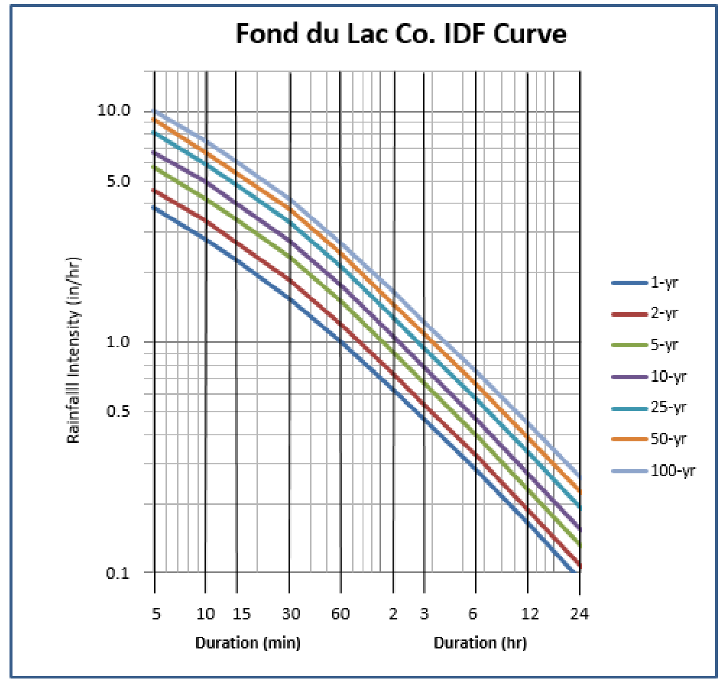
Florence County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.36	2.46	2.04	1.42	0.92	0.57	0.42	0.25	0.15	0.09
2-yr	3.96	2.88	2.36	1.66	1.08	0.66	0.49	0.29	0.17	0.10
5-yr	4.92	3.66	2.96	2.08	1.35	0.83	0.61	0.36	0.21	0.12
10-yr	5.88	4.32	3.48	2.46	1.59	0.98	0.72	0.42	0.24	0.14
25-yr	7.20	5.22	4.24	3.00	1.94	1.20	0.89	0.52	0.29	0.17
50-yr	8.16	6.00	4.88	3.42	2.23	1.38	1.03	0.60	0.34	0.19
100-yr	9.24	6.78	5.52	3.88	2.54	1.57	1.17	0.68	0.39	0.22



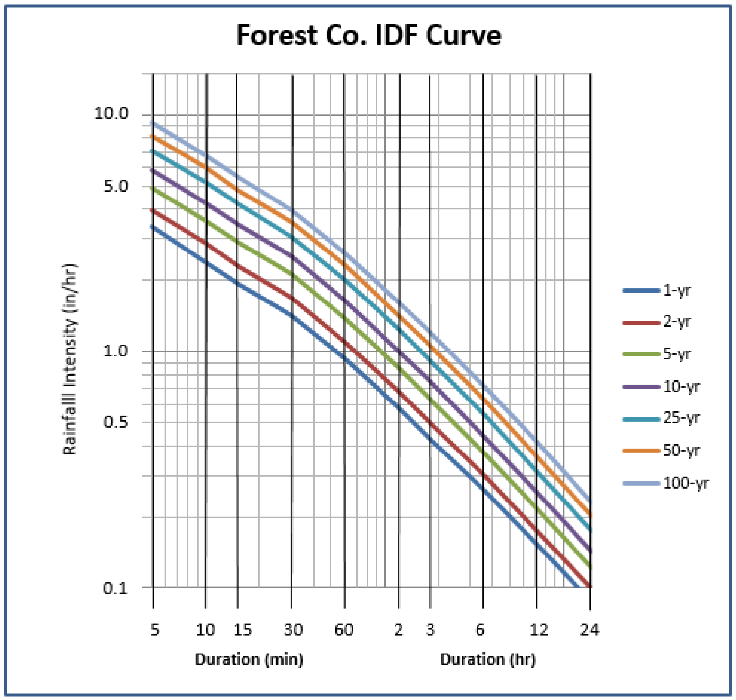
Fond du Lac County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.84	2.82	2.28	1.56	1.01	0.62	0.46	0.28	0.16	0.09
2-yr	4.56	3.36	2.72	1.88	1.20	0.73	0.54	0.32	0.19	0.11
5-yr	5.76	4.20	3.44	2.36	1.50	0.91	0.67	0.39	0.23	0.13
10-yr	6.72	4.98	4.04	2.78	1.76	1.07	0.79	0.46	0.27	0.15
25-yr	8.16	5.94	4.84	3.34	2.13	1.29	0.95	0.56	0.33	0.19
50-yr	9.24	6.72	5.48	3.78	2.41	1.47	1.09	0.65	0.38	0.22
100-yr	10.20	7.50	6.12	4.20	2.70	1.66	1.24	0.74	0.44	0.26



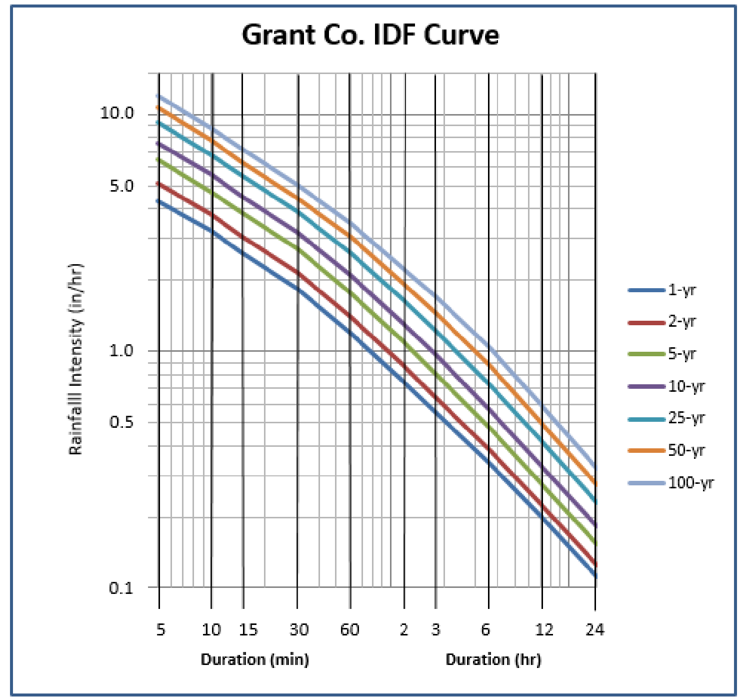
Forest County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.36	2.40	1.96	1.42	0.93	0.58	0.43	0.26	0.15	0.09
2-yr	3.96	2.88	2.32	1.68	1.10	0.68	0.50	0.30	0.17	0.10
5-yr	4.92	3.60	2.92	2.12	1.39	0.86	0.63	0.37	0.21	0.12
10-yr	5.88	4.26	3.48	2.52	1.64	1.01	0.75	0.44	0.25	0.14
25-yr	7.08	5.22	4.24	3.06	2.01	1.24	0.92	0.54	0.31	0.17
50-yr	8.16	6.00	4.88	3.52	2.31	1.43	1.06	0.62	0.35	0.20
100-yr	9.24	6.78	5.52	3.98	2.62	1.63	1.21	0.71	0.41	0.23



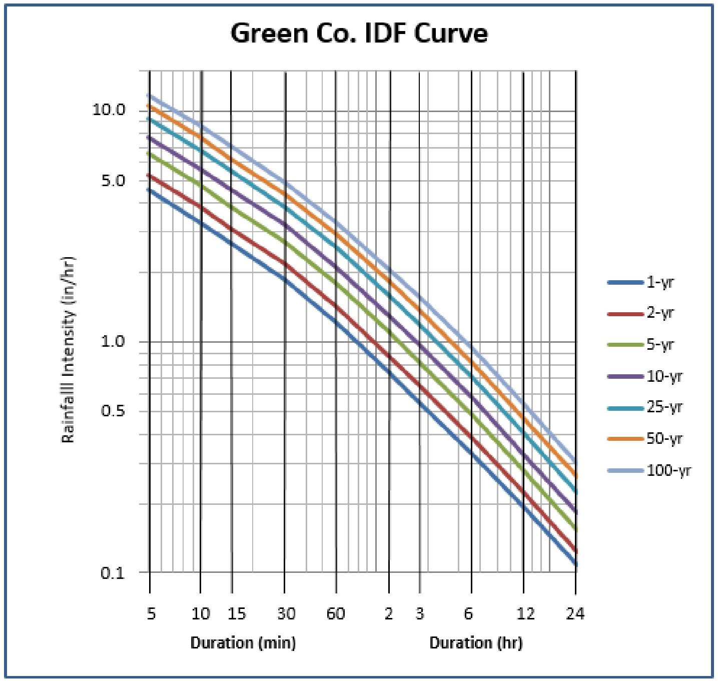
Grant County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.24	2.60	1.84	1.20	0.74	0.55	0.33	0.20	0.11
2-yr	5.16	3.78	3.04	2.16	1.40	0.87	0.64	0.38	0.22	0.13
5-yr	6.48	4.74	3.84	2.72	1.77	1.09	0.81	0.47	0.27	0.15
10-yr	7.56	5.58	4.52	3.20	2.10	1.30	0.97	0.57	0.32	0.18
25-yr	9.24	6.78	5.52	3.90	2.61	1.63	1.23	0.73	0.41	0.23
50-yr	10.68	7.80	6.32	4.48	3.04	1.92	1.46	0.87	0.49	0.27
100-yr	12.12	8.82	7.20	5.10	3.50	2.23	1.71	1.03	0.58	0.32



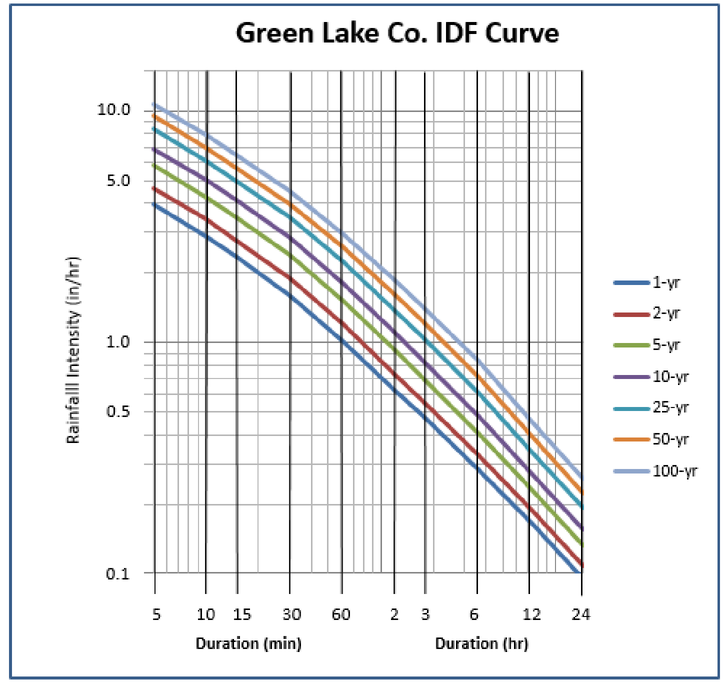
Green County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.56	3.30	2.68	1.86	1.21	0.74	0.55	0.33	0.19	0.11
2-yr	5.28	3.84	3.12	2.20	1.43	0.88	0.65	0.38	0.22	0.12
5-yr	6.60	4.80	3.88	2.74	1.79	1.11	0.82	0.48	0.27	0.15
10-yr	7.68	5.64	4.56	3.22	2.11	1.31	0.98	0.57	0.32	0.18
25-yr	9.24	6.78	5.52	3.88	2.57	1.60	1.20	0.70	0.40	0.22
50-yr	10.56	7.68	6.24	4.42	2.93	1.83	1.38	0.81	0.46	0.26
100-yr	11.76	8.64	7.04	4.96	3.31	2.07	1.56	0.93	0.53	0.30



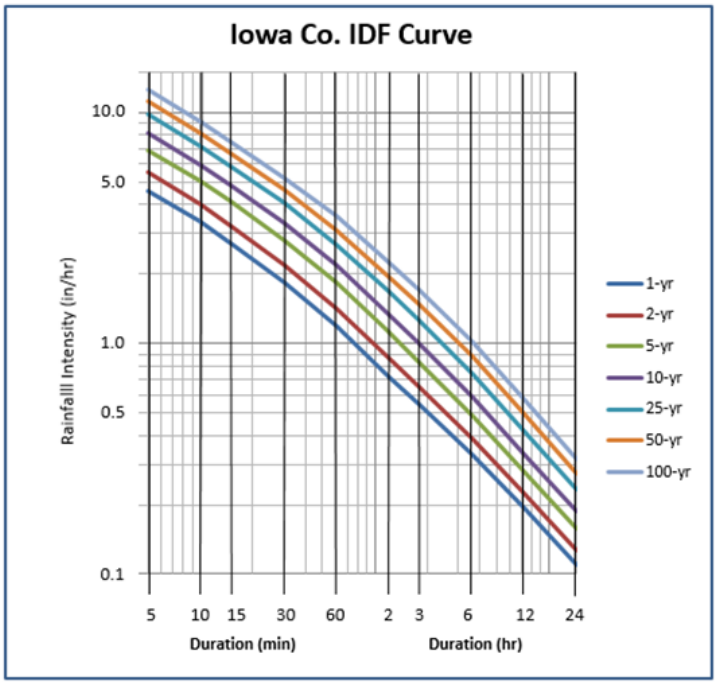
Green Lake County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.96	2.88	2.36	1.60	1.02	0.63	0.47	0.28	0.17	0.10
2-yr	4.68	3.42	2.76	1.90	1.21	0.74	0.55	0.32	0.19	0.11
5-yr	5.88	4.26	3.48	2.40	1.53	0.93	0.69	0.40	0.23	0.13
10-yr	6.84	5.04	4.12	2.84	1.82	1.11	0.82	0.48	0.27	0.16
25-yr	8.40	6.12	5.00	3.46	2.25	1.39	1.03	0.60	0.34	0.19
50-yr	9.60	7.02	5.72	3.96	2.60	1.62	1.21	0.71	0.40	0.22
100-yr	10.80	7.92	6.44	4.50	2.98	1.86	1.40	0.83	0.46	0.26



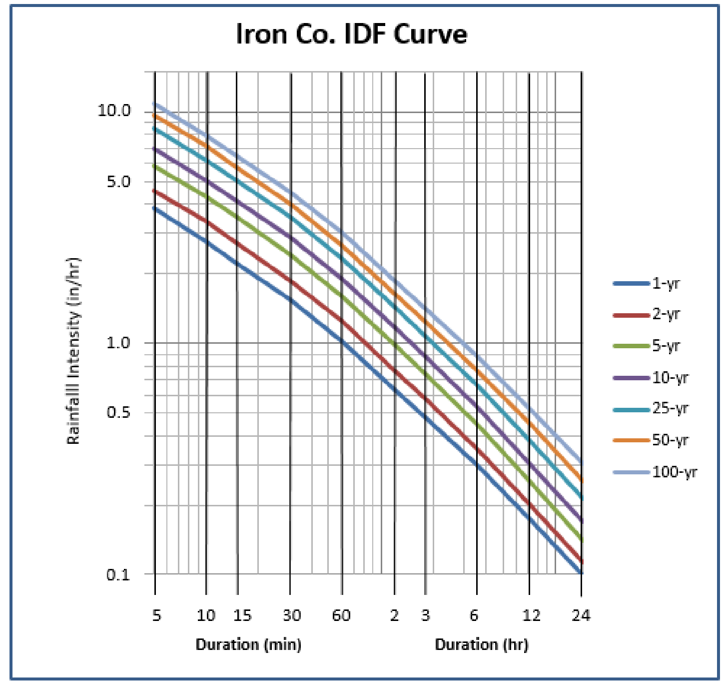
Iowa County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.56	3.36	2.72	1.84	1.19	0.73	0.55	0.33	0.19	0.11
2-yr	5.52	4.02	3.24	2.20	1.42	0.87	0.65	0.39	0.22	0.13
5-yr	6.84	5.04	4.12	2.82	1.83	1.12	0.84	0.49	0.28	0.16
10-yr	8.16	5.94	4.84	3.34	2.18	1.35	1.01	0.59	0.33	0.19
25-yr	9.84	7.20	5.88	4.08	2.70	1.68	1.26	0.74	0.42	0.23
50-yr	11.16	8.22	6.68	4.66	3.12	1.96	1.48	0.88	0.49	0.27
100-yr	12.60	9.24	7.52	5.24	3.56	2.25	1.71	1.02	0.57	0.32



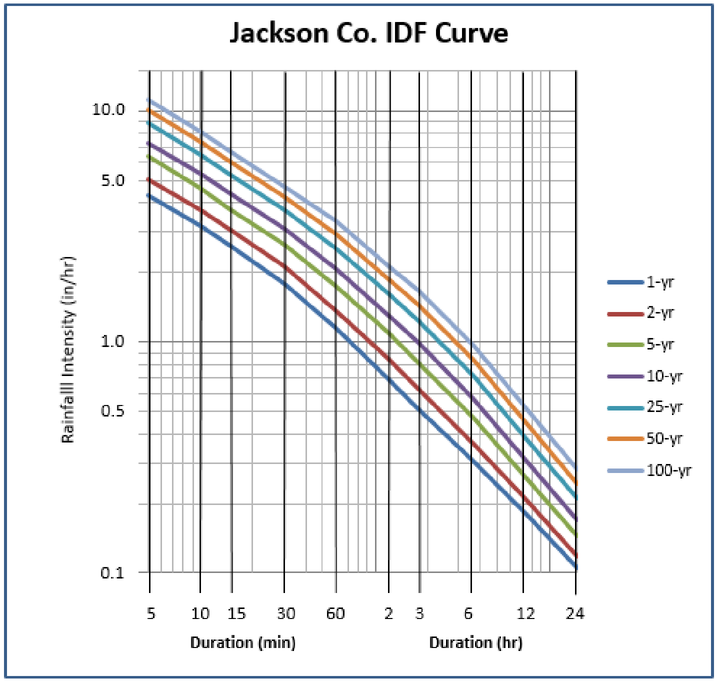
Iron County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.84	2.76	2.24	1.54	1.02	0.64	0.48	0.29	0.17	0.10
2-yr	4.56	3.36	2.72	1.88	1.24	0.77	0.58	0.35	0.20	0.11
5-yr	5.88	4.32	3.52	2.44	1.59	0.99	0.74	0.44	0.25	0.14
10-yr	6.96	5.10	4.16	2.90	1.90	1.18	0.88	0.52	0.30	0.17
25-yr	8.52	6.24	5.08	3.54	2.33	1.44	1.08	0.65	0.38	0.21
50-yr	9.72	7.14	5.80	4.04	2.66	1.65	1.25	0.76	0.45	0.26
100-yr	10.92	7.98	6.48	4.54	3.00	1.87	1.42	0.87	0.52	0.30



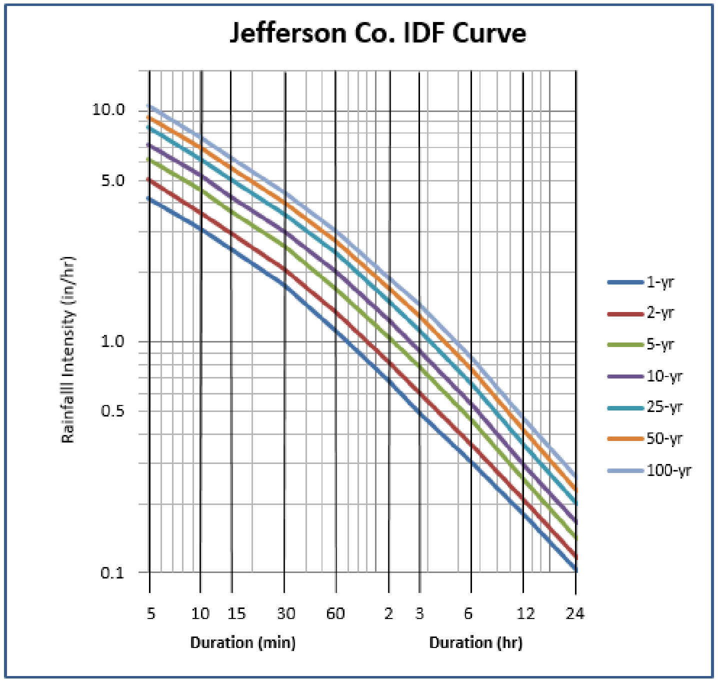
Jackson County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.32	3.18	2.60	1.80	1.14	0.69	0.51	0.31	0.18	0.11
2-yr	5.04	3.72	3.04	2.12	1.37	0.84	0.62	0.36	0.21	0.12
5-yr	6.36	4.62	3.76	2.66	1.75	1.09	0.81	0.47	0.26	0.15
10-yr	7.32	5.40	4.40	3.10	2.08	1.31	0.98	0.57	0.31	0.17
25-yr	8.88	6.48	5.28	3.74	2.55	1.62	1.23	0.72	0.39	0.21
50-yr	10.08	7.38	6.00	4.24	2.93	1.88	1.44	0.84	0.45	0.24
100-yr	11.28	8.22	6.68	4.74	3.32	2.14	1.66	0.98	0.52	0.28



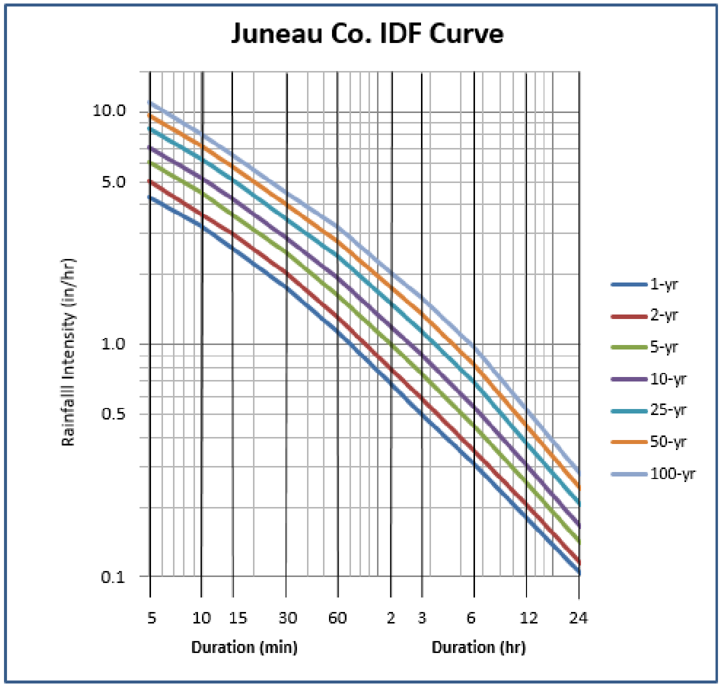
Jefferson County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.20	3.12	2.52	1.76	1.11	0.68	0.50	0.30	0.18	0.10
2-yr	5.04	3.66	2.96	2.08	1.34	0.82	0.60	0.35	0.20	0.12
5-yr	6.24	4.56	3.68	2.60	1.70	1.05	0.78	0.45	0.25	0.14
10-yr	7.20	5.28	4.28	3.02	2.00	1.25	0.93	0.53	0.29	0.16
25-yr	8.52	6.24	5.08	3.58	2.41	1.51	1.13	0.65	0.35	0.20
50-yr	9.48	6.96	5.68	4.02	2.71	1.71	1.29	0.75	0.41	0.23
100-yr	10.56	7.74	6.28	4.44	3.02	1.91	1.45	0.85	0.46	0.26



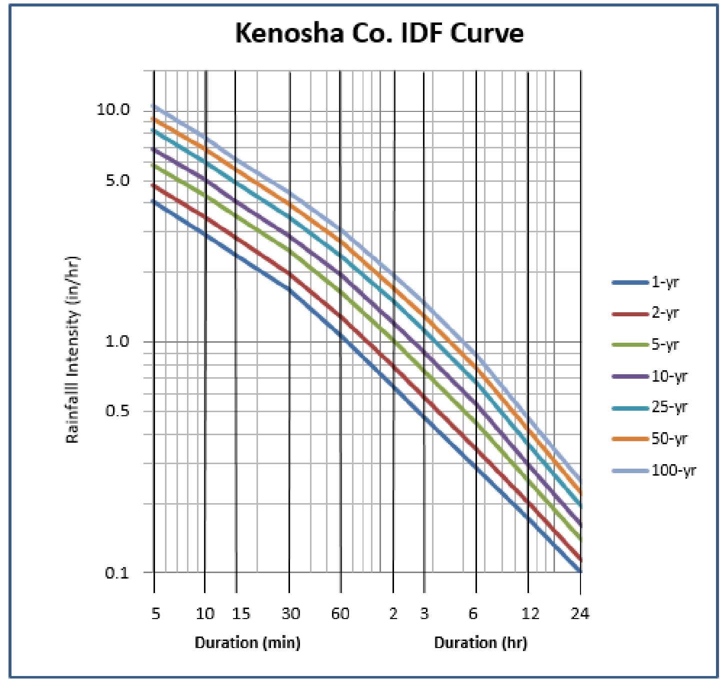
Juneau County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.24	2.60	1.76	1.12	0.68	0.50	0.30	0.18	0.10
2-yr	5.04	3.66	3.00	2.04	1.30	0.79	0.59	0.35	0.20	0.12
5-yr	6.12	4.50	3.64	2.50	1.62	1.00	0.75	0.44	0.25	0.14
10-yr	7.08	5.22	4.24	2.90	1.92	1.20	0.90	0.53	0.30	0.17
25-yr	8.52	6.30	5.12	3.50	2.38	1.50	1.15	0.68	0.37	0.20
50-yr	9.72	7.14	5.84	4.00	2.76	1.76	1.36	0.81	0.44	0.24
100-yr	11.04	8.10	6.56	4.52	3.18	2.05	1.60	0.96	0.52	0.28



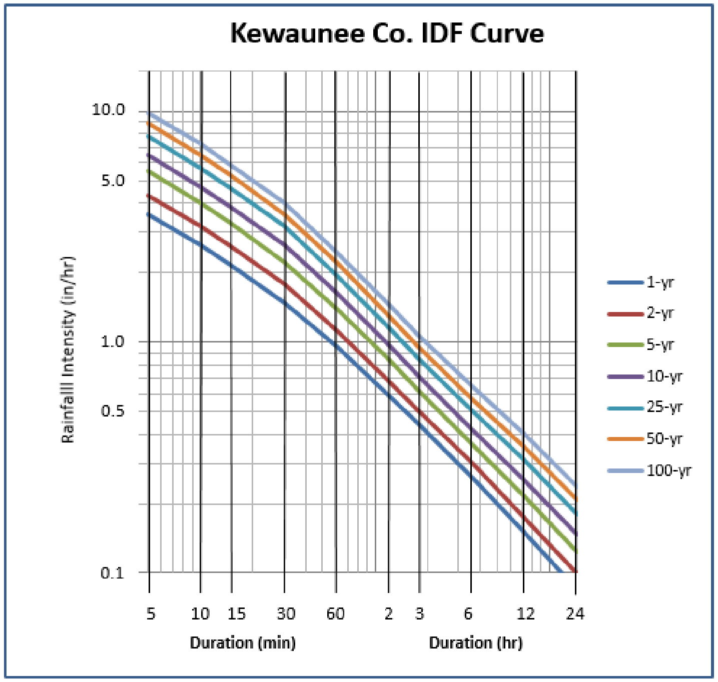
Kenosha County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.08	2.94	2.40	1.70	1.06	0.64	0.47	0.28	0.17	0.10
2-yr	4.80	3.48	2.84	1.98	1.28	0.78	0.58	0.34	0.20	0.11
5-yr	5.88	4.32	3.52	2.48	1.63	1.02	0.76	0.44	0.25	0.14
10-yr	6.84	5.04	4.08	2.90	1.94	1.22	0.91	0.53	0.29	0.16
25-yr	8.28	6.06	4.92	3.48	2.37	1.50	1.13	0.66	0.35	0.19
50-yr	9.36	6.90	5.60	3.96	2.71	1.72	1.31	0.76	0.41	0.22
100-yr	10.56	7.68	6.24	4.44	3.06	1.95	1.49	0.87	0.46	0.25



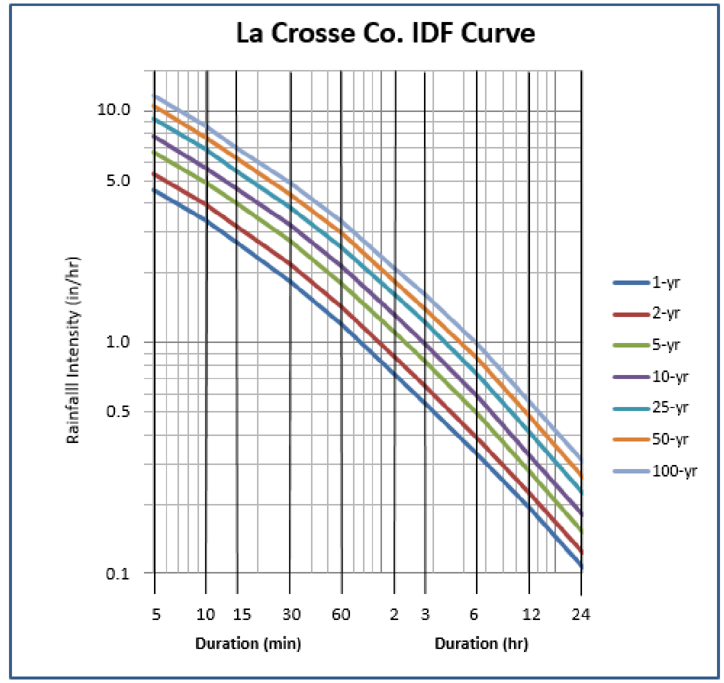
Kewaunee County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.60	2.64	2.16	1.48	0.96	0.59	0.44	0.26	0.15	0.08
2-yr	4.32	3.18	2.60	1.78	1.12	0.68	0.50	0.30	0.17	0.10
5-yr	5.52	4.02	3.28	2.24	1.40	0.84	0.61	0.36	0.21	0.12
10-yr	6.48	4.74	3.84	2.64	1.64	0.98	0.71	0.42	0.25	0.15
25-yr	7.80	5.70	4.64	3.18	1.96	1.16	0.84	0.50	0.31	0.18
50-yr	8.88	6.48	5.28	3.60	2.21	1.31	0.95	0.57	0.35	0.21
100-yr	9.84	7.26	5.88	4.02	2.46	1.46	1.06	0.64	0.40	0.24



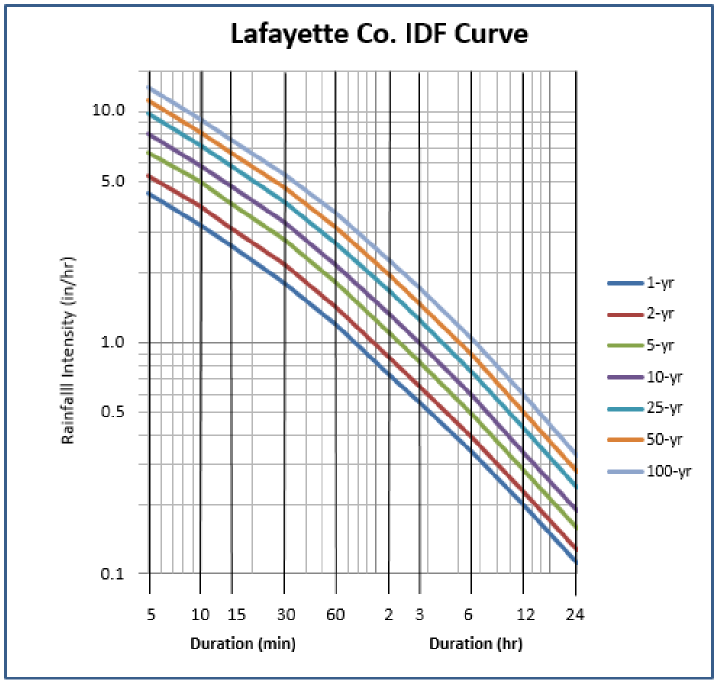
La Crosse County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.56	3.36	2.72	1.84	1.19	0.73	0.55	0.33	0.19	0.11
2-yr	5.40	3.96	3.20	2.18	1.42	0.87	0.65	0.38	0.22	0.12
5-yr	6.72	4.92	4.00	2.76	1.80	1.11	0.83	0.49	0.27	0.15
10-yr	7.80	5.70	4.64	3.22	2.12	1.32	0.99	0.58	0.32	0.18
25-yr	9.36	6.84	5.56	3.88	2.58	1.62	1.22	0.72	0.40	0.22
50-yr	10.56	7.74	6.28	4.38	2.95	1.86	1.41	0.84	0.47	0.26
100-yr	11.76	8.64	7.00	4.90	3.33	2.10	1.61	0.97	0.55	0.30



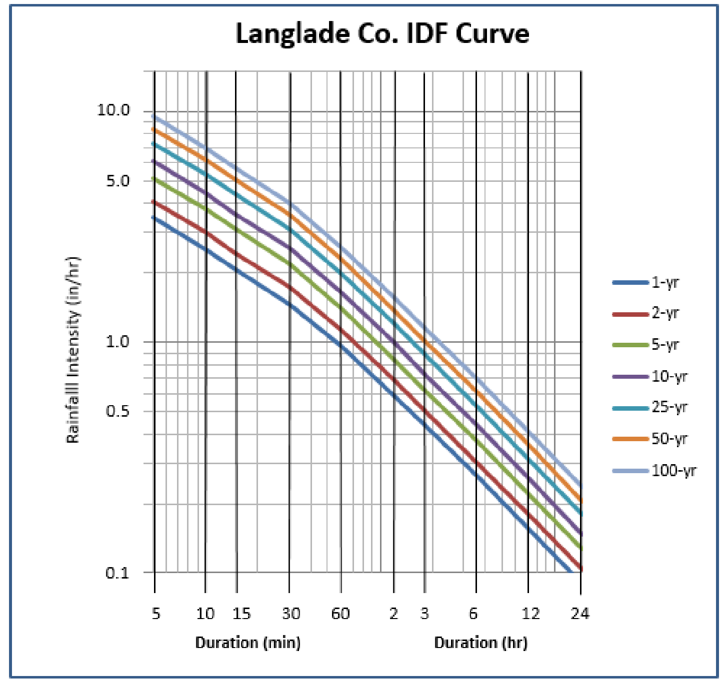
Lafayette County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.44	3.24	2.64	1.82	1.19	0.74	0.55	0.33	0.20	0.11
2-yr	5.28	3.90	3.16	2.18	1.42	0.87	0.65	0.39	0.22	0.13
5-yr	6.72	4.98	4.04	2.80	1.81	1.12	0.83	0.49	0.28	0.16
10-yr	8.04	5.88	4.76	3.32	2.17	1.34	1.00	0.59	0.33	0.19
25-yr	9.84	7.20	5.84	4.08	2.70	1.68	1.26	0.74	0.42	0.24
50-yr	11.28	8.22	6.72	4.70	3.15	1.97	1.49	0.88	0.50	0.28
100-yr	12.72	9.30	7.60	5.34	3.62	2.29	1.74	1.04	0.58	0.32



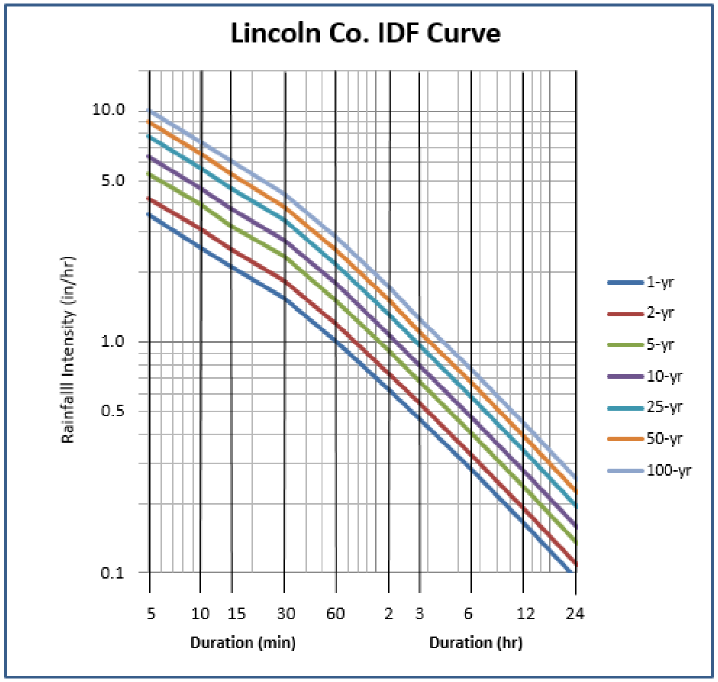
Langlade County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.48	2.52	2.08	1.46	0.96	0.59	0.44	0.26	0.15	0.09
2-yr	4.08	3.00	2.44	1.74	1.12	0.69	0.51	0.30	0.18	0.10
5-yr	5.16	3.78	3.08	2.18	1.40	0.85	0.63	0.37	0.22	0.13
10-yr	6.12	4.44	3.60	2.56	1.64	1.00	0.73	0.43	0.25	0.15
25-yr	7.32	5.40	4.40	3.12	1.99	1.21	0.89	0.53	0.31	0.18
50-yr	8.40	6.18	5.04	3.56	2.28	1.39	1.02	0.60	0.36	0.21
100-yr	9.60	7.02	5.68	4.02	2.58	1.57	1.16	0.69	0.40	0.23



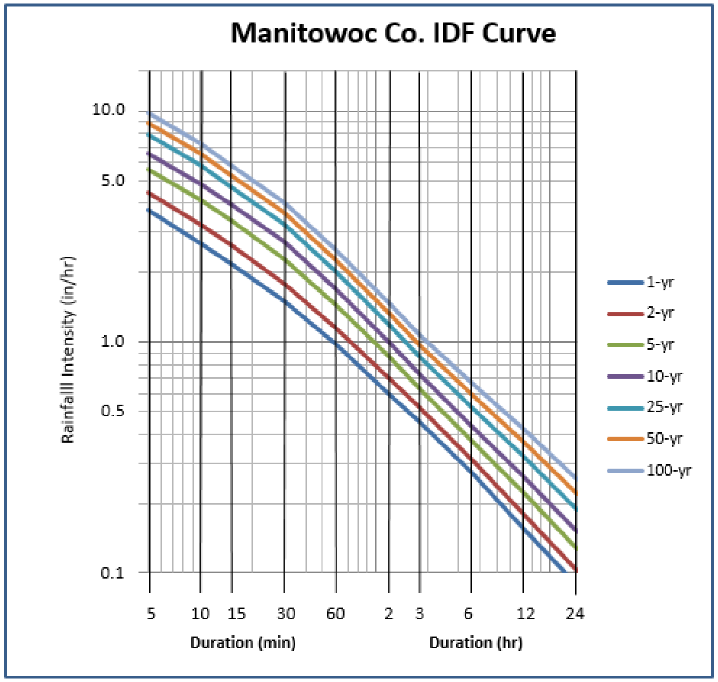
Lincoln County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.60	2.58	2.12	1.54	1.01	0.63	0.47	0.28	0.16	0.09
2-yr	4.20	3.12	2.52	1.84	1.19	0.74	0.54	0.32	0.19	0.11
5-yr	5.40	3.96	3.20	2.34	1.50	0.92	0.68	0.40	0.23	0.13
10-yr	6.36	4.68	3.80	2.76	1.78	1.09	0.80	0.47	0.27	0.16
25-yr	7.80	5.70	4.68	3.38	2.17	1.33	0.97	0.57	0.34	0.19
50-yr	9.00	6.60	5.36	3.88	2.50	1.53	1.12	0.66	0.39	0.22
100-yr	10.20	7.44	6.08	4.40	2.83	1.73	1.27	0.75	0.44	0.25



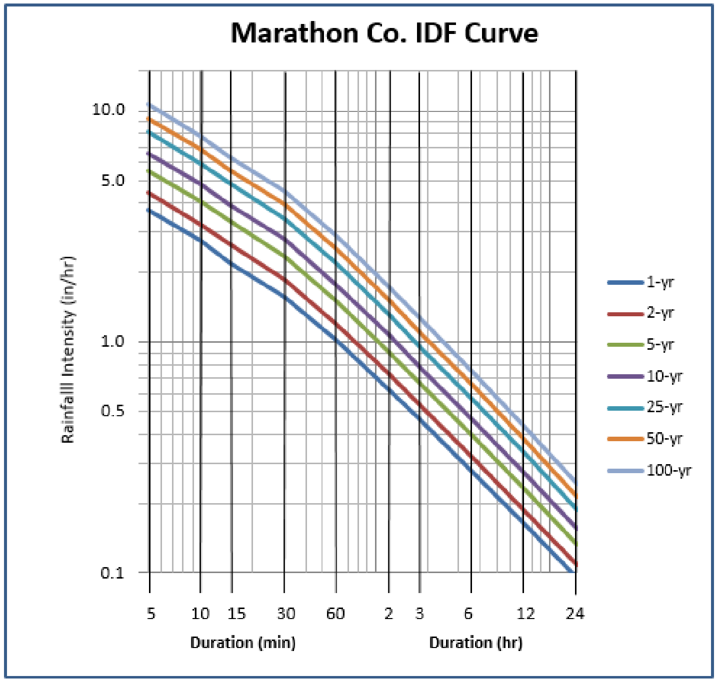
Manitowoc County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.72	2.70	2.20	1.50	0.97	0.60	0.45	0.27	0.15	0.09
2-yr	4.44	3.24	2.64	1.80	1.15	0.70	0.52	0.31	0.18	0.10
5-yr	5.64	4.14	3.36	2.30	1.44	0.87	0.63	0.37	0.22	0.13
10-yr	6.60	4.86	3.96	2.72	1.68	1.01	0.73	0.43	0.26	0.15
25-yr	7.92	5.82	4.72	3.24	2.00	1.19	0.86	0.52	0.32	0.19
50-yr	8.88	6.54	5.32	3.64	2.25	1.34	0.97	0.59	0.36	0.22
100-yr	9.84	7.26	5.88	4.04	2.48	1.48	1.08	0.66	0.42	0.25



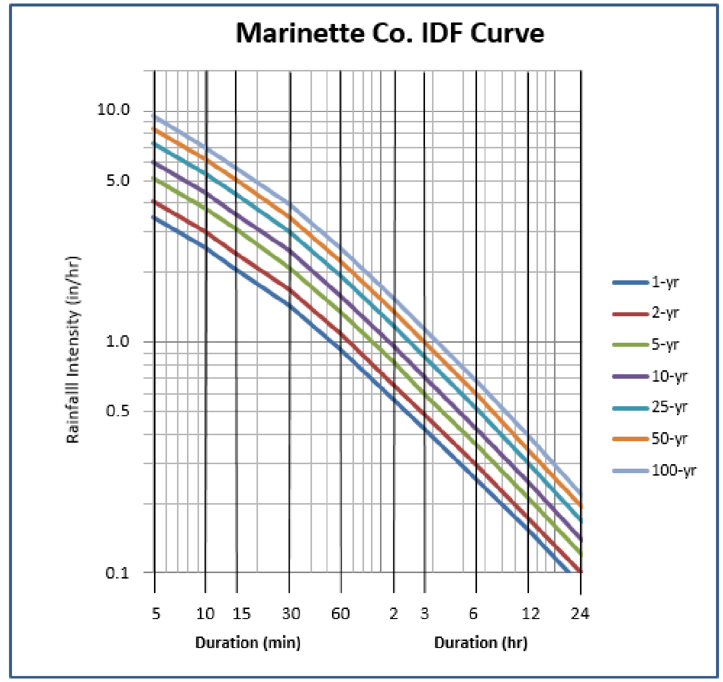
Marathon County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.72	2.76	2.20	1.58	1.02	0.63	0.46	0.27	0.16	0.09
2-yr	4.44	3.24	2.64	1.86	1.20	0.73	0.54	0.32	0.19	0.11
5-yr	5.52	4.08	3.32	2.36	1.50	0.91	0.67	0.39	0.23	0.13
10-yr	6.60	4.86	3.92	2.80	1.77	1.08	0.79	0.46	0.27	0.16
25-yr	8.16	5.94	4.84	3.44	2.18	1.33	0.96	0.56	0.33	0.19
50-yr	9.36	6.84	5.56	3.96	2.52	1.53	1.12	0.65	0.38	0.22
100-yr	10.68	7.80	6.32	4.52	2.88	1.75	1.28	0.74	0.43	0.24



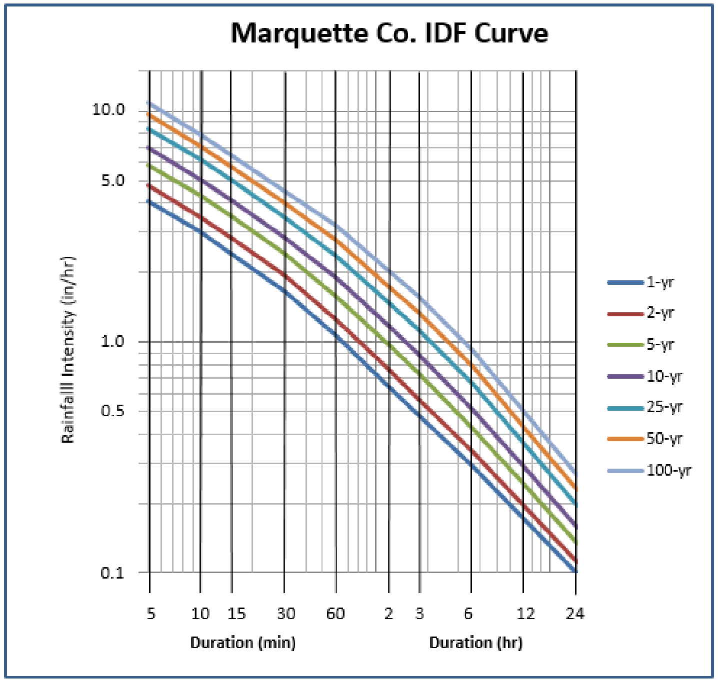
Marinette County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.48	2.58	2.08	1.44	0.92	0.57	0.42	0.25	0.15	0.09
2-yr	4.08	3.00	2.44	1.68	1.08	0.66	0.49	0.29	0.17	0.10
5-yr	5.16	3.78	3.08	2.10	1.34	0.82	0.60	0.36	0.21	0.12
10-yr	6.00	4.44	3.60	2.48	1.58	0.96	0.71	0.42	0.24	0.14
25-yr	7.32	5.40	4.36	3.02	1.93	1.18	0.87	0.51	0.29	0.17
50-yr	8.40	6.18	5.04	3.48	2.23	1.36	1.00	0.59	0.34	0.19
100-yr	9.60	7.02	5.68	3.94	2.54	1.55	1.15	0.67	0.38	0.22



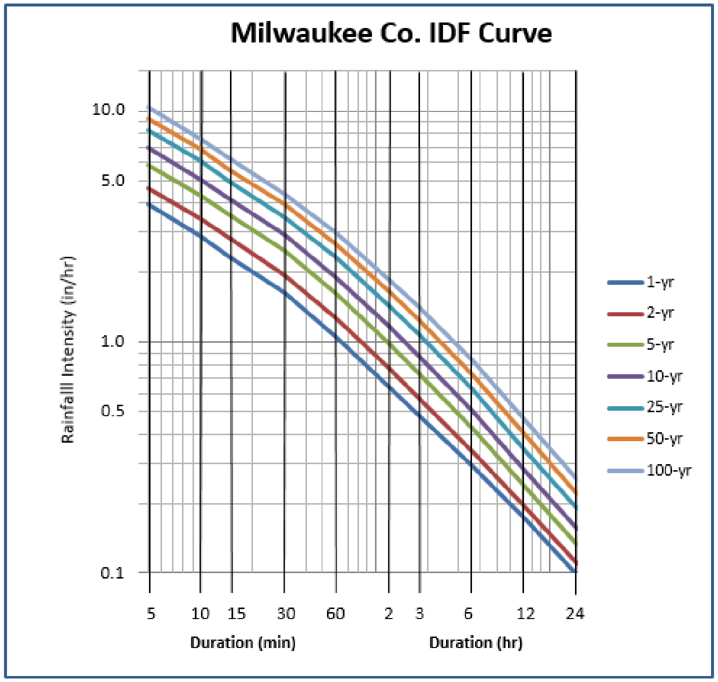
Marquette County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.08	3.00	2.44	1.66	1.06	0.65	0.48	0.29	0.17	0.10
2-yr	4.80	3.48	2.84	1.94	1.25	0.76	0.56	0.33	0.19	0.11
5-yr	5.88	4.32	3.52	2.44	1.58	0.98	0.73	0.42	0.24	0.13
10-yr	6.96	5.10	4.12	2.86	1.89	1.18	0.88	0.51	0.29	0.16
25-yr	8.40	6.18	5.04	3.50	2.35	1.48	1.12	0.66	0.36	0.20
50-yr	9.72	7.08	5.76	4.00	2.75	1.75	1.34	0.78	0.42	0.23
100-yr	10.92	7.98	6.52	4.54	3.17	2.04	1.57	0.93	0.50	0.27



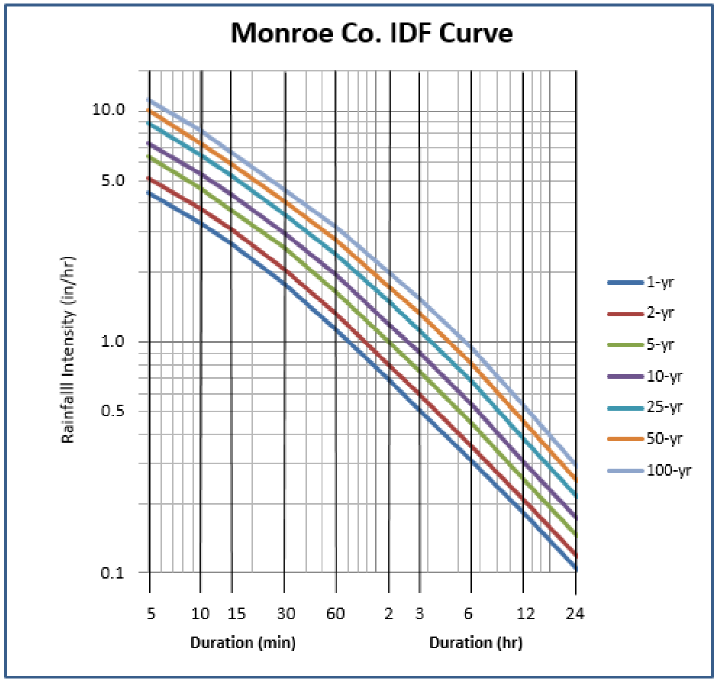
Milwaukee County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.96	2.88	2.32	1.64	1.05	0.65	0.48	0.29	0.17	0.10
2-yr	4.68	3.42	2.80	1.96	1.26	0.78	0.57	0.34	0.19	0.11
5-yr	5.88	4.32	3.52	2.48	1.61	0.99	0.73	0.42	0.24	0.13
10-yr	6.96	5.10	4.12	2.92	1.91	1.18	0.87	0.50	0.28	0.16
25-yr	8.28	6.12	4.96	3.50	2.32	1.44	1.08	0.62	0.34	0.19
50-yr	9.36	6.84	5.56	3.94	2.64	1.66	1.25	0.72	0.40	0.22
100-yr	10.44	7.62	6.20	4.36	2.96	1.88	1.42	0.84	0.46	0.25



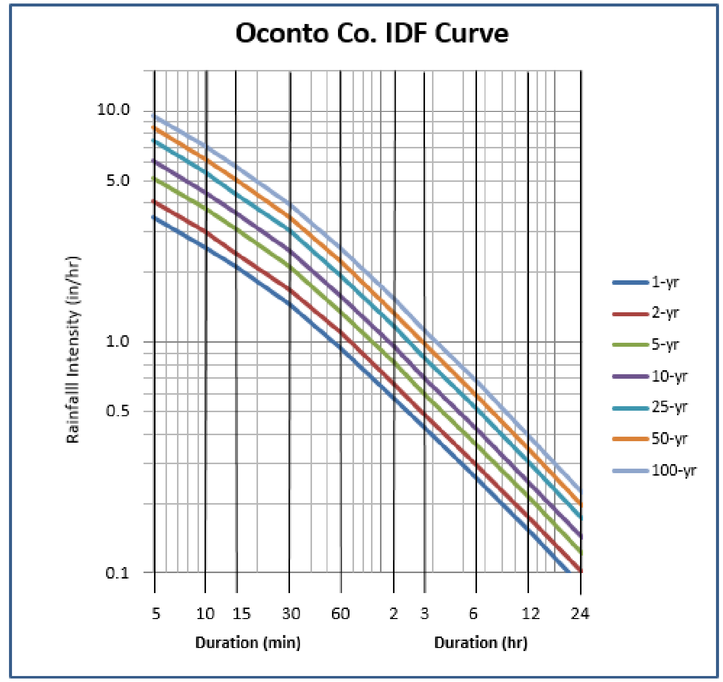
Monroe County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.44	3.30	2.68	1.80	1.13	0.69	0.51	0.30	0.18	0.10
2-yr	5.16	3.78	3.08	2.08	1.32	0.80	0.59	0.35	0.20	0.12
5-yr	6.36	4.62	3.76	2.56	1.65	1.01	0.75	0.44	0.25	0.15
10-yr	7.32	5.40	4.40	2.98	1.95	1.20	0.90	0.53	0.30	0.17
25-yr	8.88	6.48	5.28	3.60	2.39	1.50	1.13	0.67	0.38	0.21
50-yr	10.08	7.32	5.96	4.08	2.76	1.74	1.33	0.80	0.45	0.25
100-yr	11.28	8.28	6.72	4.60	3.15	2.01	1.55	0.94	0.52	0.29



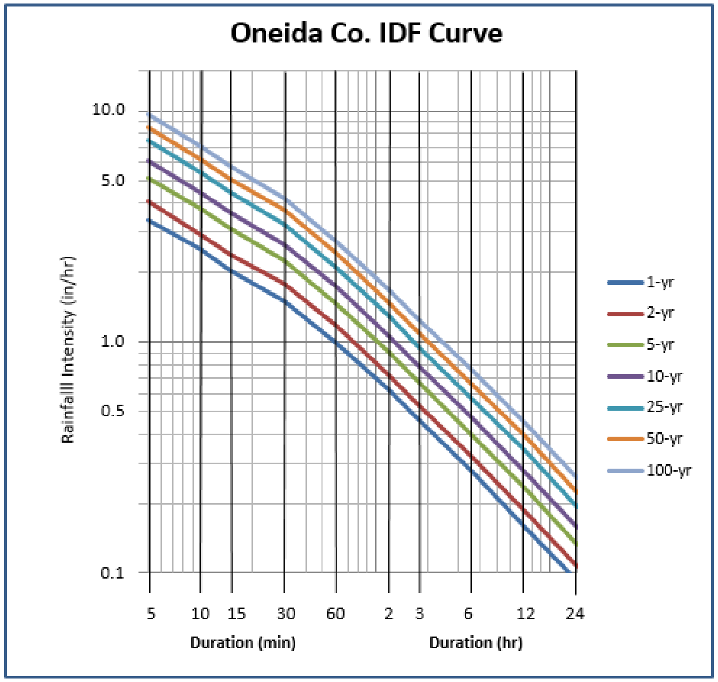
Oconto County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.48	2.58	2.12	1.46	0.94	0.58	0.43	0.26	0.15	0.09
2-yr	4.08	3.00	2.44	1.70	1.09	0.66	0.49	0.29	0.17	0.10
5-yr	5.16	3.78	3.08	2.12	1.35	0.82	0.60	0.36	0.21	0.12
10-yr	6.12	4.44	3.64	2.50	1.58	0.96	0.70	0.42	0.24	0.14
25-yr	7.44	5.46	4.40	3.04	1.93	1.17	0.86	0.51	0.30	0.17
50-yr	8.52	6.24	5.08	3.50	2.22	1.35	0.99	0.58	0.34	0.20
100-yr	9.60	7.08	5.76	3.98	2.53	1.54	1.13	0.67	0.39	0.22



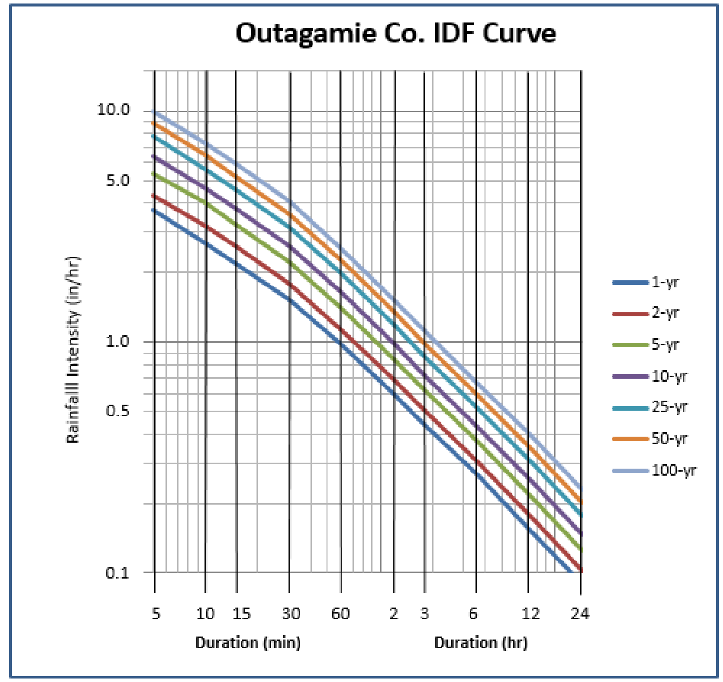
Oneida County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.36	2.52	2.04	1.50	0.99	0.62	0.46	0.27	0.16	0.09
2-yr	4.08	2.94	2.40	1.78	1.17	0.73	0.53	0.32	0.18	0.11
5-yr	5.16	3.78	3.08	2.26	1.47	0.91	0.67	0.39	0.23	0.13
10-yr	6.12	4.44	3.64	2.66	1.73	1.06	0.78	0.46	0.27	0.16
25-yr	7.44	5.46	4.44	3.26	2.11	1.30	0.95	0.57	0.34	0.19
50-yr	8.52	6.24	5.08	3.72	2.42	1.49	1.10	0.66	0.39	0.22
100-yr	9.72	7.08	5.76	4.22	2.74	1.69	1.25	0.75	0.45	0.26



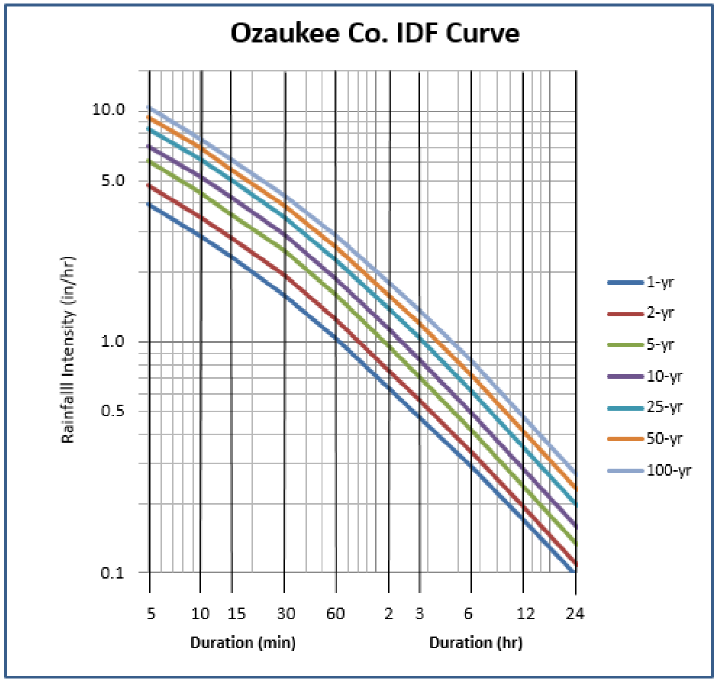
Outagamie County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.72	2.70	2.20	1.52	0.97	0.60	0.44	0.27	0.15	0.09
2-yr	4.32	3.18	2.60	1.78	1.13	0.69	0.51	0.30	0.18	0.10
5-yr	5.40	4.02	3.24	2.22	1.41	0.85	0.62	0.37	0.22	0.13
10-yr	6.36	4.68	3.80	2.62	1.64	0.99	0.72	0.43	0.25	0.15
25-yr	7.80	5.64	4.60	3.16	1.98	1.19	0.87	0.51	0.31	0.18
50-yr	8.88	6.48	5.24	3.60	2.26	1.36	0.99	0.59	0.35	0.20
100-yr	9.96	7.26	5.92	4.06	2.55	1.53	1.12	0.66	0.40	0.23



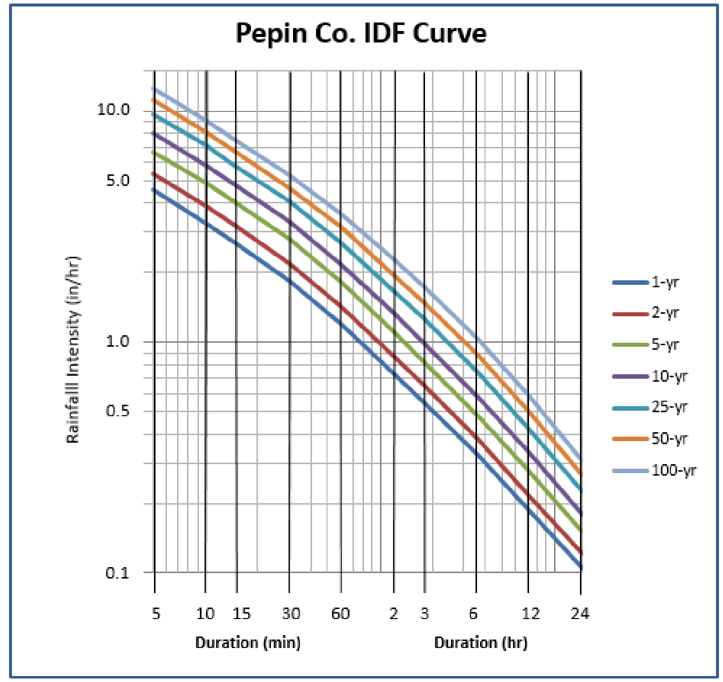
Ozaukee County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.96	2.88	2.36	1.60	1.03	0.63	0.47	0.29	0.17	0.10
2-yr	4.80	3.48	2.84	1.94	1.24	0.76	0.56	0.33	0.19	0.11
5-yr	6.12	4.44	3.60	2.50	1.59	0.97	0.71	0.41	0.23	0.13
10-yr	7.08	5.22	4.24	2.92	1.88	1.15	0.85	0.49	0.28	0.16
25-yr	8.40	6.18	5.04	3.50	2.27	1.40	1.04	0.61	0.34	0.20
50-yr	9.48	6.96	5.64	3.92	2.58	1.61	1.21	0.71	0.40	0.23
100-yr	10.44	7.62	6.20	4.32	2.89	1.81	1.38	0.82	0.47	0.27



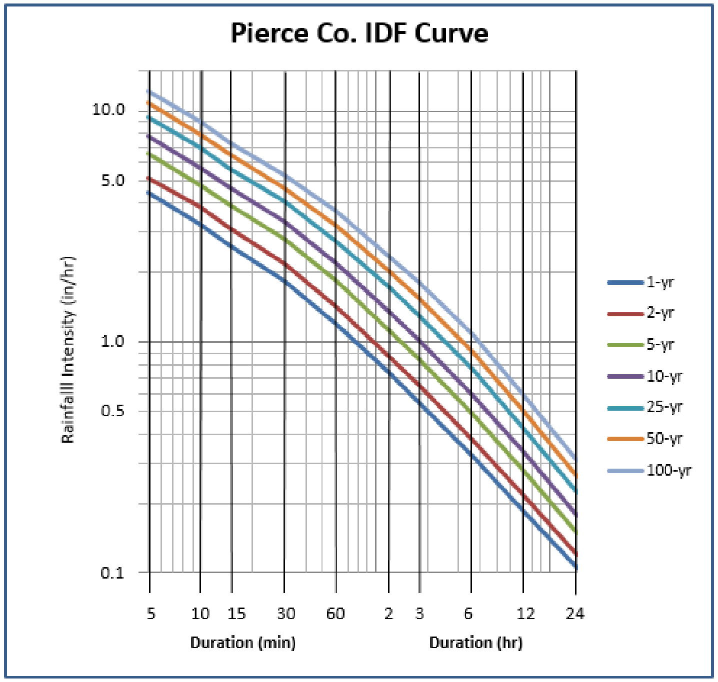
Pepin County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.56	3.30	2.68	1.84	1.19	0.74	0.55	0.33	0.18	0.10
2-yr	5.40	3.90	3.20	2.20	1.42	0.87	0.65	0.38	0.22	0.12
5-yr	6.72	4.92	4.04	2.80	1.81	1.11	0.82	0.48	0.27	0.15
10-yr	8.04	5.82	4.76	3.32	2.16	1.34	0.99	0.58	0.33	0.18
25-yr	9.72	7.14	5.80	4.06	2.69	1.68	1.26	0.74	0.42	0.23
50-yr	11.16	8.16	6.64	4.66	3.13	1.97	1.49	0.88	0.49	0.27
100-yr	12.60	9.18	7.48	5.26	3.60	2.28	1.74	1.04	0.58	0.31



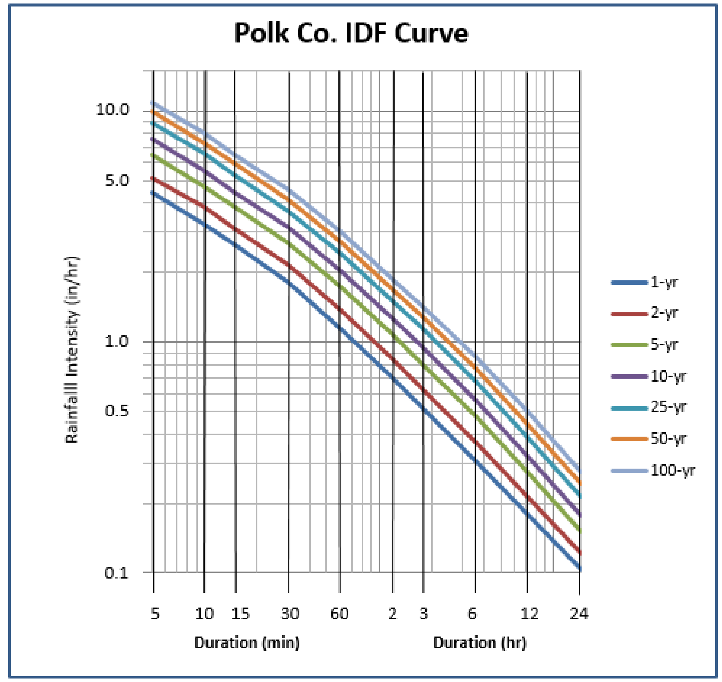
Pierce County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.44	3.24	2.60	1.84	1.20	0.74	0.55	0.32	0.18	0.10
2-yr	5.16	3.84	3.08	2.20	1.42	0.88	0.65	0.38	0.21	0.12
5-yr	6.60	4.80	3.92	2.80	1.83	1.13	0.84	0.49	0.27	0.15
10-yr	7.80	5.70	4.64	3.32	2.20	1.37	1.02	0.59	0.33	0.18
25-yr	9.48	6.96	5.64	4.06	2.74	1.73	1.30	0.76	0.42	0.22
50-yr	10.92	7.98	6.48	4.66	3.20	2.04	1.55	0.91	0.50	0.26
100-yr	12.24	9.00	7.32	5.28	3.69	2.37	1.82	1.08	0.58	0.31



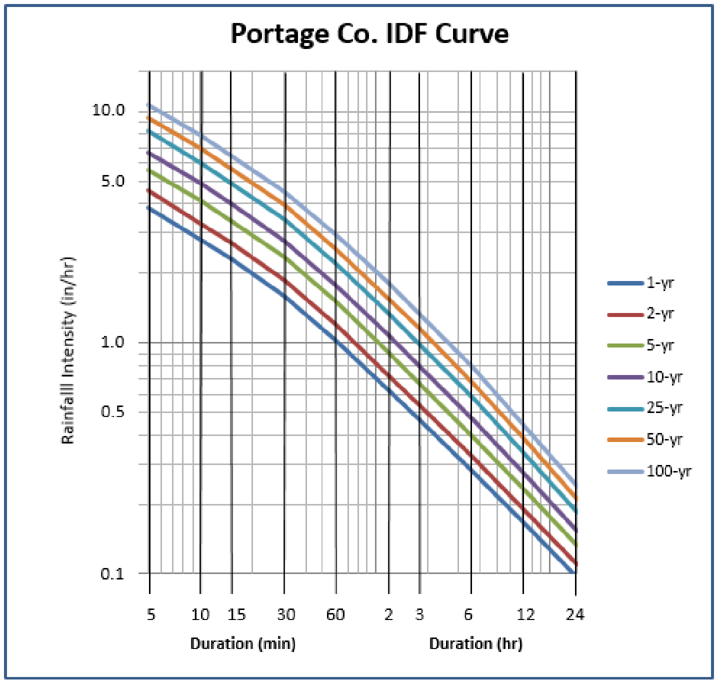
Polk County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.44	3.24	2.64	1.82	1.15	0.70	0.51	0.30	0.18	0.10
2-yr	5.16	3.84	3.12	2.16	1.38	0.85	0.62	0.37	0.21	0.12
5-yr	6.48	4.74	3.84	2.70	1.75	1.08	0.80	0.47	0.27	0.15
10-yr	7.56	5.52	4.48	3.14	2.05	1.27	0.94	0.56	0.32	0.18
25-yr	8.88	6.54	5.28	3.70	2.44	1.52	1.14	0.67	0.38	0.21
50-yr	9.96	7.26	5.92	4.14	2.73	1.70	1.28	0.76	0.44	0.24
100-yr	10.92	8.04	6.52	4.56	3.02	1.88	1.42	0.85	0.49	0.27



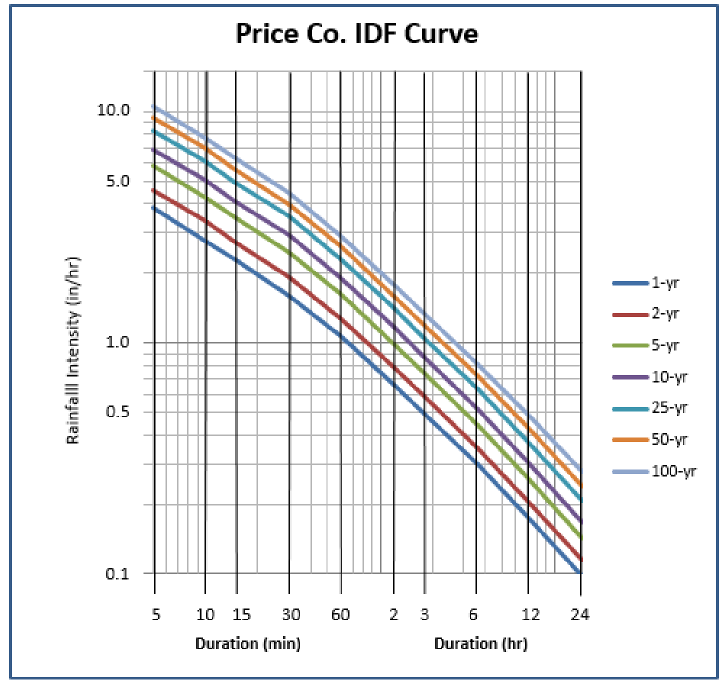
Portage County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.84	2.82	2.32	1.60	1.02	0.63	0.47	0.28	0.17	0.10
2-yr	4.56	3.30	2.72	1.88	1.19	0.73	0.54	0.32	0.19	0.11
5-yr	5.64	4.14	3.40	2.36	1.50	0.91	0.67	0.39	0.23	0.13
10-yr	6.72	4.92	4.00	2.78	1.77	1.08	0.79	0.47	0.27	0.15
25-yr	8.28	6.06	4.92	3.42	2.19	1.34	0.99	0.58	0.33	0.19
50-yr	9.48	6.96	5.68	3.94	2.55	1.56	1.16	0.67	0.38	0.21
100-yr	10.80	7.92	6.44	4.50	2.93	1.81	1.34	0.78	0.44	0.24



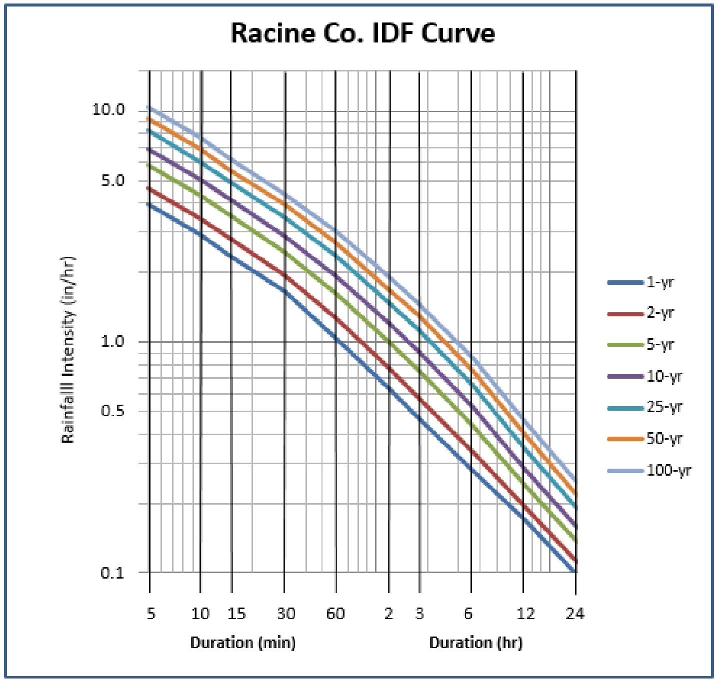
Price County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.84	2.76	2.28	1.60	1.06	0.66	0.50	0.30	0.17	0.10
2-yr	4.56	3.36	2.72	1.92	1.27	0.79	0.59	0.35	0.20	0.11
5-yr	5.88	4.26	3.48	2.46	1.61	1.00	0.74	0.44	0.25	0.14
10-yr	6.84	5.04	4.08	2.92	1.90	1.17	0.87	0.52	0.30	0.17
25-yr	8.28	6.12	4.96	3.52	2.29	1.42	1.05	0.63	0.37	0.21
50-yr	9.48	6.96	5.64	3.98	2.60	1.60	1.20	0.72	0.42	0.24
100-yr	10.56	7.74	6.32	4.46	2.90	1.79	1.34	0.81	0.48	0.28



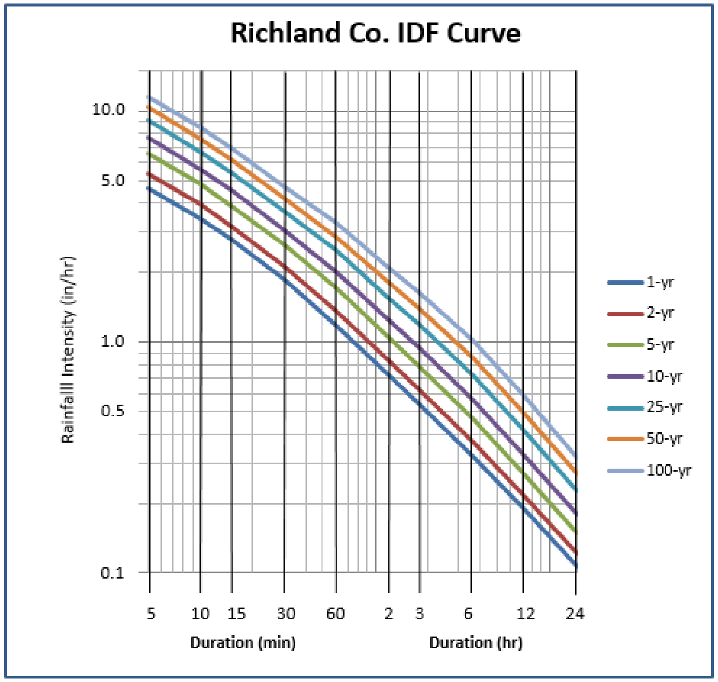
Racine County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.96	2.94	2.36	1.66	1.04	0.63	0.46	0.28	0.17	0.10
2-yr	4.68	3.42	2.80	1.96	1.26	0.77	0.57	0.34	0.19	0.11
5-yr	5.88	4.32	3.52	2.46	1.62	1.01	0.75	0.44	0.24	0.14
10-yr	6.84	5.04	4.12	2.90	1.93	1.21	0.90	0.52	0.28	0.16
25-yr	8.28	6.06	4.92	3.48	2.35	1.49	1.12	0.65	0.35	0.19
50-yr	9.36	6.84	5.56	3.94	2.69	1.70	1.29	0.75	0.40	0.22
100-yr	10.44	7.68	6.24	4.40	3.02	1.93	1.47	0.85	0.46	0.25



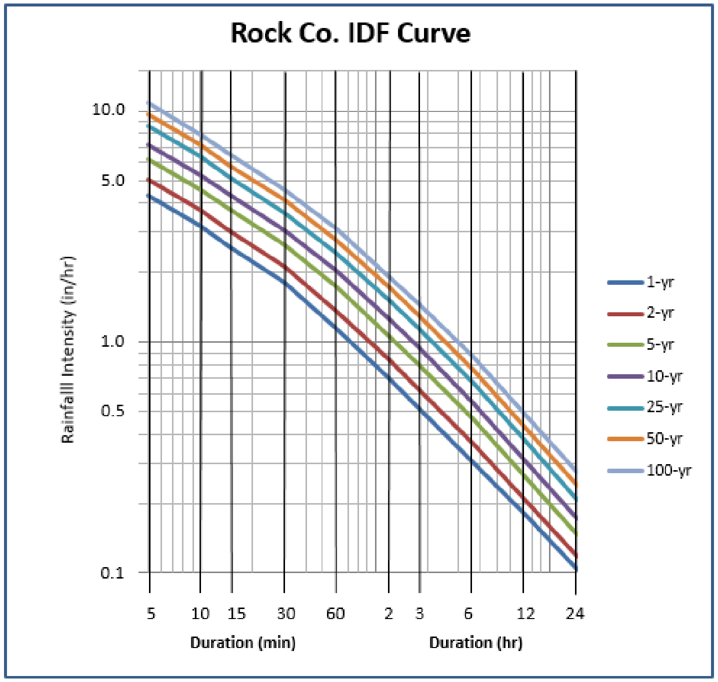
Richland County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.68	3.42	2.80	1.86	1.18	0.72	0.54	0.32	0.19	0.11
2-yr	5.40	3.96	3.20	2.14	1.37	0.84	0.62	0.37	0.21	0.12
5-yr	6.60	4.86	3.92	2.64	1.71	1.05	0.79	0.47	0.27	0.15
10-yr	7.68	5.58	4.56	3.06	2.01	1.25	0.94	0.56	0.32	0.18
25-yr	9.12	6.72	5.44	3.68	2.48	1.56	1.19	0.72	0.41	0.23
50-yr	10.44	7.62	6.20	4.18	2.86	1.82	1.41	0.86	0.49	0.27
100-yr	11.64	8.58	6.96	4.70	3.27	2.10	1.65	1.02	0.58	0.32



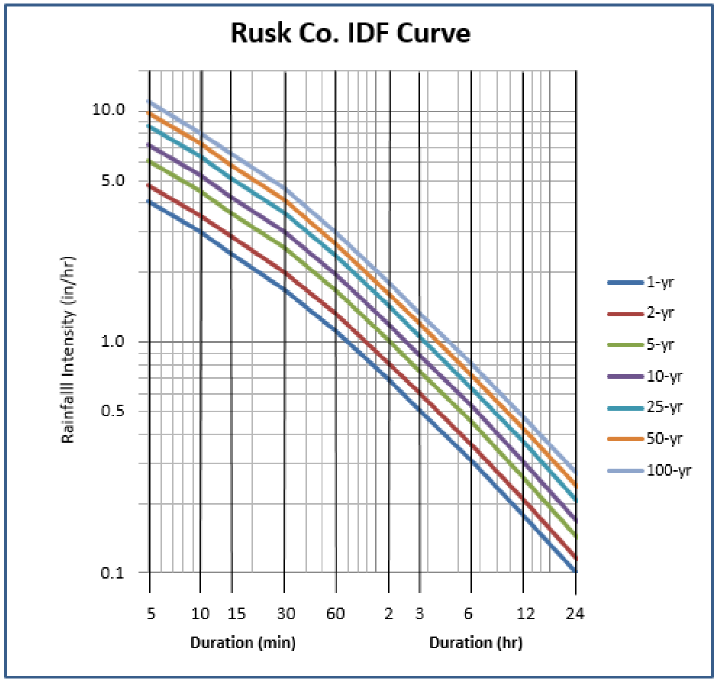
Rock County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.32	3.18	2.56	1.82	1.15	0.70	0.51	0.30	0.18	0.10
2-yr	5.04	3.72	3.00	2.12	1.37	0.84	0.62	0.36	0.21	0.12
5-yr	6.24	4.56	3.72	2.64	1.73	1.07	0.80	0.46	0.26	0.15
10-yr	7.20	5.28	4.32	3.06	2.03	1.26	0.94	0.55	0.31	0.17
25-yr	8.64	6.36	5.16	3.66	2.44	1.53	1.14	0.67	0.37	0.21
50-yr	9.72	7.14	5.80	4.14	2.76	1.73	1.30	0.77	0.43	0.24
100-yr	10.92	7.92	6.48	4.60	3.08	1.94	1.46	0.86	0.49	0.27



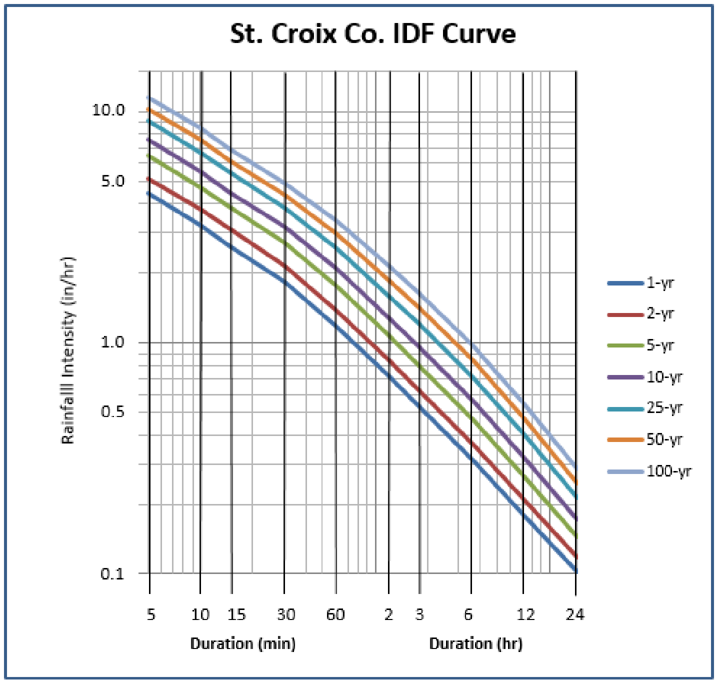
Rusk County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.08	3.00	2.44	1.70	1.11	0.69	0.51	0.30	0.17	0.10
2-yr	4.80	3.54	2.88	2.02	1.32	0.81	0.60	0.36	0.20	0.12
5-yr	6.12	4.50	3.64	2.56	1.66	1.02	0.76	0.45	0.26	0.14
10-yr	7.20	5.28	4.28	3.00	1.95	1.20	0.89	0.52	0.30	0.17
25-yr	8.64	6.36	5.16	3.64	2.35	1.44	1.06	0.63	0.36	0.21
50-yr	9.84	7.26	5.88	4.14	2.66	1.63	1.20	0.71	0.42	0.24
100-yr	11.04	8.10	6.60	4.64	2.98	1.82	1.35	0.80	0.47	0.27



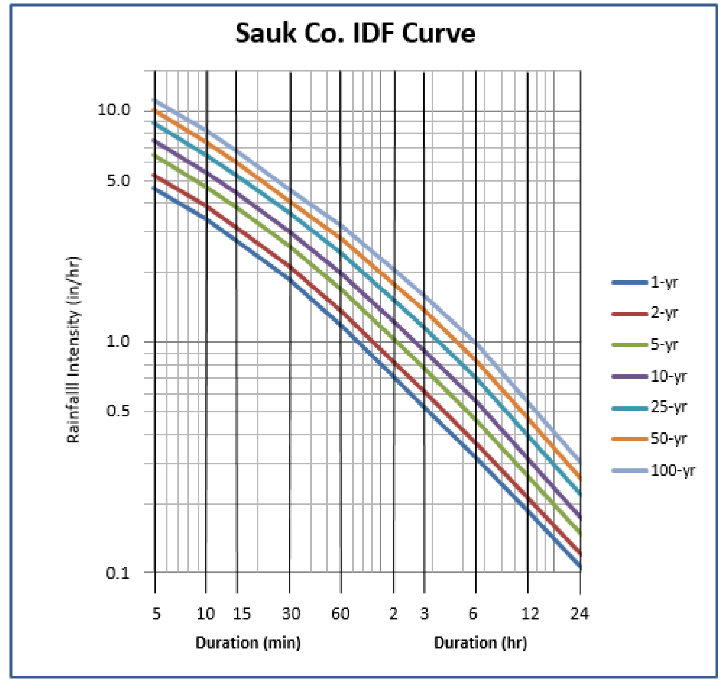
St. Croix County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.44	3.24	2.60	1.84	1.18	0.72	0.53	0.31	0.18	0.10
2-yr	5.16	3.78	3.08	2.16	1.39	0.85	0.63	0.36	0.21	0.12
5-yr	6.48	4.74	3.84	2.72	1.76	1.08	0.80	0.47	0.26	0.15
10-yr	7.56	5.52	4.48	3.18	2.09	1.29	0.96	0.56	0.32	0.17
25-yr	9.12	6.66	5.44	3.84	2.57	1.61	1.21	0.71	0.40	0.21
50-yr	10.32	7.56	6.16	4.38	2.96	1.87	1.42	0.84	0.47	0.25
100-yr	11.64	8.52	6.92	4.90	3.38	2.15	1.65	0.98	0.54	0.29



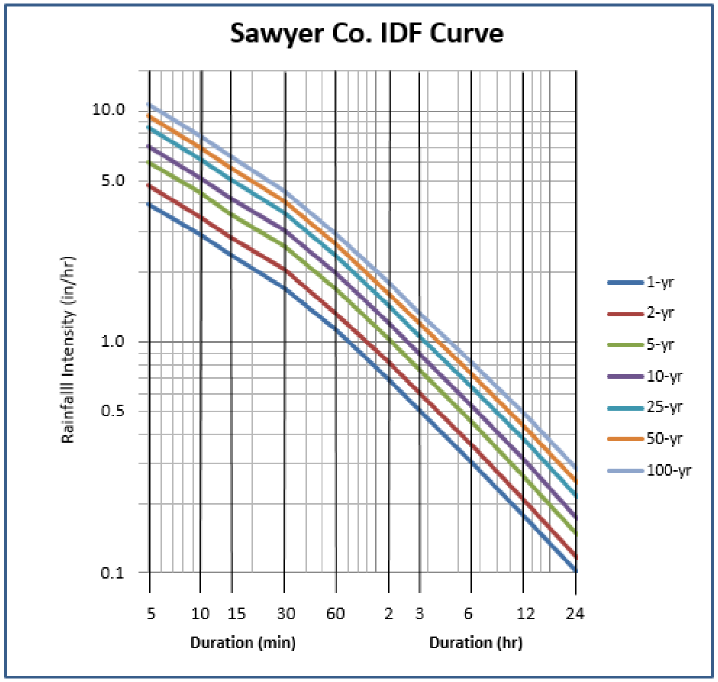
Sauk County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.68	3.42	2.76	1.86	1.18	0.71	0.52	0.31	0.18	0.11
2-yr	5.28	3.90	3.16	2.14	1.36	0.83	0.61	0.36	0.21	0.12
5-yr	6.48	4.74	3.84	2.60	1.69	1.04	0.77	0.45	0.26	0.15
10-yr	7.44	5.46	4.44	3.02	1.98	1.23	0.92	0.54	0.31	0.17
25-yr	8.88	6.48	5.28	3.62	2.43	1.53	1.17	0.69	0.39	0.22
50-yr	10.08	7.38	6.00	4.10	2.81	1.78	1.37	0.83	0.46	0.25
100-yr	11.28	8.28	6.76	4.60	3.21	2.06	1.60	0.97	0.54	0.30



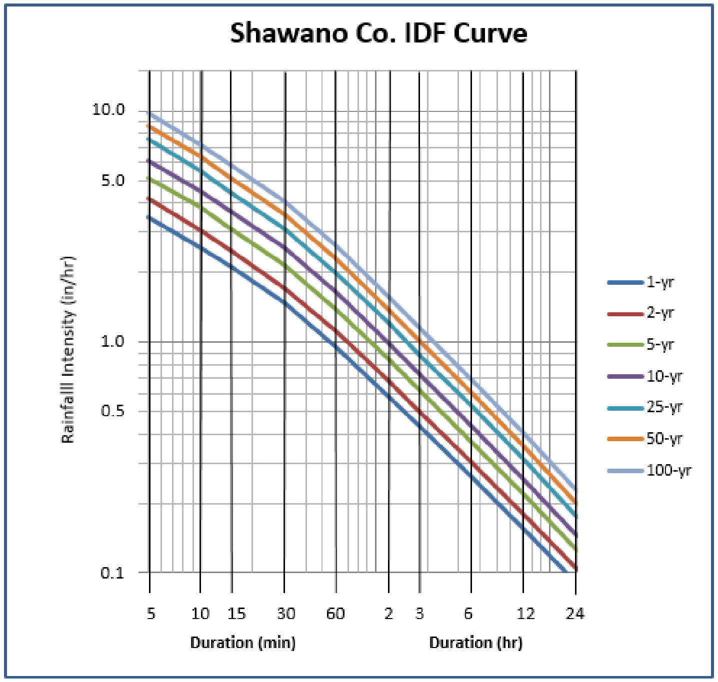
Sawyer County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.96	2.94	2.40	1.72	1.12	0.69	0.51	0.30	0.17	0.10
2-yr	4.80	3.48	2.84	2.06	1.33	0.82	0.61	0.36	0.21	0.12
5-yr	6.00	4.44	3.60	2.60	1.68	1.04	0.76	0.45	0.26	0.15
10-yr	7.08	5.16	4.20	3.04	1.97	1.21	0.89	0.53	0.31	0.17
25-yr	8.52	6.24	5.08	3.64	2.36	1.45	1.07	0.64	0.37	0.21
50-yr	9.60	7.02	5.72	4.10	2.65	1.63	1.21	0.72	0.43	0.25
100-yr	10.68	7.80	6.36	4.54	2.94	1.81	1.34	0.81	0.49	0.28



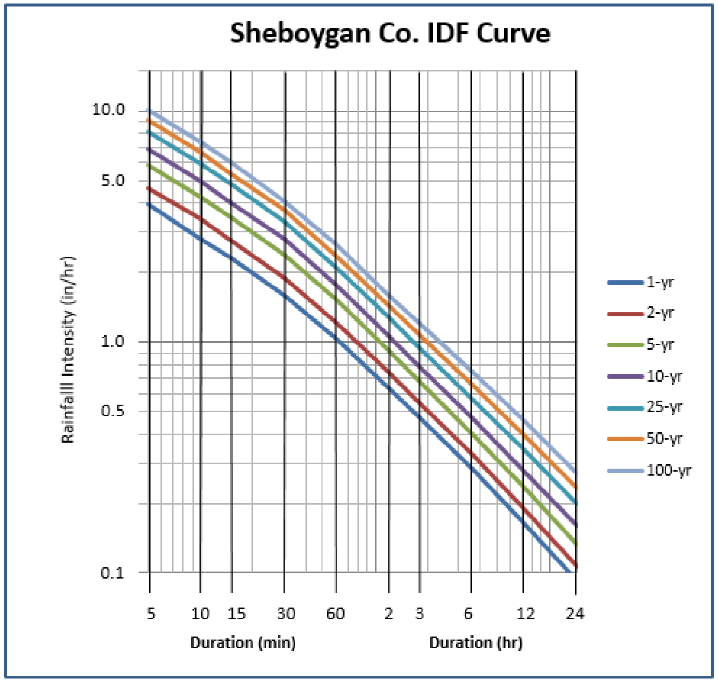
Shawano County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.48	2.58	2.12	1.48	0.95	0.58	0.43	0.26	0.15	0.09
2-yr	4.20	3.06	2.48	1.72	1.11	0.68	0.50	0.30	0.18	0.10
5-yr	5.16	3.84	3.12	2.16	1.38	0.84	0.62	0.37	0.22	0.13
10-yr	6.12	4.50	3.68	2.56	1.63	0.99	0.73	0.43	0.25	0.15
25-yr	7.56	5.52	4.48	3.12	1.99	1.21	0.89	0.52	0.31	0.18
50-yr	8.64	6.36	5.16	3.60	2.29	1.39	1.02	0.60	0.35	0.20
100-yr	9.84	7.20	5.88	4.08	2.60	1.58	1.16	0.68	0.40	0.23



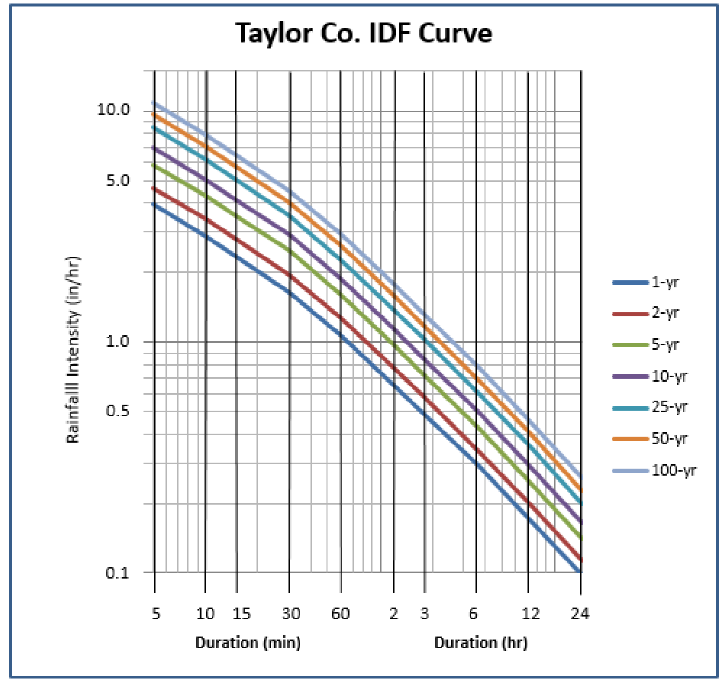
Sheboygan County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.96	2.82	2.32	1.60	1.03	0.64	0.47	0.28	0.16	0.09
2-yr	4.68	3.42	2.76	1.90	1.22	0.74	0.55	0.32	0.19	0.11
5-yr	5.88	4.26	3.48	2.40	1.52	0.92	0.68	0.40	0.23	0.13
10-yr	6.84	4.98	4.04	2.80	1.77	1.07	0.79	0.47	0.27	0.16
25-yr	8.16	5.94	4.84	3.32	2.11	1.28	0.95	0.57	0.34	0.20
50-yr	9.12	6.66	5.40	3.72	2.37	1.44	1.07	0.65	0.39	0.23
100-yr	10.08	7.38	6.00	4.10	2.63	1.61	1.21	0.74	0.46	0.27



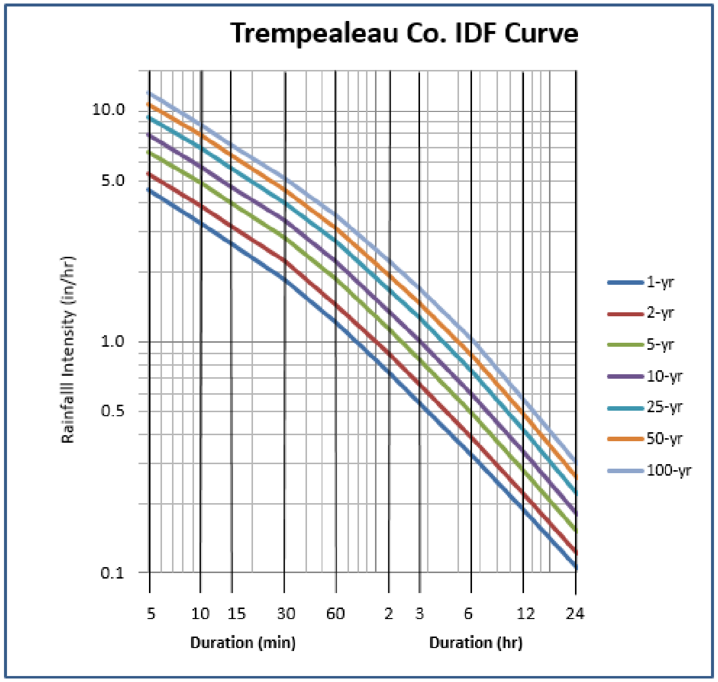
Taylor County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.96	2.88	2.36	1.64	1.06	0.66	0.49	0.29	0.17	0.10
2-yr	4.68	3.42	2.80	1.94	1.26	0.78	0.58	0.34	0.20	0.11
5-yr	5.88	4.32	3.52	2.48	1.59	0.98	0.72	0.43	0.25	0.14
10-yr	6.96	5.10	4.16	2.92	1.87	1.15	0.85	0.50	0.29	0.16
25-yr	8.52	6.18	5.04	3.54	2.27	1.39	1.03	0.61	0.35	0.20
50-yr	9.72	7.08	5.76	4.04	2.59	1.59	1.17	0.69	0.40	0.23
100-yr	10.92	7.98	6.48	4.54	2.92	1.79	1.32	0.78	0.46	0.26



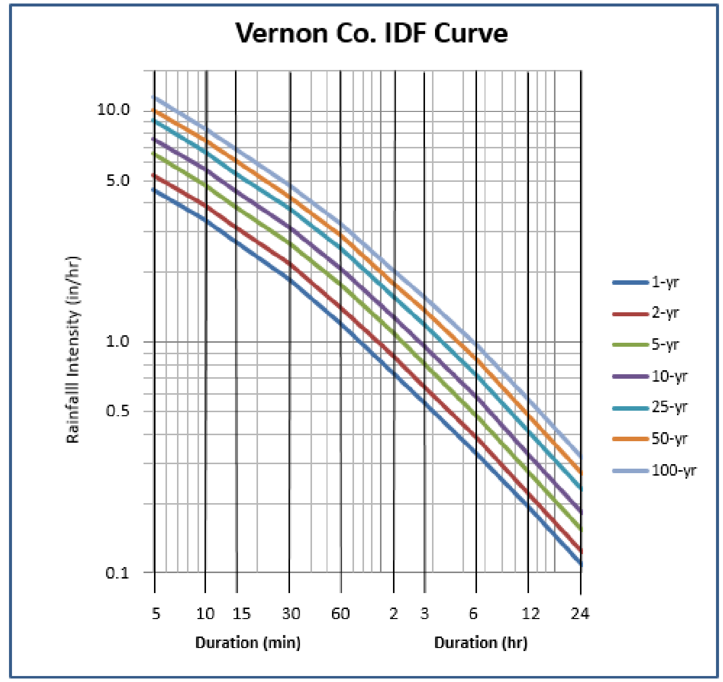
Trempealeau Co. Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.56	3.30	2.68	1.88	1.21	0.75	0.55	0.32	0.19	0.11
2-yr	5.40	3.90	3.20	2.26	1.45	0.89	0.66	0.38	0.22	0.12
5-yr	6.72	4.92	4.00	2.86	1.86	1.15	0.85	0.49	0.27	0.15
10-yr	7.92	5.76	4.72	3.36	2.21	1.37	1.02	0.59	0.33	0.18
25-yr	9.48	6.96	5.68	4.04	2.71	1.70	1.28	0.74	0.41	0.22
50-yr	10.80	7.92	6.44	4.58	3.11	1.97	1.49	0.87	0.48	0.26
100-yr	12.12	8.82	7.20	5.12	3.53	2.25	1.72	1.02	0.55	0.30



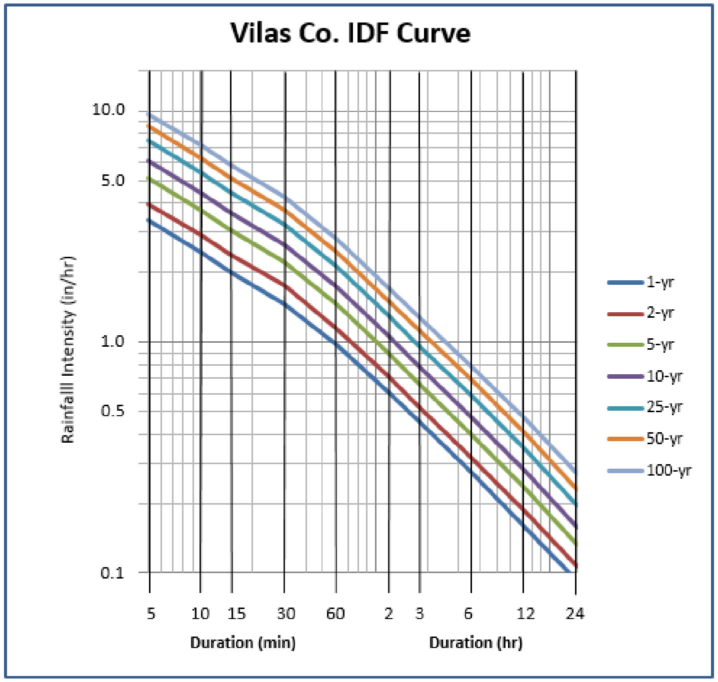
Vernon County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.56	3.36	2.72	1.86	1.20	0.73	0.54	0.32	0.19	0.11
2-yr	5.28	3.90	3.16	2.18	1.41	0.87	0.64	0.38	0.22	0.12
5-yr	6.60	4.80	3.88	2.70	1.77	1.09	0.81	0.48	0.27	0.15
10-yr	7.56	5.58	4.52	3.16	2.07	1.29	0.97	0.57	0.32	0.18
25-yr	9.12	6.66	5.40	3.78	2.52	1.57	1.19	0.71	0.40	0.23
50-yr	10.20	7.50	6.12	4.28	2.87	1.80	1.37	0.83	0.48	0.27
100-yr	11.52	8.40	6.84	4.78	3.23	2.04	1.57	0.96	0.56	0.32



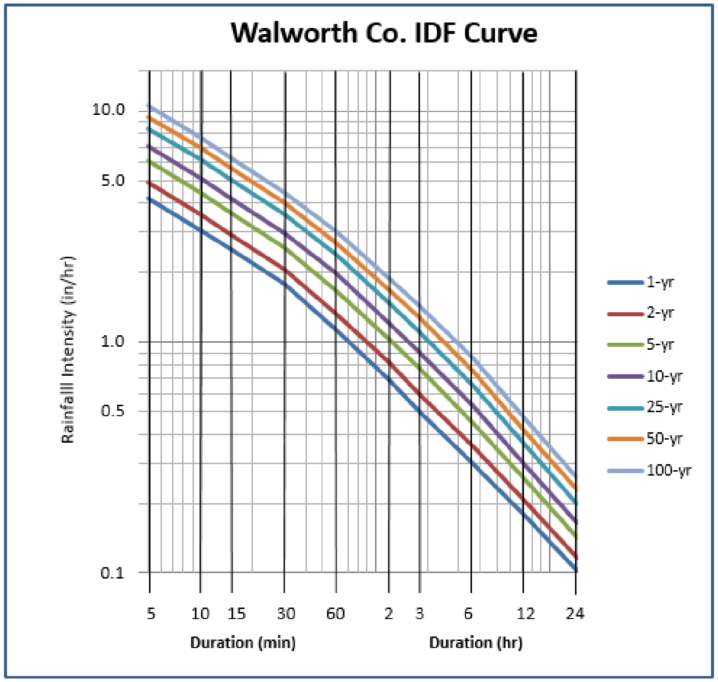
Vilas County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.36	2.46	2.00	1.46	0.97	0.61	0.45	0.27	0.16	0.09
2-yr	3.96	2.94	2.40	1.76	1.15	0.71	0.53	0.31	0.18	0.11
5-yr	5.16	3.72	3.04	2.24	1.46	0.90	0.66	0.39	0.23	0.13
10-yr	6.12	4.44	3.64	2.66	1.73	1.06	0.78	0.47	0.28	0.16
25-yr	7.44	5.46	4.44	3.26	2.12	1.31	0.97	0.58	0.35	0.20
50-yr	8.64	6.30	5.12	3.76	2.45	1.51	1.12	0.68	0.40	0.23
100-yr	9.72	7.14	5.84	4.26	2.79	1.73	1.28	0.78	0.47	0.27



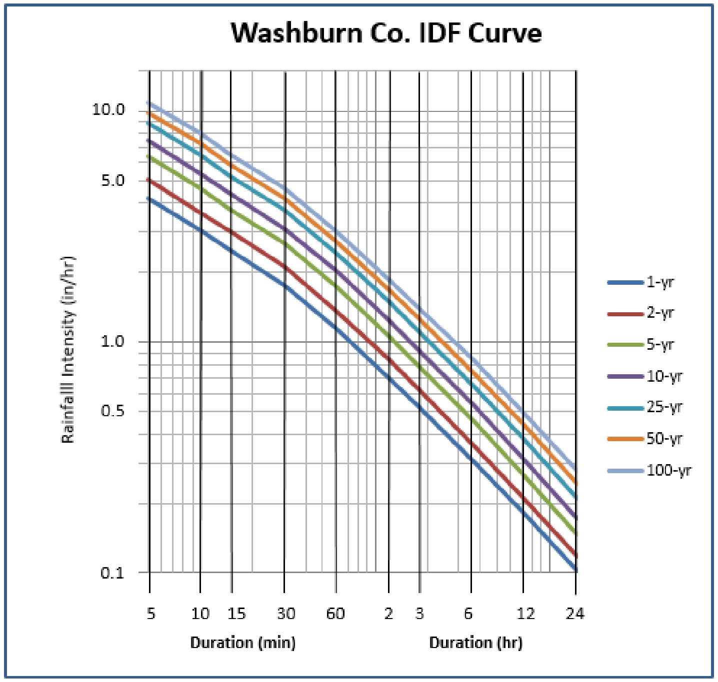
Walworth County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	4.20	3.06	2.52	1.78	1.13	0.69	0.50	0.30	0.18	0.10
2-yr	4.92	3.60	2.92	2.06	1.33	0.82	0.60	0.35	0.20	0.12
5-yr	6.12	4.44	3.64	2.56	1.67	1.04	0.77	0.45	0.25	0.14
10-yr	7.08	5.16	4.20	2.98	1.97	1.22	0.91	0.53	0.30	0.17
25-yr	8.40	6.18	5.04	3.56	2.38	1.49	1.12	0.65	0.36	0.20
50-yr	9.48	6.96	5.68	4.02	2.70	1.70	1.28	0.75	0.41	0.23
100-yr	10.56	7.74	6.32	4.48	3.03	1.91	1.45	0.85	0.47	0.26



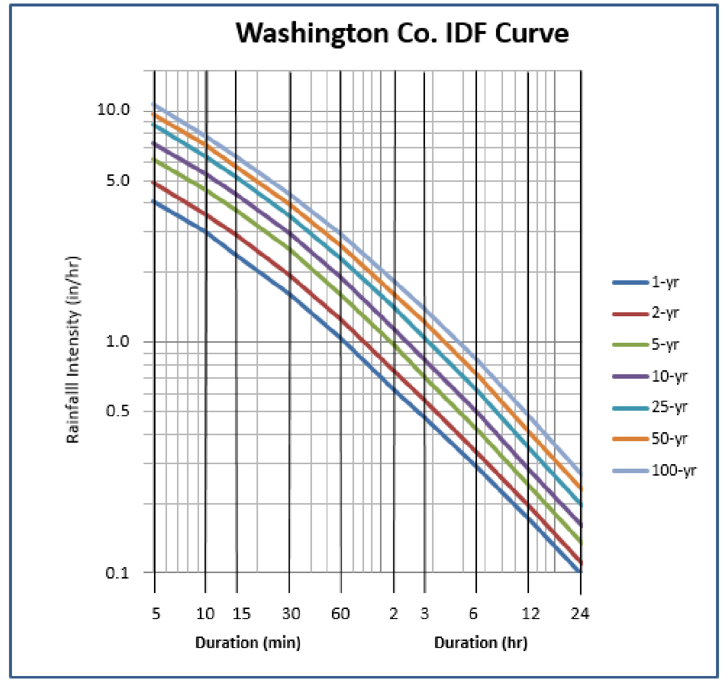
Washburn County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.20	3.06	2.48	1.76	1.14	0.70	0.52	0.31	0.18	0.10
2-yr	5.04	3.66	3.00	2.12	1.37	0.84	0.62	0.36	0.21	0.12
5-yr	6.36	4.62	3.76	2.68	1.73	1.06	0.78	0.46	0.26	0.15
10-yr	7.44	5.40	4.40	3.12	2.03	1.25	0.92	0.54	0.31	0.17
25-yr	8.88	6.48	5.24	3.74	2.43	1.50	1.11	0.65	0.38	0.21
50-yr	9.84	7.26	5.88	4.18	2.73	1.69	1.26	0.75	0.43	0.24
100-yr	10.92	8.04	6.52	4.62	3.03	1.88	1.41	0.84	0.49	0.28



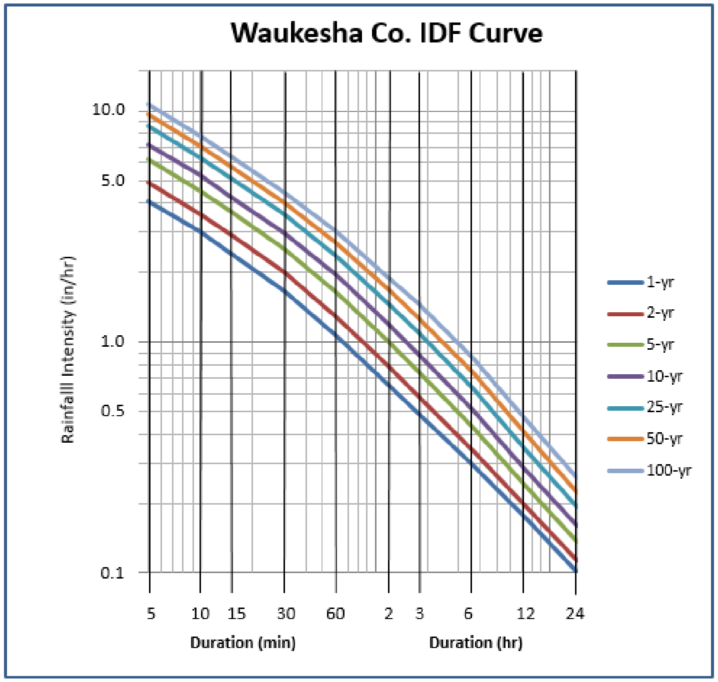
Washington Co. Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.08	3.00	2.40	1.62	1.03	0.63	0.47	0.29	0.17	0.10
2-yr	4.92	3.60	2.92	1.96	1.24	0.76	0.56	0.33	0.19	0.11
5-yr	6.24	4.56	3.72	2.52	1.60	0.97	0.71	0.41	0.24	0.14
10-yr	7.32	5.34	4.36	2.96	1.89	1.15	0.85	0.49	0.28	0.16
25-yr	8.76	6.36	5.20	3.54	2.30	1.42	1.05	0.62	0.35	0.20
50-yr	9.72	7.14	5.80	3.98	2.62	1.63	1.22	0.72	0.41	0.23
100-yr	10.80	7.86	6.40	4.40	2.94	1.84	1.40	0.83	0.47	0.27



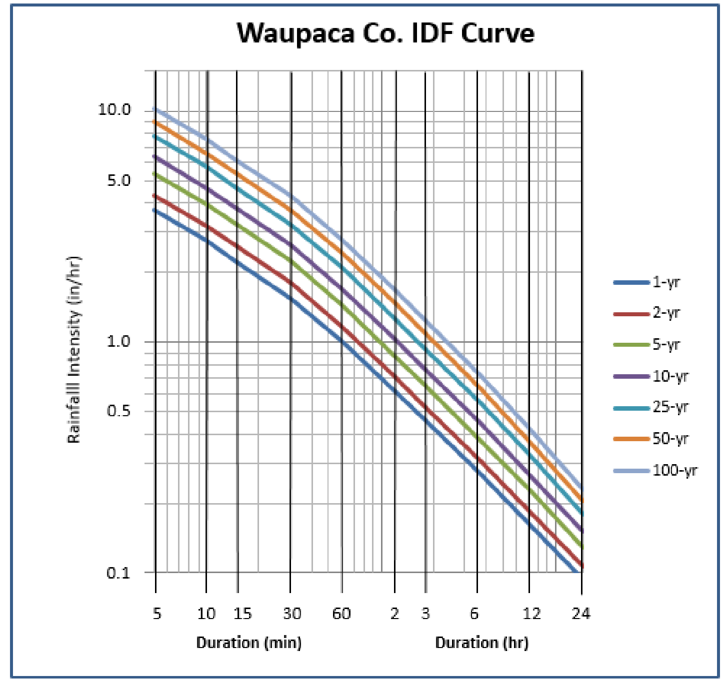
Waukesha County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.08	3.00	2.44	1.66	1.07	0.65	0.49	0.29	0.17	0.10
2-yr	4.92	3.60	2.92	2.00	1.28	0.78	0.58	0.34	0.20	0.11
5-yr	6.24	4.50	3.68	2.54	1.64	1.00	0.74	0.43	0.24	0.14
10-yr	7.20	5.28	4.28	2.98	1.94	1.19	0.88	0.51	0.28	0.16
25-yr	8.64	6.30	5.12	3.58	2.36	1.47	1.10	0.63	0.35	0.19
50-yr	9.72	7.08	5.76	4.02	2.69	1.69	1.27	0.74	0.40	0.22
100-yr	10.80	7.86	6.40	4.46	3.02	1.91	1.46	0.85	0.46	0.26



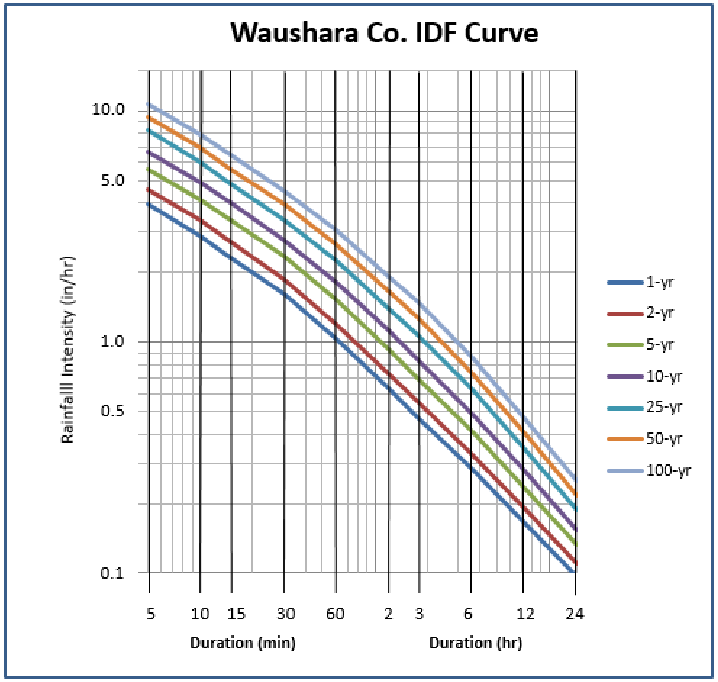
Waupaca County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.72	2.76	2.24	1.56	1.01	0.62	0.46	0.27	0.16	0.09
2-yr	4.32	3.18	2.60	1.82	1.16	0.71	0.52	0.31	0.18	0.11
5-yr	5.40	3.96	3.24	2.26	1.44	0.88	0.65	0.38	0.23	0.13
10-yr	6.36	4.68	3.80	2.66	1.70	1.03	0.76	0.45	0.26	0.15
25-yr	7.80	5.76	4.68	3.26	2.09	1.27	0.94	0.55	0.32	0.18
50-yr	9.00	6.60	5.36	3.76	2.42	1.48	1.09	0.64	0.37	0.20
100-yr	10.32	7.56	6.16	4.30	2.78	1.70	1.25	0.73	0.42	0.23



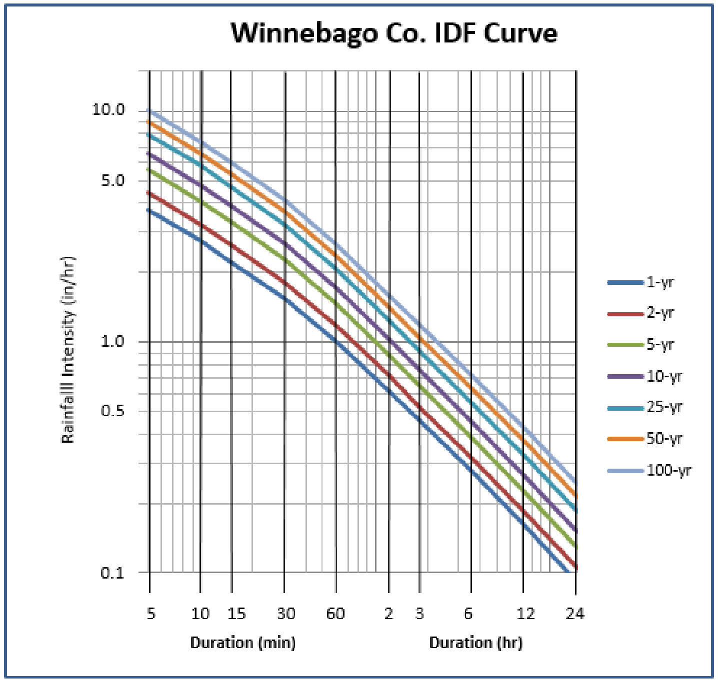
Waushara County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.96	2.88	2.32	1.62	1.03	0.63	0.47	0.28	0.17	0.10
2-yr	4.56	3.36	2.72	1.88	1.20	0.74	0.55	0.32	0.19	0.11
5-yr	5.64	4.14	3.40	2.34	1.52	0.94	0.69	0.41	0.23	0.13
10-yr	6.72	4.92	4.00	2.78	1.81	1.12	0.84	0.49	0.28	0.15
25-yr	8.28	6.00	4.88	3.40	2.26	1.41	1.06	0.62	0.34	0.19
50-yr	9.48	6.96	5.64	3.94	2.65	1.67	1.26	0.74	0.40	0.22
100-yr	10.80	7.92	6.44	4.50	3.06	1.94	1.47	0.86	0.47	0.25



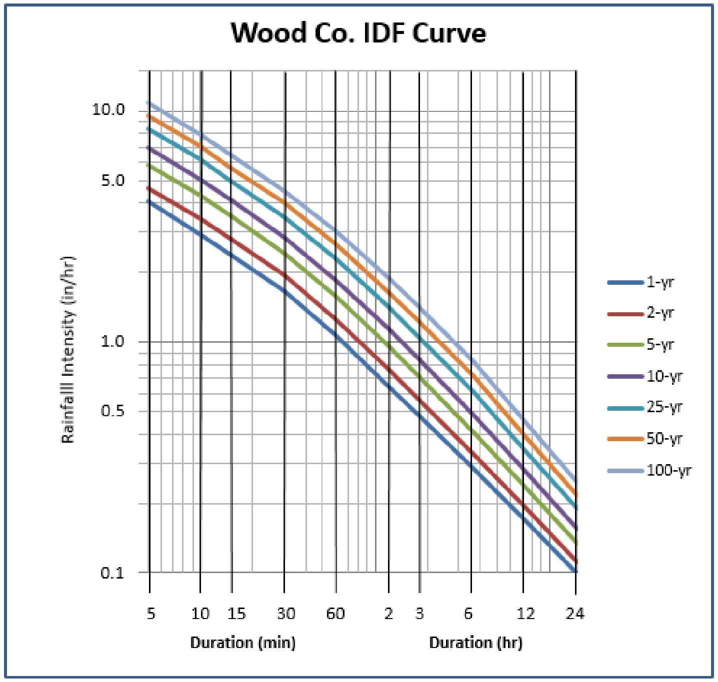
Winnebago County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	5	10	15	30	60	120	180	360	720	1440
1-yr	3.72	2.76	2.24	1.54	1.00	0.61	0.46	0.27	0.16	0.09
2-yr	4.44	3.24	2.64	1.82	1.17	0.72	0.53	0.31	0.18	0.10
5-yr	5.64	4.08	3.32	2.30	1.46	0.89	0.65	0.38	0.22	0.13
10-yr	6.60	4.80	3.92	2.70	1.71	1.04	0.76	0.45	0.26	0.15
25-yr	7.92	5.82	4.72	3.26	2.06	1.25	0.92	0.54	0.32	0.18
50-yr	9.00	6.60	5.36	3.70	2.35	1.43	1.05	0.62	0.37	0.21
100-yr	10.20	7.44	6.04	4.16	2.65	1.61	1.19	0.71	0.42	0.24



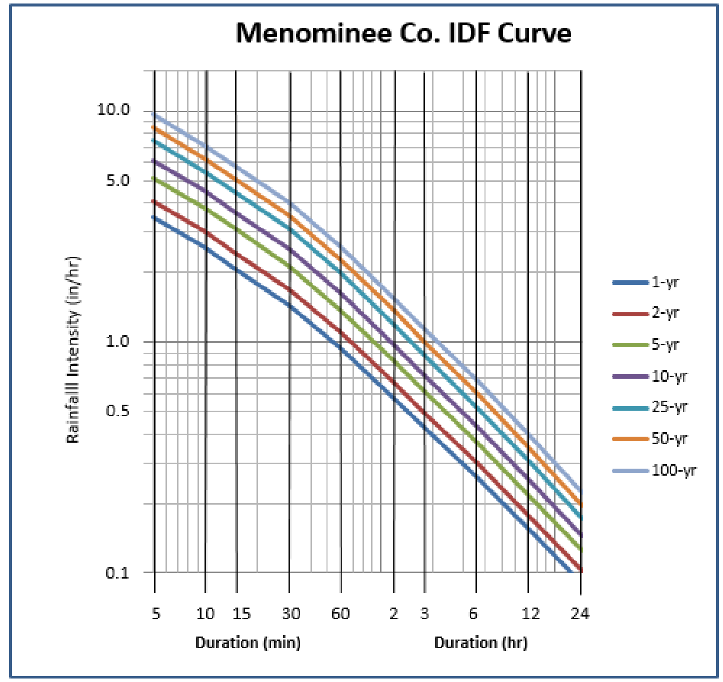
Wood County Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	4.08	2.94	2.40	1.66	1.06	0.65	0.48	0.29	0.17	0.10
2-yr	4.68	3.42	2.80	1.94	1.24	0.76	0.56	0.33	0.19	0.11
5-yr	5.88	4.32	3.52	2.44	1.57	0.96	0.71	0.41	0.24	0.13
10-yr	6.96	5.04	4.12	2.86	1.85	1.14	0.84	0.49	0.28	0.16
25-yr	8.40	6.18	5.00	3.50	2.29	1.42	1.05	0.61	0.34	0.19
50-yr	9.60	7.08	5.72	4.00	2.65	1.65	1.23	0.72	0.39	0.22
100-yr	10.92	7.98	6.48	4.54	3.03	1.90	1.43	0.83	0.45	0.25

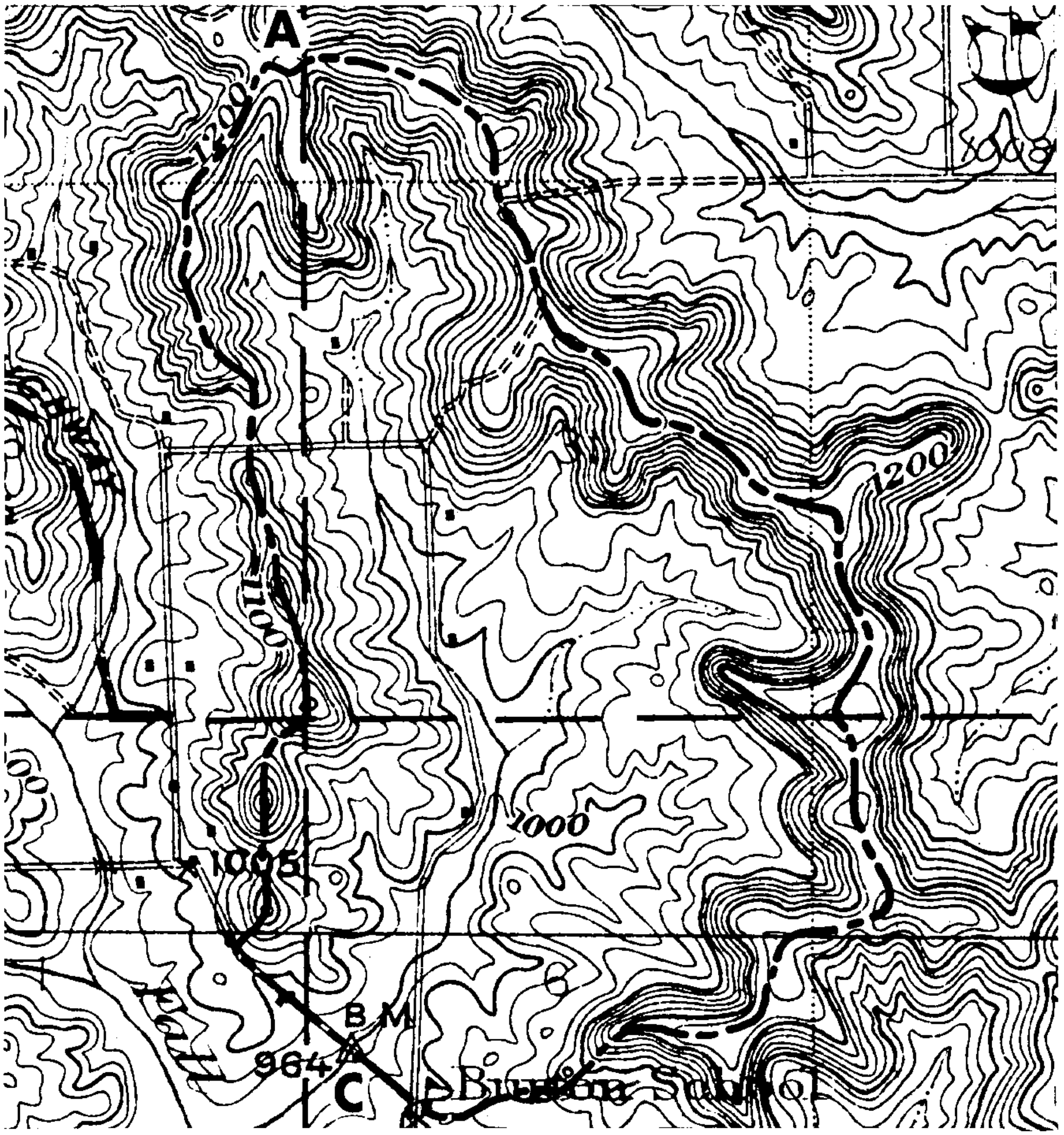


Menominee Co. Rainfall Intensity-Duration-Frequency- NOAA Atlas 14, Volume 8

	Duration (min)									
RI (yr)	**5**	**10**	**15**	**30**	**60**	**120**	**180**	**360**	**720**	**1440**
1-yr	3.48	2.58	2.08	1.44	0.94	0.58	0.43	0.26	0.15	0.09
2-yr	4.08	3.00	2.44	1.70	1.09	0.67	0.50	0.30	0.18	0.10
5-yr	5.16	3.78	3.08	2.14	1.37	0.84	0.62	0.37	0.22	0.12
10-yr	6.12	4.50	3.64	2.54	1.61	0.98	0.72	0.43	0.25	0.14
25-yr	7.44	5.46	4.44	3.10	1.97	1.20	0.88	0.52	0.30	0.17
50-yr	8.52	6.24	5.08	3.54	2.26	1.38	1.01	0.59	0.35	0.20
100-yr	9.72	7.08	5.76	4.02	2.57	1.56	1.15	0.68	0.39	0.22



CONTOUR MAP FOR EXAMPLE PROBLEM



0 2000 4000
0 1000 3000 5000
SCALE IN FEET

—— — —— INDICATES BASIN LIMITS

Location	- NW Jackson County
Drainage Basin Area	- 1067 Acres
Length	- 10,800 ft. = 2.05 mi., from inlet (C) along natural waterway to most remote point (A)
Soil	- Sandy silt loams over sand and limestone
Cover(estimated)	- 40% woods, 60% mixed cover
Design frequency	- 50 years
Contour interval	- 20 foot

Runoff Curve Number (CN) NRCS - TR55 Method

Soil Types

A. (Lowest runoff potential). Includes deep sands with very little silt and clay, also deep, rapidly permeable loess.

B. Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the type has above average infiltration after thorough wetting.

C. Comprises shallow soils and soil containing considerable clay and colloid, through less than D.

D. (Highest runoff potential). Includes mostly clays of high swelling percent, but the group also includes some shallow soils with nearly impermeable sub-horizons near the surface.

Runoff Curve Number CN

Cover	Surface Condition	Soil Type A	B	C	D
Fallow	Straight Row	77	86	91	94
Row Crops	Straight Row	70	80	87	90
	Contoured	67	77	83	87
	Contoured & Terraced	64	73	79	82
Small Grains	Straight Row	64	76	84	88
	Contoured	62	74	82	85
	Contoured & Terraced	60	71	79	82
Lequmes or Rotation Meadow	Straight Row	62	75	83	87
	Contoured	60	72	81	84
	Contoured & Terraced	57	70	78	82
Native Pasture or Range	Poor	68	79	86	89
	Normal	49	69	79	84
	Good	39	61	74	80
	Contoured, Poor	47	67	81	88
	Contoured, Normal	25	59	75	83
	Contoured, Good	6	35	70	79
Meadow (Permanent)	Normal	30	58	71	78
Woods (farm wood lot)	Sparse	45	66	77	83
	Normal	36	60	73	79
	Dense	25	55	70	77
Farmsteads	Normal	59	74	82	86
Roads	Dirt	72	82	87	89
	Hard Surface	74	84	90	92
Forest	Very Sparse	56	75	86	91
	Sparse	46	68	78	84
	Normal	36	60	70	76
	Dense	26	52	62	69
	Very Dense	15	44	54	61
Impervious Surface		100	100	100	100
Suburban Areas	Range depending on density or impervious areas as roofs, street, asphalt lots, etc.	50 to 67	67 to 80	80 to 85	85 to 90

TR-55 Graphical Discharge Method Version 1.11

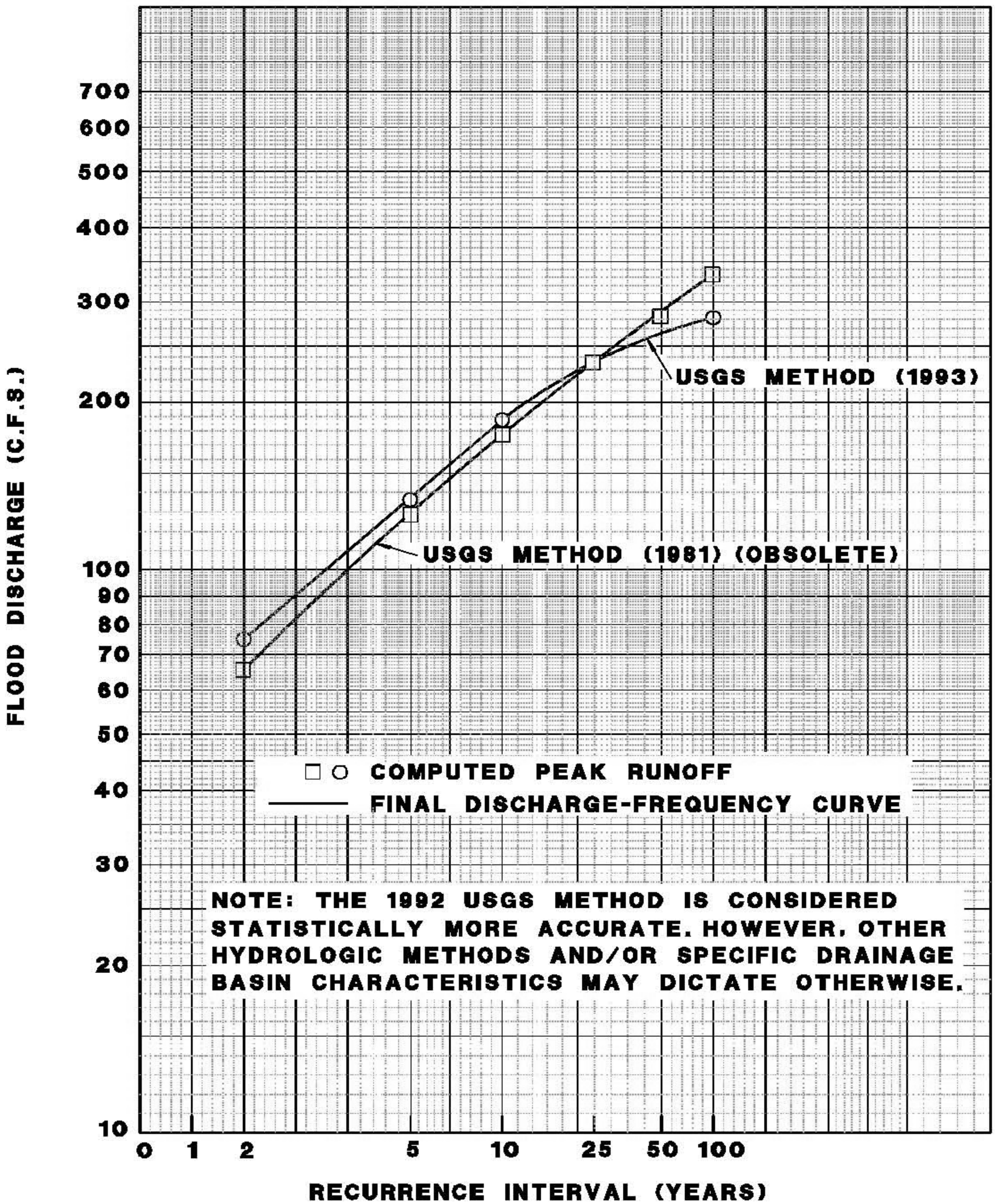
Project:	Example Problem	User: DOT	Date: 03-14-95
County:	Jackson	State: WI	Checked:_____ Date:________

Data:

Drainage Area:	1067 Acres
Runoff Curve Number:	70
Time of Concentration:	1.43 Hours
Rainfall Type:	II
Pond and Swamp Area:	None

Storm Number	1	2	3	4	5	6	7
Frequency (yrs)	1	2	5	10	25	50	100
24-Hr Rainfall (in)	2.4	2.8	3.6	4.2	4.8	5.3	6
Ia/P Ratio	0.36	0.31	0.24	0.20	0.18	0.16	0.14
Runoff (in)	0.41	0.61	1.07	1.46	1.89	2.26	2.81
Unit Peak Discharge (cfs/acre/in)	0.322	0.358	0.388	0.402	0.412	0.419	0.427
Pond and Swamp Factor 0.0% Ponds Used	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Peak Discharge (cfs)	140	232	443	628	831	1011	1277

DISCHARGE FREQUENCY GRAPH
N/W JACKSON COUNTY



Basin Watershed Data (Attachment 1A)

Project ____________________ Location__________________________ By____________ Date___________

Circle One: Present Developed_____________ Frequency (yr) _______ Checked _________ Date _______

Subarea Name									
Drainage Area A_m (mi^2)									
Time of Concentration T_c (hr)									
Travel Time through subarea T_t (hr)									
Downstream Subarea names									
Travel time summation to outlet ΣT_t (hr)									
24-hr Rainfall P (in.)									
Runoff Curve number CN									
Runoff Q (in.)									
A_mQ (mi^2 - in.)									
Initial abstration I_a (in.)									
I_a /P									

Basin Watershed Data Example (Attachment 1B)

Project *Detention Example* Location____________ By ***WisDOT*** Date____________

Circle One: Present Developed____________ Frequency (yr) ***50*** Checked ________ Date ________

Subarea Name	**#1**								
Drainage Area A_m (mi^2)	**10 ACRE =.0156**								
Time of Concentration T_c (hr)	**.3**								
Travel Time through subarea T_t (hr)	**0**								
Downstream Subarea names	**--**								
Travel time summation to outlet ΣT_t (hr)	**--**								
24-hr Rainfall P (in.)	**5**								
Runoff Curve number CN	**75**								
Runoff Q (in.)	**2.45**								
A_mQ (mi^2 - in.)	**.038**								
Initial abstration I_a (in.)	**.667**								
I_a /P	**.1**								

Hydrograph Development Work Sheet (Attachment 2A)

Project ____________________ Location ________________________________ By ___________ Date ______________

(Circle One) Present Developed _______________________________ Frequency (yr) ________________ Date ______________

Subarea Name	Basic Watershed Data used *				Select and enter hydrography times in hours **												
	Sub-area Tc (hr)	ΣTt to oulet (hr)	Ia/P	AmQ (mi2-in)	Discharge at selected hydrography time ** ---------(cfs)---------												
Composite hydrography at outlet																	

* From FDM 13-10-10, Attachment 1

** Use rainfall distribution Type II for Wisconsin

*** Hydrography discharge for selected times is AmQ multipled by tabular discharge.

Hydrograph Development Work Sheet Example (Attachment 2B)

Project Detention Example Location ______________________________ By WisDOT__ Date ______________

(Circle One) Present Developed ______________________________ Frequency (yr) 50___ Checked __________Date ___________

Subarea Name	Basic Watershed Data used *				Select and enter hydrography times in hours **												
	Sub-area Tc (hr)	ΣTt to oulet (hr)	Ia/P	AmQ (mi2-in)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8
					Discharge at selected hydrography time ** ------(cfs)------												
#1	.3	.0	.1	.038	.76	1.1	1.6	4.5	8.9	17	25.7	25.7	17.4	10.6	7.4	5.5	4.3
									HOURS								
#1	.3	0	.1	.038	13.0	13.2	13.4	13.6									
					3	2.5	2.2	1.9									
Composite hydrography at outlet																	

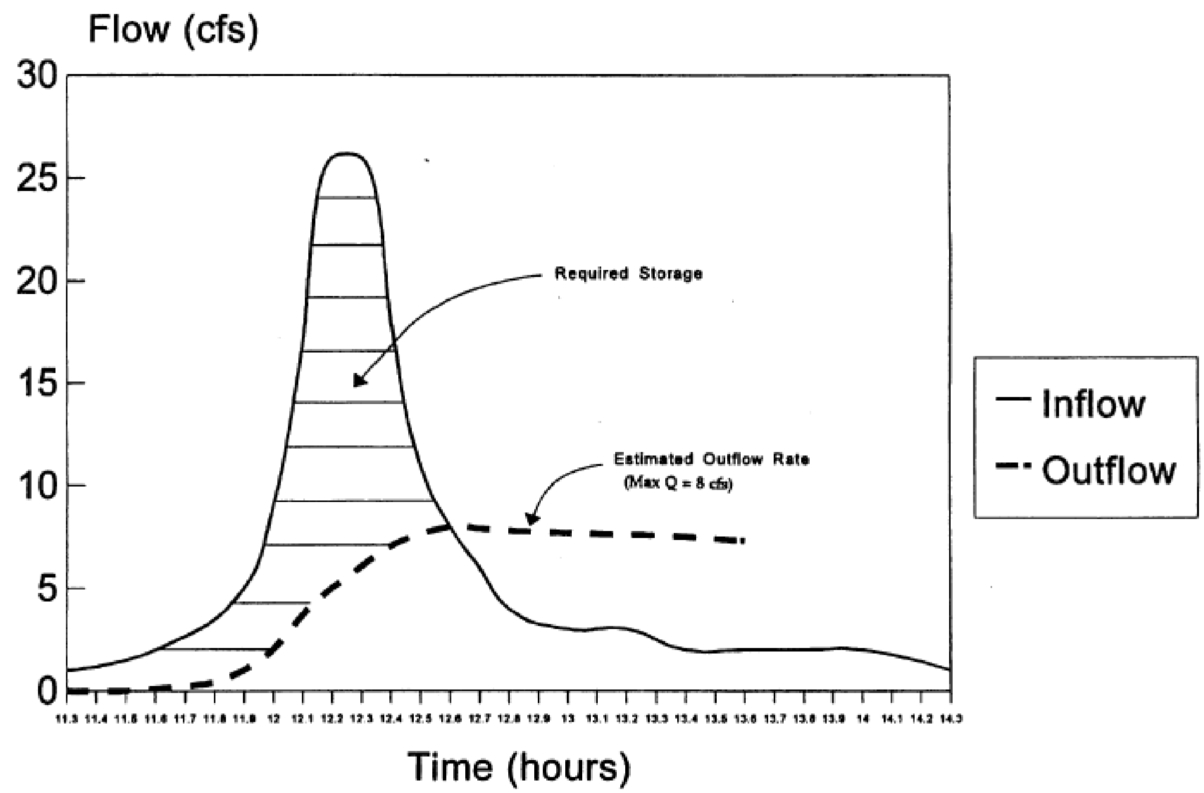
* From FDM 13-10-10, Attachment 1

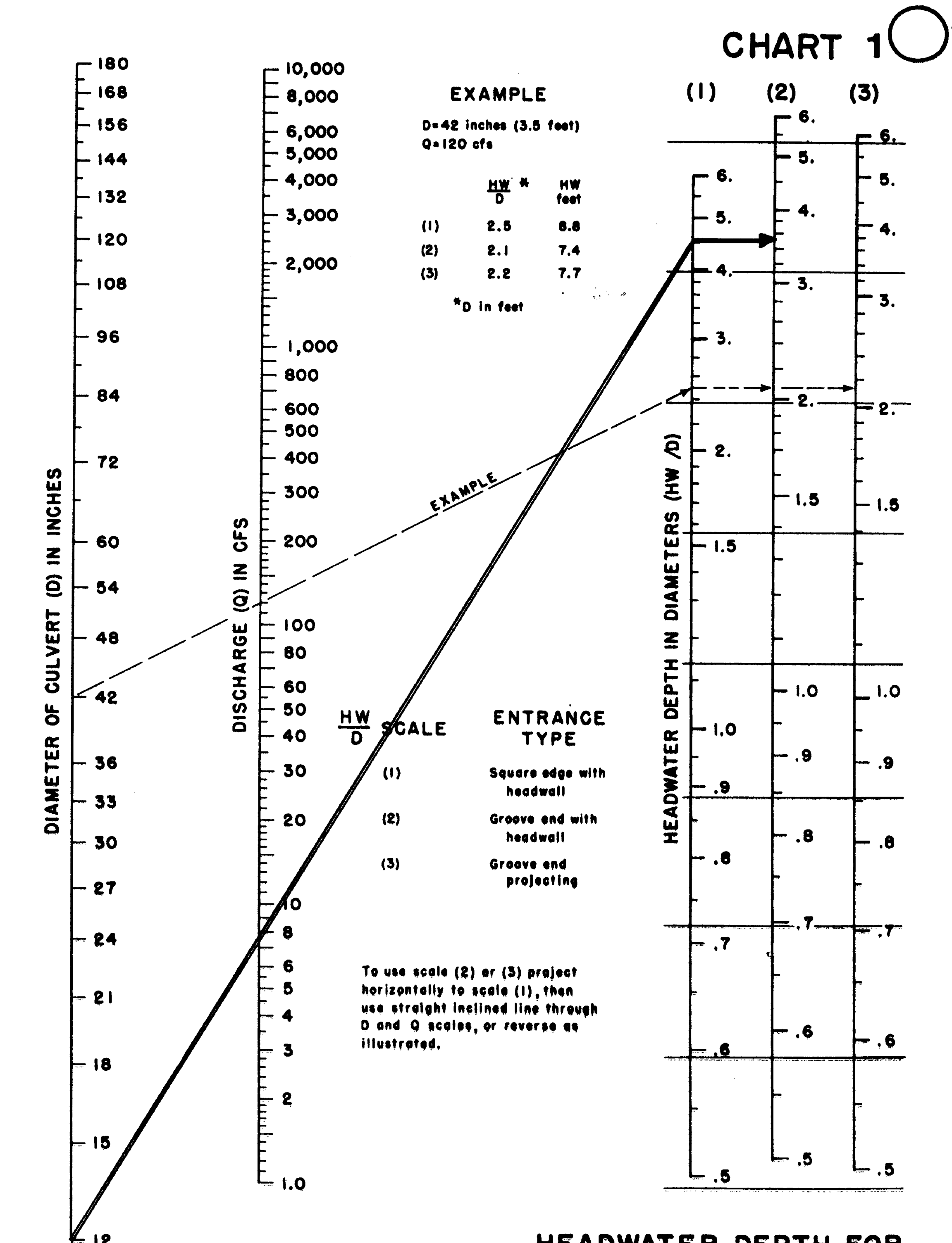
** Use rainfall distribution Type II for Wisconsin

*** Hydrography discharge for selected times is AmQ multipled by tabular discharge.

Detention Example

50 year, 24hr storm





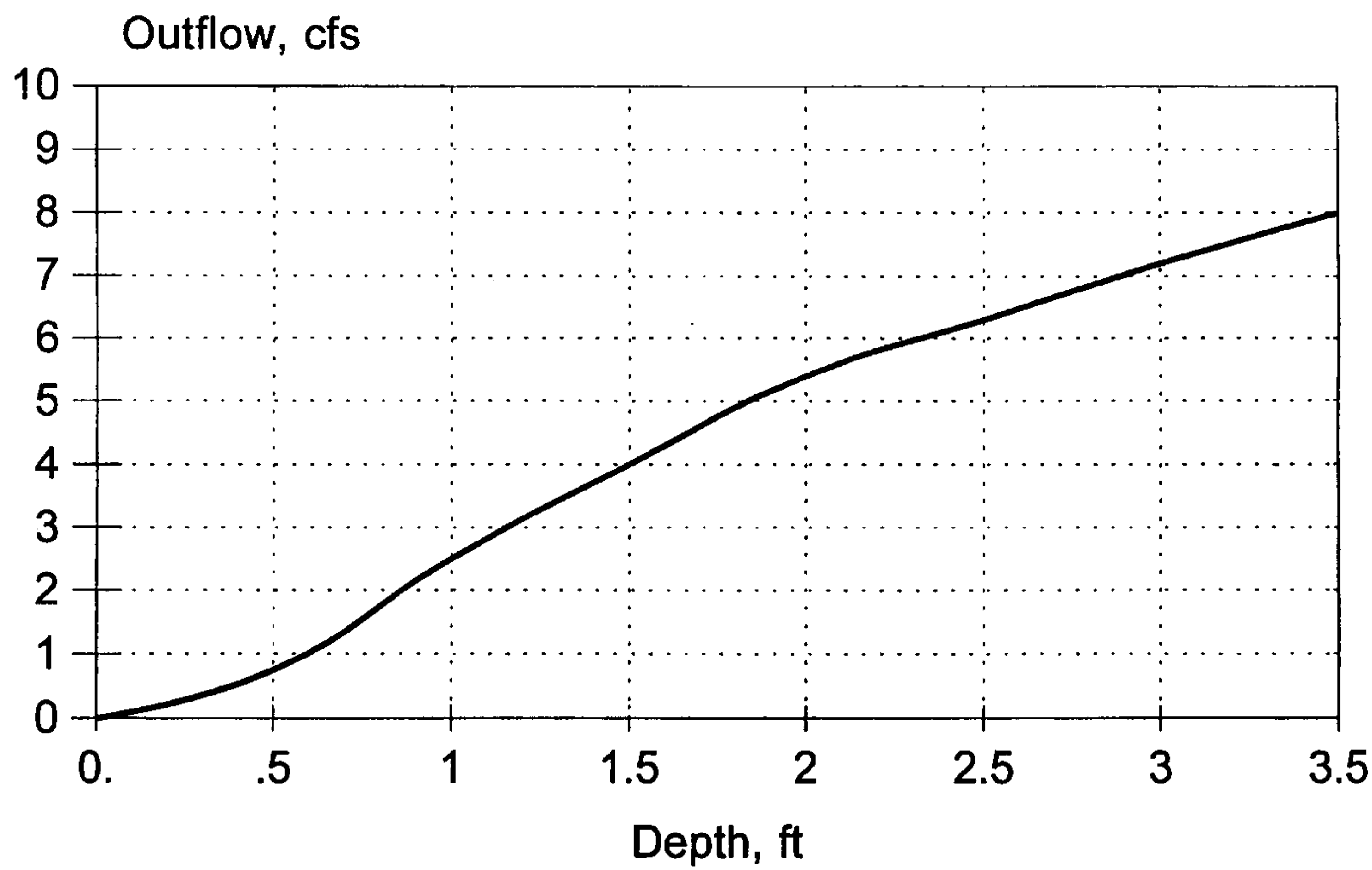
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3 REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

181

Depth - Outflow
12" RCP



Storage Indicator Curve Work Sheet (Attachment 6A)

Table Number 1

Elevation (ft) (1)	Discharge (ft) (2)	Storage (ft^3) (3)	$\frac{0_2}{2}$ (4)	$\frac{S_2}{\Delta T}$ (5)	$\frac{S_2}{\Delta T} + \frac{0_2}{2}$ (6)

$\Delta T = 360$ sec.

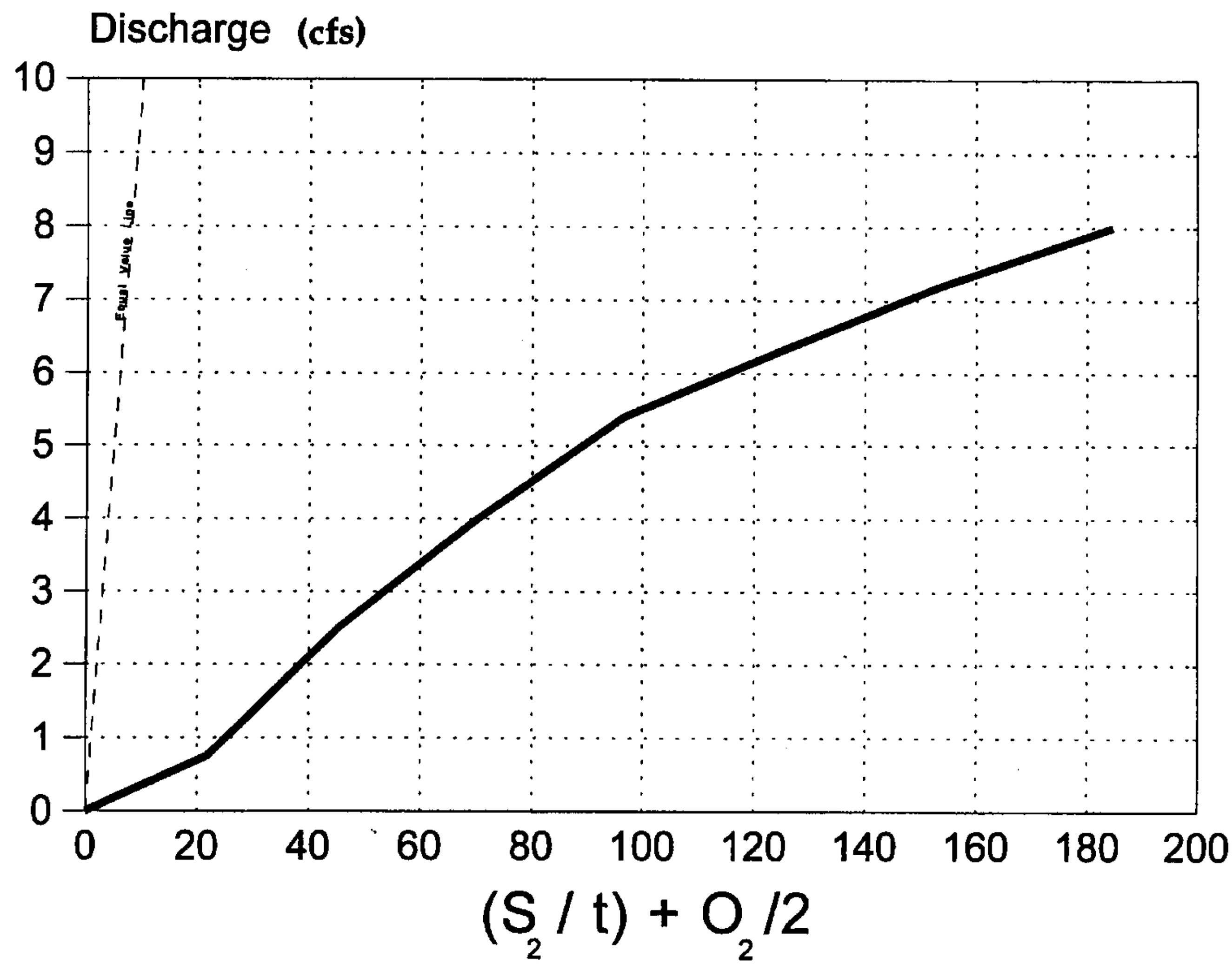
Storage Indicator Curve Work Sheet Example (Attachment 6B)

Table Number 1

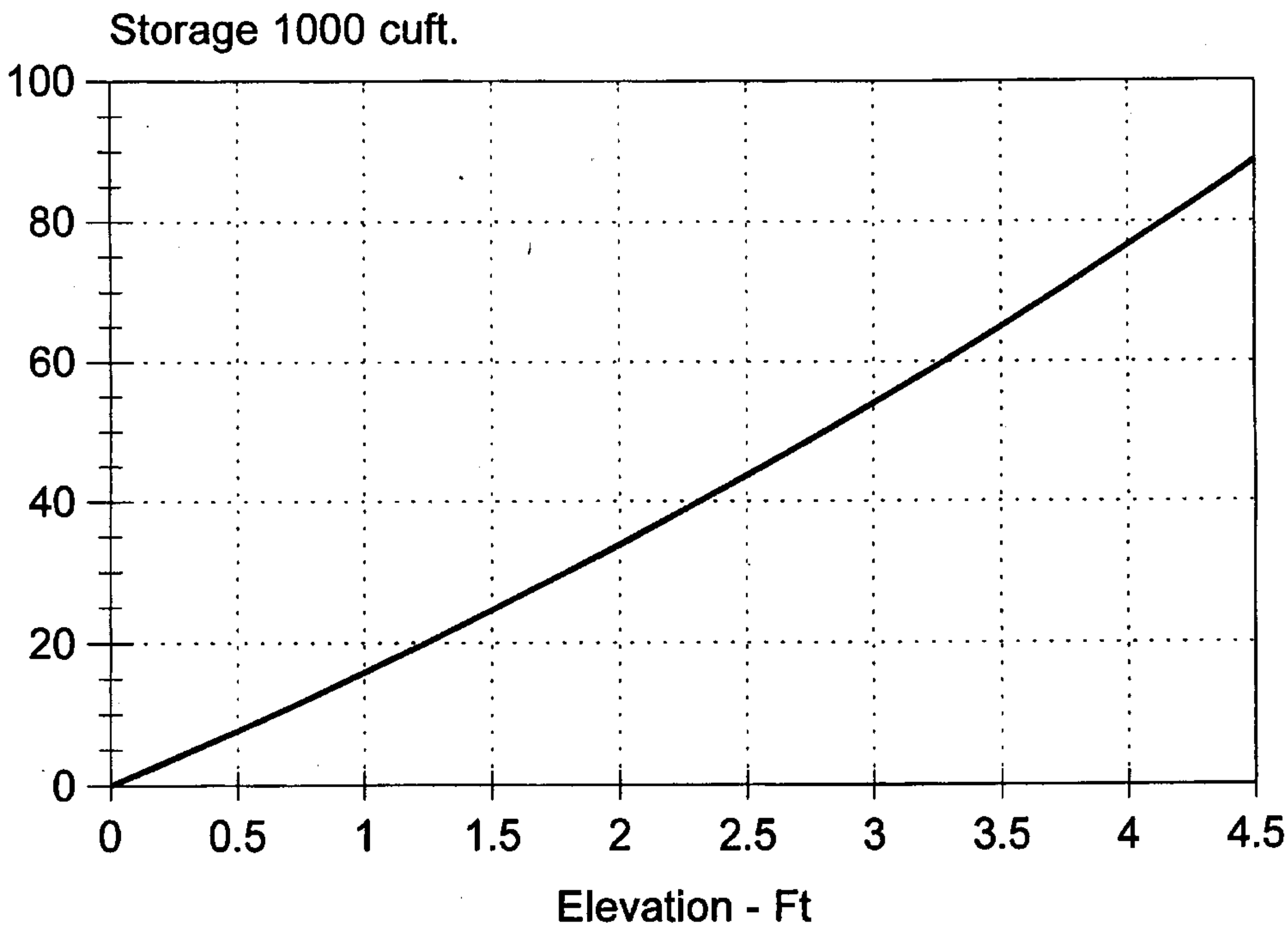
Elevation (ft) (1)	Discharge (ft) (2)	Storage (ft3) (3)	$\frac{O_2}{2}$ (4)	$\frac{S_2}{\Delta T}$ (5)	$\frac{S_2}{\Delta T} + \frac{O_2}{2}$ (6)
0.0	0.0	0.0	0	0	0
0.5	.75	7688	0.38	21.4	21.78
1.0	2.5	15881	1.25	44.1	45.35
1.5	4.0	24594	2.0	68.3	70.3
2.0	5.4	33842	2.7	94	96.7
2.5	6.3	43643	3.15	121.2	124.35
3.0	7.2	54011	3.6	150.0	153.6
3.5	8.0	64963	4.0	180.5	184.5

ΔT = 360 sec.

Storage - Indicator Curve



Stage - Storage Curve



Hydrograph Data Work Sheet (Attachment 9A)

Table Number 2

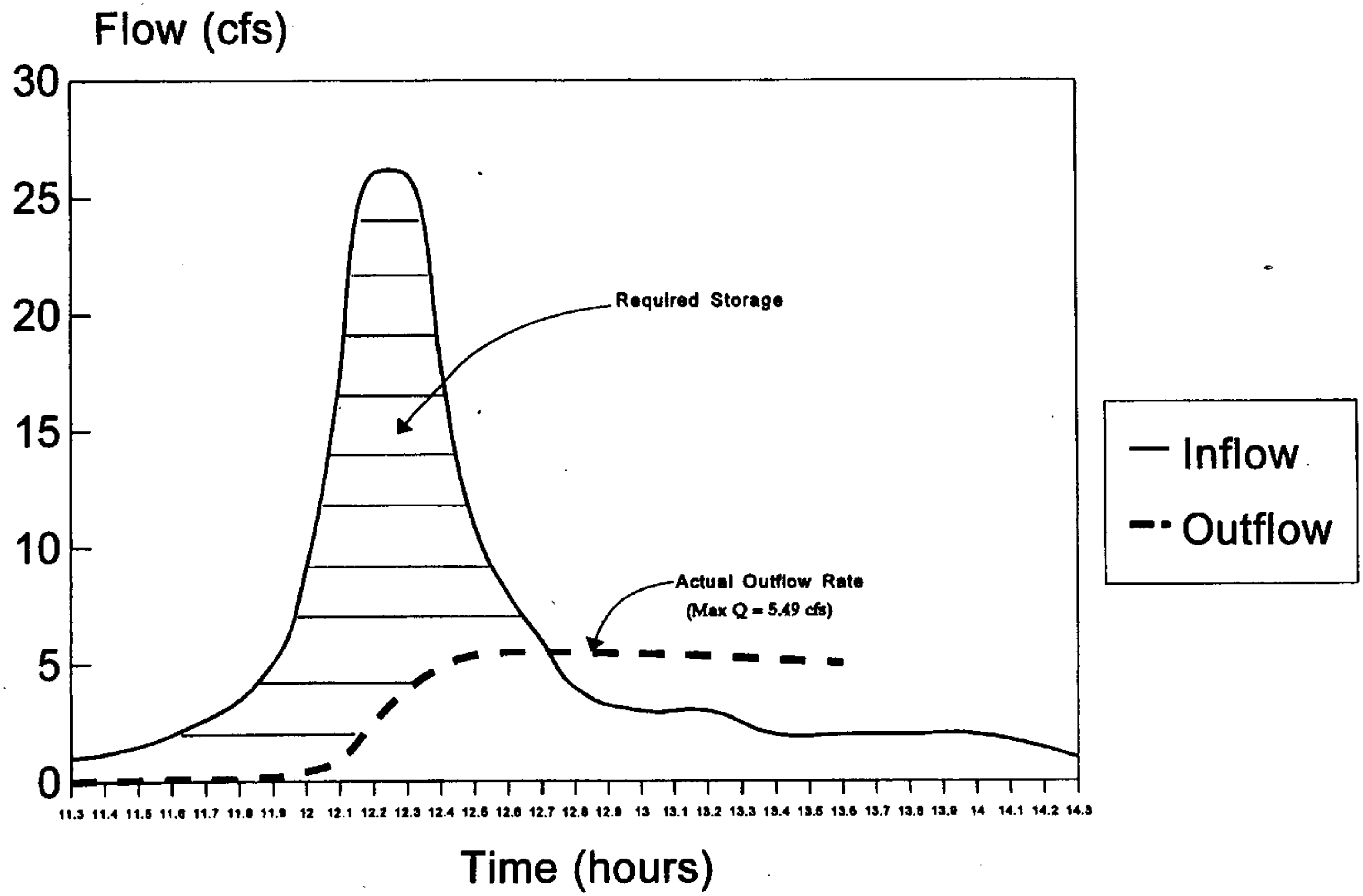
Time (hrs) (1)	Inflow (cfs) (2)	$\frac{I_1 + I_2}{2}$ (3)	$\frac{S_1}{\Delta T} + \frac{0_1}{2}$ (4)	O_1 (cfs) (5)	$\frac{S_2}{\Delta T} + \frac{0_2}{2}$ (6)	O_2 (cfs) (7)

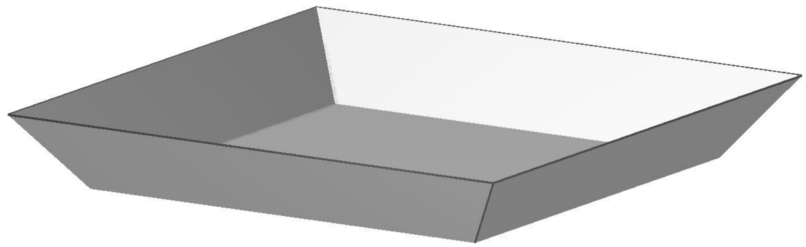
Hydrograph Data Work Sheet Example (Attachment 9B)

Table Number

Time (hrs) (1)	Inflow (cfs) (2)	$\frac{I_1 + I_2}{2}$ (3)	$\frac{S_1}{\Delta T} + \frac{0_1}{2}$ (4)	O_1 (cfs) (5)	$\frac{S_2}{\Delta T} + \frac{0_2}{2}$ (6)	O_2 (cfs) (7)
11.0	.76					
11.3	1.1	.093	0	0.0	.93	0.0
11.6	1.6	1.35	.93	0.0	2.28	0.11
11.9	4.5	3.1	2.28	0.11	5.27	0.19
12.0	8.9	6.7	5.27	0.19	11.8	0.42
12.1	17	12.95	11.8	0.42	24.3	0.94
12.2	25.7	21.35	24.3	0.94	44.7	2.52
12.3	25.7	25.7	44.7	2.52	67.9	3.89
12.4	17.4	21.6	67.9	3.89	85.6	4.84
12.5	10.6	14.0	85.6	4.84	94.8	5.37
12.6	7.4	9.0	94.8	5.37	98.4	5.47
12.7	5.5	6.45	98.4	5.47	99.4	5.49
12.8	4.3	4.90	99.4	5.49	98.8	5.47
13.0	3	3.65	98.8	5.47	97.0	5.43
13.2	2.5	2.75	97.0	5.43	94.3	5.33
13.4	2.2	2.35	94.3	5.33	91.3	5.16
13.6	1.9	2.10	91.3	5.16	88.2	5.03
13.8	1.8	1.85	88.2	5.03	85.02	4.95
14.0	1.7	1.75	85.02	4.95	81.82	5.40
14.3	1.0	1.35	81.82	4.50	78.67	4.40

Detention Example
50 year, 24hr storm





Given :

side slopes of pond = 4:1 = Z

depth = D = 3.5 feet, use 0.5 foot increments

dimensions of pond bottom = W = L = 122 feet

122 feet x 122 feet

For a trapezoidal basin:

Using equation: $Volume = LWD + (L + W)ZD^2 + 4/3Z^2D^3$

$Volume_{1.5\ feet} = (122)(122)(1.5) + (122+122)(4)(1.5)^2 + 4/3(4)^2(1.5)^3$

$Volume_{1.5\ feet} = 24594\ ft^3$

FDM 13-15-1 Economic Analysis

August 8, 1997

1.1 Introduction

It is desired practice to keep headwater at a minimum. However, economic design in many instances requires that the pipe flow at least full or with some headwater. Full-flow structures or structures designed to flow full under certain conditions are subject to close study. Full-flow culverts that are so designed to increase headwater and reduce the size of the culvert, and therefore reduce the cost, result in a savings for the conduit itself; but many effects that cannot be accurately determined enter into the economic design of culverts with headwater. Each individual location should be analyzed to determine the allowable headwater for a specific design frequency.

The economic design of culverts with headwater for a specific design frequency requires that consideration be given to the following effects:

1. Hydraulic uplift or buoyancy, which is especially significant for large pipes in permeable soils and/or pipes with no headwalls. This danger is augmented when the culvert entrance becomes blocked with debris.
2. Accuracy of the estimate of design discharge. When determining allowable headwater depth, the designer should keep in mind that the estimate of design discharge is an approximation.
3. Exfiltration from pipes due to pressure.
4. Erosion of the embankment due to receding headwater.
5. Danger of transverse seepage through fills, especially in side hill locations.
6. Debris protection.
7. Maintenance.
8. Damage to property.
9. Hazards to life.
10. Public image.
11. Flooding of land affected by headwater. It must be understood that raising headwater over the levels of former flooding is to be avoided.
12. The installation cost includes the pipe, structure excavation, backfill, and other special features, and occasionally maintenance costs that occur on a regular, predictable basis, or specific material features that offer additional resistance to such things as silting, sliding, rupture, or corrosion.

FDM 13-15-5 Design Criteria

August 8, 1997

5.1 Introduction

In this procedure the criteria for the hydraulic design of culverts are discussed under two broad headings--Culvert Location, and Structure Size Selection. When designing culverts, the hydraulic design engineer should employ these criteria along with FDM 13-15-10, "Culvert Hydraulics," of this manual.

A Structure Survey Report containing the site data information should be submitted to the Structures Design Section in the Bureau of Highway Development for the design of all cast-in-place culverts (refer to Bridge Manual Chapter 6 for reporting procedures). In addition, each cast-in-place culvert requires a Hydraulic Report as described in Bridge Manual Chapter 8.

5.2 Culvert Location

The culvert location should be selected so the culvert passes the expected discharge with as little interruption as practical. Where water is confined in a channel, the culvert should be located at or near the point where the channel reaches the project and as much in line with the channel as possible. Where other considerations indicate a less desirable location, the roadbed and special ditch must be protected against turbulence generated by the change in direction of flow.

When a highway is to be reconstructed essentially on the location of the existing highway, the engineer shall evaluate old culvert locations for possible culvert replacement. This will aid in minimizing changes in existing drainage conditions on private lands.

When a highway is to be constructed on relocation, the designer shall provide a culvert wherever there is an appreciable natural draw or depression. If there are no significant draws or depressions, culverts shall be placed so as not to collect and concentrate a large drainage flow.

5.3 Structure Size Selection

In general, pipe drainage structures shall be selected in accordance with the current culvert selection standard (refer to FDM 13-1-15). The size of culvert may be chosen knowing the following data:

1. Estimated runoff (Q).
2. Approximate length and slope of culvert.
3. Allowable headwater depth in feet, which is taken as the vertical distance from the conduit flow line at the entrance to the water surface in the channel.
4. Entrance type. The type of entrance must be predetermined by the designer.
5. Barrel cross-sectional shape. Determined considering available headroom.
6. Barrel roughness factor. The roughness factor that produces the largest size pipe should be used when alternate materials are allowed at the contractor's option.
7. Tail-water conditions known or computed. Tail water is defined as the distance in feet from the outlet invert to the water surface in the outlet channel.

Following is a detailed discussion of the minimum required data for culvert design along with a discussion of additional design criteria required to perform a thorough hydraulic analysis of a culvert.

5.3.1 Minimum Pipe Size

In accordance with Department of Transportation practice, the minimum size of pipe for culvert cross drains shall be 24 inches, except that on multi-lane highways in fill of 10 feet or more, the minimum size shall be 30 inches.

5.4 Allowable Headwater

As noted previously, existing field conditions and channel geometry will determine a maximum depth of water that can be tolerated at the entrance of a culvert. This depth of water is known as the maximum allowable headwater depth. The information that has been accumulated during the field review of the culvert crossing can now be used to determine the maximum allowable headwater depth for the structure. When the highway profile is established, the headwater depth may also be controlled by the low point in the roadway subgrade or by high points in the roadway ditches. Generally, the maximum high-water elevation should not be higher than the subgrade shoulder point.

Sometimes field conditions will dictate a depth of headwater that is too low to allow an economical design for the culvert crossing. When this occurs, the engineer may consider the use of artificial conditions that will allow a greater depth of headwater to develop. These artificial means are berms or dikes at the inlet ends of culverts, or a depressed profile for the culvert. These artificial means of forcing a headwater depth should not cause any appreciable increase of existing flooding conditions upstream from the culvert.

5.5 Design Freeboard and Headwater-to-Depth Ratio

The headwater depth at the inlet of a pipe culvert is normally expressed as a ratio (HW/D) where HW is the total depth of water (measured from the invert of a culvert) and D is the interior height of the culvert barrel.

The design ratio can be as high as 1.50 for the culverts under 15 feet in diameter or rise. The design ratio for culverts in excess of 15 feet in diameter or rise should be 1.00. Smaller ratios for pipes less than 15 feet may be justified by safety factors of flooding conditions, velocities, scouring, economy, etc. If damage to the culvert is anticipated, or if adverse flooding conditions will be caused upstream of the culvert from the accumulation of ice and debris, the headwater-depth ratio shall be reduced. The reduction of the ratio shall be sufficient to allow an increase in the design flow capacity and freeboard (if needed) at the entrance of the structure to eliminate flood damage or to reduce it to within acceptable limits.

Since the advent of the large sections for round pipe, it has become economical in special cases to design for these culverts with a low headwater to depth ratio. In essence, the headwater to depth ratio plays no part as a control for the design of these culverts, but instead economics is the major controlling factor. To illustrate this point, a large flow rate with a shallow allowable headwater may dictate using two or more corrugated structural plate pipes or round pipes side by side. The headwater-depth ratio of these pipes would be equal to one. A more economical design may be the use of a large round pipe with the water using a small portion of the cross-sectional area of the pipe and with the hydraulics of the stream channel flow being satisfied. In this case, a close analysis of the economics of the situation would be the controlling factor and not the headwater to depth ratio.

Note: Except in special cases where debris and ice are a problem or possible upstream flooding cannot be tolerated, allowing a freeboard is not necessary for pipes up to 20 feet in span. A two-foot freeboard is desirable for single pipe drainage with structures greater than 20 feet in span (bridges). For freeboard requirements over navigable waterways the designer is referred to the Wisconsin Administrative Code, Chapter NR 320, "Bridges and Culverts in or Over Navigable Waterways."

5.6 Inlet Treatments

The shape, geometry, and skew of an inlet all affect culvert capacity. As stated in Hydraulic Design Series (HDS) No. 5 (1): The inlet edge configuration is a major factor in inlet control performance and it can be modified to improve performance.

In outlet control, the type of inlet has some affect on capacity but generally the edge geometry is less important than in inlet control. The skew of the entrance has some affect on capacity but the result is minor.

For culverts flowing with inlet control, the constriction at the inlet may limit the flow that the culvert can carry in comparison to its potential barrel capacity. Performance curves can be used to advantage, as explained in HDS No. 5 (1), to obtain the most efficient balanced design for a given culvert size. Considerable improvements may sometimes be made in a culvert's performance by using a depressed and/or improved inlet. Performance curves are particularly useful in comparing the desirability of alternate culvert designs and the ability of the culverts to accommodate flows in excess of the design discharge.

Further reference is made to the design of the entrance for the various types of culverts in chapter III of HDS No. 5 (1) The entrance loss coefficient (K_e) varies from 0.2 to 0.9 depending on the type of structure and the configuration of the entrance; 0.2 applies to concrete pipe with the socket end projecting from fill, and 0.9 applies to a corrugated metal pipe or pipe arch projecting from the fill with no headwall (refer to Attachment 5.1).

5.7 Improved Inlets

An improved inlet is a means of increasing the capacity of a given culvert pipe size without raising the headwater. Their use has resulted in considerable savings on various projects throughout the United States. These savings have resulted primarily when inlets are used with reinforced concrete box structures. When used with corrugated metal structures, the savings have been very small. The cost savings for a single pipe installation will not usually be as great as for a large box culvert. However, the potential cost savings in achieving balanced designs on many pipe culverts by using improved inlets may be very significant. The savings in these structures are a result of reducing the size of the barrel of the structure by using a more efficient inlet. Conventional culvert inlet configurations are as listed on page 5 of HDS No. 5 (1) (for example, projecting from fill, end section conforming to fill slope, etc. Improved inlet configurations include bevel-edged, side-tapered, and slope-tapered inlets.

The <u>bevel-edged inlet</u> is the most economical method of improving the capacity of a conventional culvert. The addition of bevels to a conventional culvert with a square-edged inlet increases culvert capacity by five to 20 percent.

Note: Bevels should be used on all cast-in-place culvert entrance headwalls, both conventional and improved inlet types.

The <u>side-tapered inlet</u> consists of an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. This type of inlet increases flow capacity of a conventional culvert with a square-edged inlet by 25 to 40 percent.

The <u>slope-tapered inlet</u> increases the flow capacity of a side-tapered inlet by also providing a fall within the enclosed entrance section. This means that more head is available at the throat section (point at which the improved inlet joins the culvert barrel). This type of inlet can have over a 100 percent greater capacity than a conventional culvert with square edges.

Since culverts operating in outlet control are usually flowing full, an improved inlet will not increase its capacity. However, culverts in inlet control lie on relatively steep slopes and flow only partly full, and will exhibit marked increases in capacity with the use of an improved inlet.

Improved inlets are not prefabricated or precast; therefore, an expensive cast-in-place, rectangular, side-tapered inlet would be a logical alternate. Obviously, the increased cost of using an improved inlet may outweigh the increased cost of using a larger culvert size. Therefore, when proposing an improved inlet, a thorough economic analysis of all feasible designs must be performed. In general, improved inlets can be economically justified for long culverts and for existing culverts that require more capacity.

Note: Reducing the structure size by utilizing improved entrance design is generally not practiced; but its practice is appropriate in very large, very long or very costly culvert installations.

For further information on improved inlets, refer to HDS No. 5 and FHWA Technical Advisory T 5140.6, "Improved Inlets for Culverts-Example Structural Plans," January 16, 1979.

5.8 End Protection

Preformed metal, aluminum, or reinforced concrete apron endwalls should be used at the inlet and discharge end of all single installation culvert pipe 84 inches and under in diameter, and pipe arch 72 inches and under in span. This applies to all cross drains, private entrances, and median installations. For larger pipe sizes or installations of two or more pipes, use concrete masonry endwalls as shown in SDD, "Concrete Masonry Headwalls."

5.9 Type, Shape, and Roughness of Culvert

Culverts are comprised of different types of material and different shapes, both of which have differing degrees of roughness. The predominant types of material for culverts are corrugated steel, precast concrete, and cast-in-place concrete.

The factor that describes roughness of culverts is usually expressed as "Manning Roughness Coefficients," designated by the letter "n." Table XIV on page 84 of "Design and Construction of Sanitary Sewers," ASCE and WEF, sixth printing, 1991 gives a listing of roughness coefficients (Darcy-Weisbach, Manning, and Hazen-Williams) to be used for conduits.

The various shapes of culverts are round, oblate, oval, and rectangular. The most efficient shape of culverts hydraulically may not be the most efficient shape economically. The preference for use of one shape over another will depend on the structural capabilities and the type of material composition of the culvert. Also, the hydraulic efficiency of the various shapes of culverts with or without coatings or linings and the final economic analysis of the various combinations of shapes, roughness factors, and hydraulic efficiencies will determine the final selection of the culvert for a specific location.

5.10 Design Tail Water

A culvert that is designed to flow under outlet control is affected by a tail-water at the outlet. Tail-water depth is the depth of water at the outlet of a structure that will affect the flow of water through a structure. The information regarding the tail water recorded during the field review of the drainage areas will now be used in the design of the culvert, either as a controlling factor or as a check factor, to ensure that the culvert is flowing under inlet control.

For the controlling cross section of an outlet channel, the tail-water depth may be approximated by the normal depth of flow as computed by Manning's Equation.

The design formulas and charts that will be needed by the engineer to design a culvert with a tail-water control are given in HDS No. 5 (1) and discussed in FDM 13-15-10.

5.11 Allowable Velocity

Culverts shall be placed, if possible, at or near the critical slope, as determined from FHWA "Open Channel" charts, in order to obtain maximum capacity. Culvert outlet velocities shall be computed for all pipes to determine whether scour will occur. These computed velocities should then be compared with outlet velocities of alternate culvert design, existing culverts in the area, or the natural stream velocities to determine the need for channel protection. In addition, the designer is referred to Chapter III-Culvert Outlet Velocity and Chapter V-Outlet Scour Computation of H.E.C. #14 (3).

5.12 Depth of Flow

For inlet control, the depth of flow in the pipe may be determined by using Manning's Equation:

$$\frac{V = 1.49\ R^{2/3}\ S^{1/2}}{n}$$

However, the determination of flow depth by this formula involves a process of trial and error, and hence the depth of flow and velocity is determined through the use of the design charts in HDS #3 (3).

For outlet control, the depth of flow determined by the above formula may be drowned out by the tail-water depth; that is, the computed depth may apply near the inlet end of the conduit, but near the outlet end the depth will be higher than the computed depth.

5.13 Check Discharges

Check discharges are used during the design of culvert crossings and should be investigated for all drainage structures. The use of a check discharge is to determine the safety factor or lack of a safety factor being incorporated into the culvert crossing. When definite danger to the safety of the traveling public or extensive and costly property damage can develop from surcharging a pipe with flood flows equivalent to the check discharge, necessary precautions must be taken to alleviate the possible dangers

5.14 References

1. U.S. Department of Transportation Federal Highway Administration, Hydraulic Design of Highway Culverts Hydraulic Design Series No. 5, 2001, 348 pp

2. U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular #14, Washington, D.C., 2006

3. U.S. Department of Transportation Federal Highway Administration, Design Charts for Open-Channel Flow Hydraulic Design Series #3, Washington, D.C., 1961, 105 pp.

LIST OF ATTACHMENTS

FDM 13-15-10 Culvert Hydraulics

March 9, 1998

10.1 Introduction

The designer must be aware of the design aids available for designing hydraulic structures operating with inlet or outlet control, or with improved inlets. In addition, selecting the most economical and feasible hydraulic structure for a particular stream crossing is facilitated by constructing a performance curve for each of the viable hydraulic structures. This procedure gives references for the required design aids, a general discussion of inlet-outlet control along with sample problems, a short discussion on where to find literature and design aids for improved inlets, and a general discussion on using the culvert performance curve.

10.2 Available Design Aids

The hydraulic design aids used in this manual are contained in Hydraulic Design Series (HDS) #5 (1) and Hydraulic Design Series (HDS) #3. It is recommended that any designer using these charts should first thoroughly study the above-listed publications.

Culvert designs can be accomplished by using the following types of charts: the capacity charts the inlet and outlet control nomographs from critical depth charts. They are grouped by type and shape pipe in HDS #5 (1).

A direct solution of the Manning Equation for rectangular open channels and closed circular pipe channels can be obtained with the aid of HDS #3 (2), Charts 1 to 14, and Charts 35 to 51, respectively.

Culvert design can also be accomplished through the use of computer programs. HY8, which is part of FHWA's hydrain package, is a suggested computer program to use for designing:

1. Pipe arch culvert.
2. Elliptical pipe culvert.
3. Circular pipe culvert.
4. Concrete box culvert and other culvert shape.

Moreover, the HY8 computer program can design one or more of the inlet configurations that follow:

1. Conventional Inlet.
2. Bevel-Edged Inlet.
3. Side-Tapered Inlet (circular or rectangular).
4. Slope-Tapered Inlet.

Further literature on the HY8 computer program and other HYDRAIN computer programs, can be found in the

HYDRAIN. Integrated Drainage Design Computer System, Version 5.0, Volumes I to VII, January 1994 manual.

10.3 Inlet-Outlet Control

Laboratory tests and field observations show two major types of culvert flow: (1) flow with inlet control and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert.

The controlling factors for inlet control are:

1. Inlet Area.
2. Inlet Shape.
3. Inlet Edge Configuration.
4. Allowable Headwater.

The controlling factors for outlet control are:

1. Inlet Area.
2. Inlet Shape.
3. Inlet Edge Configuration.
4. Allowable Headwater.
5. Tail Water Elevation.
6. Slope of Culvert.
7. Roughness of Culvert.
8. Length of Culvert.
9. Area of Barrel.
10. Barrel Shape.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth (HW) is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth and velocity head). Because of the low velocities in most entrance pools and difficulty in determining the velocity head for all flows, the water surface and the energy line at the entrance are assumed to be coincident.

See Attachment 10.1 for a graphical depiction of the energy balance on a culvert pipe in outlet control. The letters in Attachment 10.1 that are not easily self-explanatory are defined as follows:

He = entrance loss

Hf = friction loss

Hv = velocity head

Inlet Control Problem:

Given: Culvert Length = 200 feet

Culvert Slope = 2 percent

Allowable Headwater = 7.5 feet

Design Discharge = 190 cfs

Find: The minimum required pipe sizes for corrugated metal pipe and concrete pipe.

Solution: Refer to Attachment 10.2 for the solution that follows:

Corrugated Metal Pipe

1. Headwater Depth for metal pipe culverts with inlet control, HDS No. 5 Chart 2.
   a. Assume projecting entrance
   b. Try a 66” metal pipe using chart 2. HDS No. 5.
      HW/D = 1.25
      HW = 1.25 x 5.5 ft. = 6.9 ft.

Concrete Pipe

See Attachment 10.2 for the solution, which follows the same methodology used for the corrugated metal pipe solution.

Assume groove end projecting

try a 60” concrete pipe using chart 1. HDS No. 5.

HW/D = 1.3

HW = 1.3 x 5 = 6.5 ft.

Conclusions:

At this drainage crossing, either one of the following designs can be used:

1. A 66-inch CMP with a projecting entrance that produces a headwater of 6.9 feet or,
2. A 60-inch concrete pipe with a grooved edge projecting entrance that produces a headwater of 6.5 feet

The final selection should be based on an economic analysis and/or established policy and/or outlet protection required.

Outlet Control Problem:

Given: Culvert Length = 200 feet

Culvert Slope = 0.2 percent

Allowable Headwater = 7.0 feet

Design Discharge = 190 cfs

(322.8 m3/min)

Find: The minimum required pipe sizes for corrugated metal pipe and concrete pipe.

Solution: Refer to Attachment 10.2 for the solution that follows:

Corrugated Metal Pipe

1. Head for standard metal pipe culverts, n = .024 Chart number 6, HDS number 5
   1. assume projecting entrance
   2. entrance loss coefficient (K_e = .9)
   3. from chart 6, HDS number 5, 84 in. pipe, 200 feet long H = 1.4 feet
   4. from chart 4, HDS number 5, critical depth (d_c) is 3.6
   5. h_o equals (d_o+ D)/2 equals 5.3, or TW equals 3, whichever is greater
   6. actual HW equals H + h_o- L_{so} equals 6.3 feet

Concrete Pipe

See Attachment 10.2 for the solution, which follows the same methodology used for the corrugated metal pipe solution.

Conclusions:

At this drainage crossing, either one of the two following designs can be used:

1. An 84-inch cmp with a projecting entrance that produces a headwater of 6.3 feet; or,
2. A 60-inch concrete pipe with a groove-edged projecting entrance that produces a headwater of 6.8 feet.

The final selection should be based on an economic analysis and/or established policy and/or outlet protection required.

10.4 Discharge Velocity

A culvert pipe, because of its hydraulic characteristics, increases the velocity of flow over that in the natural channel. The erosive potential of this discharge velocity can be ascertained by comparing this velocity with existing culverts in the area or the natural stream velocities. Normally, changing the culvert size will not appreciably change the discharge velocity.

For culverts with supercritical flow (culverts in inlet control), the outlet velocity can be calculated by using Manning's Equation:

$$V = \frac{1.49\ R^{2/3}\ S^{1/2}}{n}$$

For culverts with subcritical flow (normally culverts in outlet control), the discharge velocity is equal to the discharge divided by the cross-sectional area of flow at the outlet. This flow area can be either that corresponding to critical depth, if $d_c > tw$; tail water depth, if $tw > d_c$ and $tw < d$; and diameter of pipe, if $d < tw$.

For a more detailed discussion of culvert discharge velocities, the designer is referred to FHWA H.E.C. #14 (3).

Discharge Velocity Problems

Inlet Control: See Attachment 10.2 (Inlet) for all pertinent data.

1. From the above equation, for Q equals 190 cfs,, D equals 66 inches, and n equals 0.024, flow velocity in the pipe is 11 fps from chart, HDS number 5, the critical depth d_c is 6 feet.

Outlet Control: See Attachment 10.2 (Outlet) for all pertinent data.

1. For an 84-inch cmp, dc equals 3.6 feet tw equals 3.0 feet, and d equals 7.0 feet. Therefore, the critical depth of 3.6 feet is used to compute the cross-sectional flow area at the outlet of the pipe.
2. D flowing/Diameter equals 3.6/7.0 equals 0.51. From Attachment 10.3, with d/D equals 0.51, read A/A_{full} equals 0.51.
3. A equals A_f x .51 equals π x $(7/2)^2$ x .51 equals 19.63 s.f.
4. V equals Q/A equals 190 cfs/19 63 s.f. equals 9.7 fps

10.5 Improved Inlets

For a further discussion of improved inlets, the designer should read the section on improved inlets contained in FDM 13-15-5 of this manual.

For further information on improved inlets, including sample problems, the designer is referred to FHWA HDS number 5 (1). In addition to manual methods for designing improved' inlets, the designer may elect to use the previously mentioned computer program entitled “HY8.”

10.6 Culvert Performance Curve

A performance curve for a culvert is a plot of discharge versus headwater depth or elevation (stage). It is a means of ascertaining at a glance how a particular culvert will operate over a range of discharges. In particular, the performance curves of alternate culvert designs are used to evaluate the potential for damage to the highway and adjacent property from floods greater than the design discharge.

See Attachment 10.3 for a schematic performance curve for a culvert with either a side-tapered or slope-tapered inlet. HDS number 5 (1) explains this performance curve as follows:

> "Each potential control section (face, throat, and outlet) has a performance curve, based on the assumption that a particular section controls the flow. Calculating the plotting the various performance curves results in a graph similar to (figure 3)," containing the face control, throat control and outlet control curves. The overall culvert performance is represented by the hatched line.

In a like manner, for conventional culverts in the lower discharge range, inlet control governs; and in the higher discharge range, outlet control governs.

10.7 References

1. U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series #5 McLean, Virginia, 1985, 235 pp.

2. U.S. Department of Transportation Federal Highway Administration, Design Charts for Open Channel Flow, Hydraulic Design Series #3, Washington, D.C., 1961, 105 pp.

3. U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Energy Dissipaters for Culverts and Channels, Hydraulic Engineering Circular #14, Washington, D.C., 1983.

LIST OF ATTACHMENTS

FDM 13-15-15 Special Hydraulics

August 8, 1997

15.1 Introduction

For the purpose of this manual, special hydraulics is defined as hydraulic structures that are considered unique by highway engineers because of their limited use in highway engineering. This procedure contains general discussions on drainage disposal by pumping, and siphons and sag culverts.

15.2 Drainage Disposal by Pumping

15.2.1 General Practices

1. Drainage disposal by pumping should be avoided where gravity drainage is reasonable. Because pumping installations have high initial cost, maintenance expense, power costs, and the possibility of power failure during a storm, large expenditures can be justified for gravity drainage. In some cases this can be accomplished with long runs of pipe or continuing the depressed grade to a natural low area.
2. Horizontal pumps in a dry location should be specified for ease of access, safety, and standardization of replacement parts.
3. Whenever possible, drainage originating outside the depressed area shall be excluded.
4. Stand-by power installations for pumping plants shall be made only in special cases.

15.2.2 Surface Inlets

Grate and combination inlets should be used for surface drainage. If conditions dictate the use of a side-opening inlet, a trash rack should be provided.

15.2.3 Maintenance Access

Access to the pumping plant for maintenance from the lower roadway should generally consist of a stairway or paved ramp adjacent to the pumping plant. A stairway or ramp should generally extend from the top of cut slope to the toe of cut slope. Parking space for maintenance vehicles shall be provided in the vicinity of the pumping plant. Access to the pump room should be through a vertical doorway with the bottom at or near floor level, and never through a hatch.

15.3 Siphons and Sag Culverts

15.3.1 General Notes

There are two kinds of conduits called siphons: the true siphon and the inverted siphon or sag culvert. The true siphon is a closed conduit, a portion of which lies above the hydraulic grade line. This results in less than atmospheric pressure in that portion. The sag culvert lies entirely below the hydraulic grade line; it operates under pressure without siphonic action.

Under the proper conditions, there are hydraulic advantages and cost economies to be obtained by using the siphon principle in culvert design.

15.3.2 Sag Culverts

This type is most often used to carry an irrigation canal under a highway when the available headroom is insufficient for a normal culvert. The top of a sag culvert should be at least 4.5 feet below the finished grade where possible to ensure against damage from heavy construction equipment. The culvert should be on a straight grade and sumps provided at each end to facilitate maintenance. Sag culverts should not be used:

1. When any other alternative is possible at reasonable cost.
2. For intermittent flows where the effects of standing water are objectionable.
3. When the flow carries trash and detritus in sufficient quantity to cause heavy deposits.

15.4 Type of Conduit

Siphons should have water-tight joints. Gaskets are required to be water-tight at the pipe joints. The following are kinds of pipes used for siphons and sag culverts to prevent leakage:

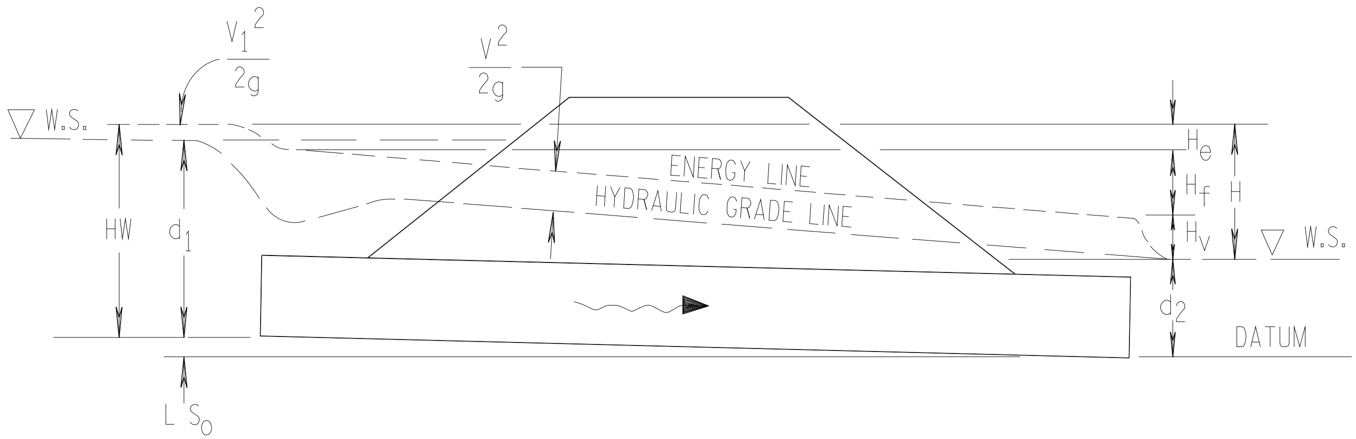
1. Reinforced concrete pipe.
2. Ductile iron pipe.
3. Welded smooth steel pipe.

Siphons that are subjected to internal pressure should have watertight joints. Welded smooth steel pipe with internal ceramic coating as well as precast reinforced concrete pressure pipe, ductile iron pipe, or reinforced plastic mortar pressure pipe are commonly used. Jointed pipe require gaskets to ensure water tightness. Inverted siphons must be able to withstand the internal hydrostatic head measured to the centerline of the siphon.

Methods for designing siphons can be found in the following recommended hydraulics design book, such as "Handbook of Hydraulics," Horace Williams King; "Design and Construction of Sanitary and Storm Sewers," ASCE and WPCF; "Handbook of Concrete Culvert Pipe Hydraulics," Portland Cement Association; and "Roadway Drainage Manual," AASHTO.

Type of Structure and Design Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7 or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edge at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edge at crown	0.7
Side-or slope-tapered inlet	0.2

SCHEMATIC DEPICTION OF THE ENERGY LOSSES THROUGH A CONDUIT



Where;

H_e = entrance loss

H_f = friction loss

H_v = velocity head

INLET CONTROL PROBLEM

PROJECT: ____________ **DESIGNER:** ____________

DATE: ____________

HYDROLOGIC AND CHANNEL INFORMATION

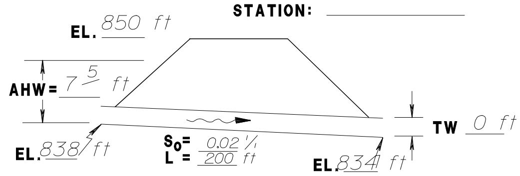
Q_1 = 190 cfs TW_1 = 0 ft

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ____________



MEAN STREAM VELOCITY = ______
MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	HW	OUTLET CONTROL $HW = H + h_0 - LS_0$: K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_0	LS_0	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
Prog. C.M.P.	190	66	1.25	6.9										11		n = .024
Groove Conc	190	60	1.3	6.5										19		n = .013

SUMMARY & RECOMMENDATIONS:

OUTLET CONTROL PROBLEM

PROJECT: ____________ **DESIGNER:** ____________

DATE: ____________

HYDROLOGIC AND CHANNEL INFORMATION

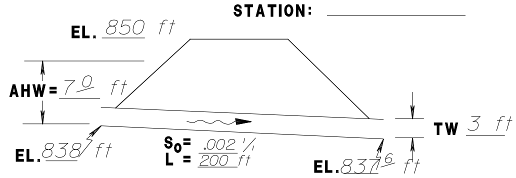
Q_1 = 190 cfs TW_1 = 3 ft

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ____________

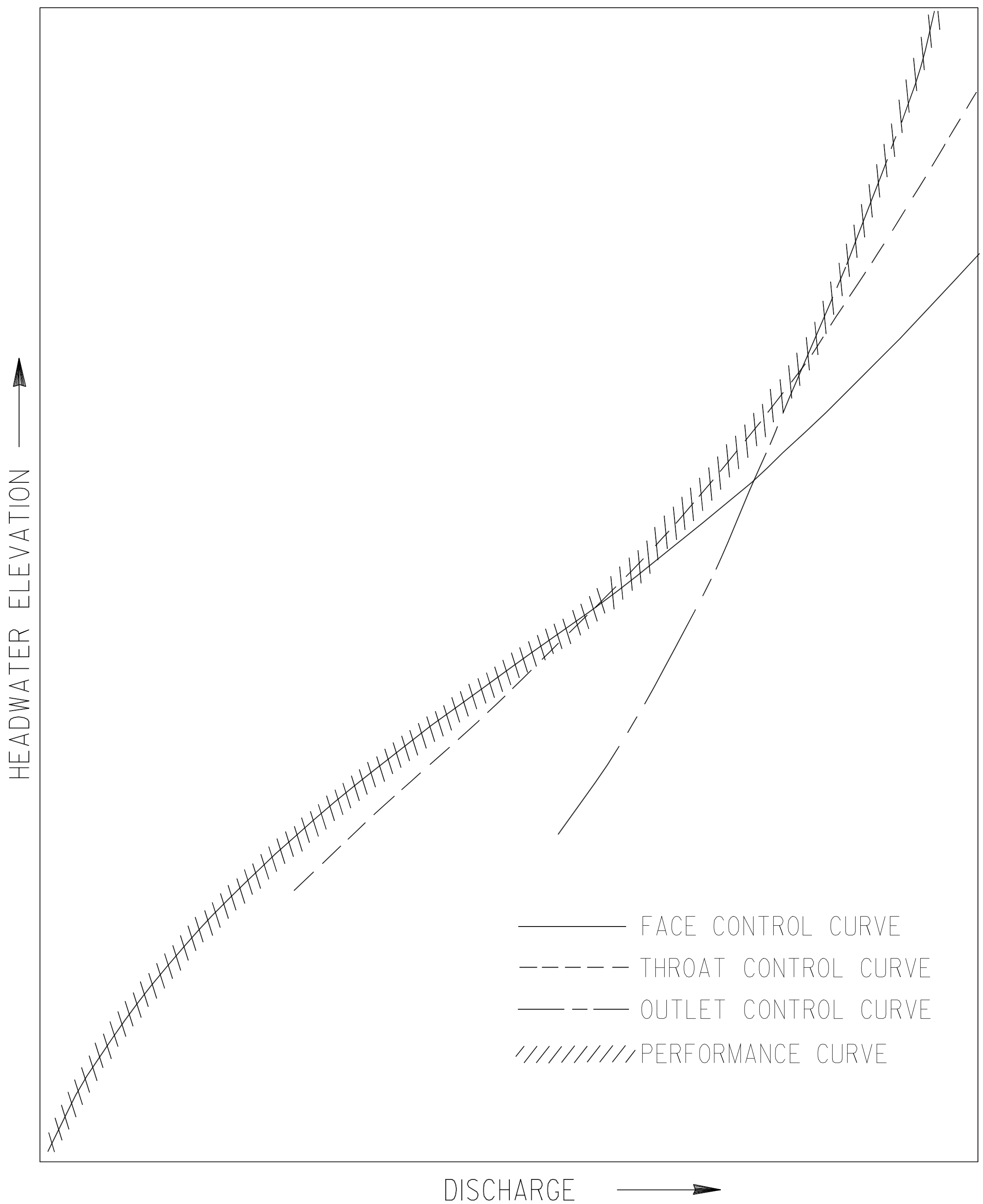


MEAN STREAM VELOCITY = ______
MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	HW	OUTLET CONTROL $HW = H + h_0 - LS_0$: K_e	H	d_c	$\frac{d_c + D}{2}$	TW	h_0	LS_0	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
Prog. C.M.P.	190	84			.9	1.4	3.6	5.3	3	5.3	.4	6.3		9.7		
Groove Conc	190	72			.2	1.25	3.9	4.95	3	4.95	.4	5.8		9.7		
" "	190	60			.2	2.8	3.9	4.45	3	4.45	.4	6.9		11.5		

SUMMARY & RECOMMENDATIONS:

h_0 = The greater of $\frac{d_c + D}{2}$ or TW $H = [1 + K_e + (29 n^2 L)/R^{1.33}] V^2/2g$



SCHEMATIC PERFORMANCE CURVE

Source: HDS No. 5, "Hydraulic Design of Highway Culverts", FHWA Sept., 1985.

FDM 13-20-1 Design Methods

June 19, 2013

1.1 Definition

A bridge is defined as a structure having a span of more than 20 feet from face to face of abutments or end bents, measured along the center line of the roadway. This definition also applies to box culverts (measured from inside face of outer cells) and multiple pipes (measured from outer face of outer pipes provided the clear distance between adjacent pipes is less than half the diameter of the smaller contiguous pipe).

The region is required to submit a Structure Survey Report and a Hydraulic Report for each bridge structure. For more information regarding Structure Survey Reports, refer to FDM 3-20-30. Reporting procedures are provided in Chapter 6 and Chapter 8, respectively, of the WisDOT Bridge Manual.

1.2 Type of Flow

The three types of flow that may be encountered in bridge design (see Attachment 1.1, Detail B) are labeled as Type **I** flow (subcritical), Type **II** flow (passes through critical), and Type **III** flow (critical). The symbols in this attachment are defined as follows:

h_I*=total backwater or rise above normal stage at Section 1

N.W.S.=normal water surface

S_O=slope of channel bottom or normal water surface

y_x=depth of flow at Section X

y_n=normal depth of flow in model

y_{xc}=critical depth at Section X

W.S.=water surface

Most of the streams in Wisconsin exhibit flat gradients and hence Type **I**, or subcritical flow, is normally encountered.

1.3 Methods

Normally, determining the required waterway areas for minor drainage structures is performed by the region staff through the hydrological/hydraulic analysis of the site with the aid of region-collected data. The Bureau of Structures (BOS) is responsible for the hydraulic/hydrologic analysis of bridges and box culverts they design while consultants perform this function for structures they design.

Due to economics, most bridges are not designed to span the entire floodway that occurs at a specified flood flow. Instead, only a part of the floodway is spanned, thus producing a constriction. The loss of energy produced by this constriction must be balanced by a rise in the upstream water surface. This rise, denoted by h_1* in Attachment 1.1, is called the backwater. The backwater of a bridge is defined as the upstream water surface rise above normal stage of the natural stream. The designer should determine the impact of the backwater on the floodplain as described in FDM 13-10-1.

Bridge design involves determining the waterway area, location, and configuration that will produce a backwater equal to or less than some specified value. This goal may be achieved by a number of methods. Three of the most common ways to accomplish the calculation of bridge and culvert hydraulics are:

1. WSPRO, Water-Surface Profiles, (HY-7)
2. Hydraulic Design of Highway Culverts, (HDS-5 & HY-8)
3. Water Surface Profiles, (HEC-2) & (HEC-RAS)

1.3.1 WSPRO, Water-Surface Profiles (HY-7)

FHWA and WisDOT endorse and employ this design methodology in the majority of its stream crossing bridges. WSPRO is a computer model developed by the U.S. Geologic Survey for the computation of water surface profiles using a one-dimensional step-backwater approach. Profile computations for free-surface flow through bridges are based on relatively recent developments in bridge backwater analysis and recognize the influence of bridge geometry variations. The model has the ability to compute subcritical as well as supercritical profiles. Pressure flow situations are computed using FHWA techniques. Embankment overtopping flows, in conjunction with either free-surface or pressure flow through the bridge, can be computed. WSPRO is also capable of computing profiles at stream crossings with multiple openings including culverts.

Updates to the WSPRO program will include metric input and output as well as scour analysis routines that are based on the FHWA Hydraulic Engineering Circular No. 18 "Evaluating Scour at Bridges." References for user include:

1. J.O. Sherman, W. H. Hirby, V.R. Schneider, H. N. Flippo, "Bridge Waterways Analysis Model: Research Report," Federal Highway Administration Report No. FHWA/RD-86/108.
2. J.O. Sherman, "Users Manual for WSPRO, A Computer Model for Water-Surface Profile Computations," Federal Highway Administration Report No. FHWA-SA-98-080.
3. "Hydraulics," Bridge Manual, State of Wisconsin, Department of Transportation, Chapter 8.

1.3.2 Hydraulic Design of Highway Culverts, (HDS-5 & HY-8)

HY-8 is a comprehensive culvert design program that includes analysis and design capabilities for conventional culvert, culverts with improved inlets, energy dissipators, multiple culvert analysis, storage-routing techniques, and hydrologic analysis. The program offers the user a wide range of alternate drainage structure shapes for analysis that include circular, box, elliptical, pipe-arch, and user defined. The user has a choice of materials that include steel, concrete, and aluminum. The program also offers a variety of options to define the tail-water elevation.

The HY-8 methodology is based on the application of three Federal Highway Administration publications: "Hydraulic Design of Highway Culverts (HDS No.5)" dated September 2001, "Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC No. 14)," and "Hydrology (HEC No. 19)."

References for users include:

1. Ginsberg, Abigail, "HY-8 Culvert Analysis Microcomputer Program Applications Guide", Federal Highway Administration Report FHWA-EPD-87-101.
2. "Hydraulic Design of Highway Culverts (HDS No. 5)," Federal Highway Administration Report No. FHWA-NHI-01-020.
3. "Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC No. 14),"Federal Highway Administration Report No. FHWA-NHI-06-086.
4. "Hydrology" (HEC No. 19), Federal Highway Administration Report No. FHWA-IP-84-15.

1.3.3 Water Surface Profiles, HEC-2 and HEC-RAS

HEC-2 and the update HEC-RAS are step-backwater programs similar to the WSPRO program. The U.S. Army Corps of Engineers developed and maintains this model. The program is intended for calculating water surface profiles for steady gradually varied flow in natural or man-made channels. Both subcritical and flow profiles can be calculated. The effects of various obstructions such as bridges, culverts, weirs, and structures in the floodplain may be considered in the computation. The computational procedure is based on the solution of the one-dimensional energy equation with energy losses due to friction evaluated with Mannings Equation. This computational procedure is generally known as the standard step method.

This program is most frequently and historically used in the development of flood profiles and floodway delineation for use in flood insurance studies as part of the National Flood Insurance Program.

References for users include:

1. "HEC-2 Water Surface Profiles Users Manual," Computer Program 723-X6-L202A, Hydraulic Engineering Center, Davis, CA. May 1991.

1.4 Additional Literature

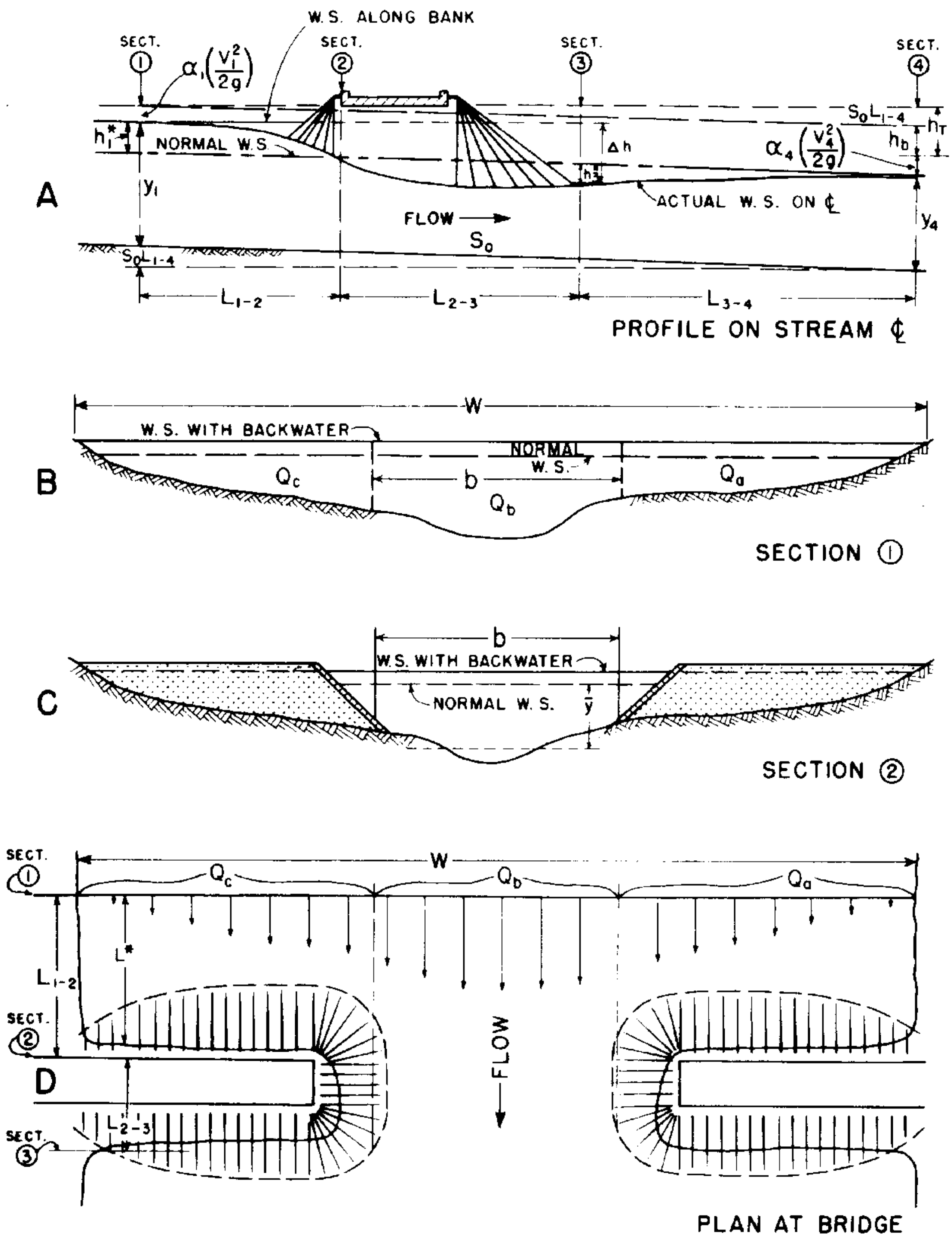
For designs involving local scour, overflow sections, and spur dikes, the designer is referred to the following:

1. HDS #1, "Hydraulics of Bridge Waterways," U.S. Department of Transportation, FHWA, 1978.
2. "Hydraulics," Bridge Design Manual, State of Wisconsin, Department of Transportation, Chapter 8.

LIST OF ATTACHMENTS

Attachment 1.1	Types of Flow Encountered at Bridges

DETAIL A



Normal Crossings: Spillthrough Abutments

Source: HDS No. 1, Hydraulics of Bridge Waterways, FHWA, 1973.

FDM 13-25-1 Introduction *August 8, 1997*

1.1 Introduction

For the design of an underground storm drain system, it is necessary to have complete and accurate information regarding the existing system. If the existing system is to be used it should be carefully checked hydraulically and physically to show it is adequate to accommodate the additional water from the highway improvement. If possible, a check of the energy losses associated with the system should be performed (refer to Pressure Flow, FDM 13-25-35).

On new projects where the existing sewer is a combination storm and sanitary system, every effort should be made to remedy the situation by separating the sewers.

Storm sewers shall not alter the existing drainage pattern, and the sewer outfalls shall be located at existing drainage ditches and/or natural low points.

The latter steps in the design of the storm sewer may require the designer to go back and change initial assumptions.

If catch basins are installed on a project, a maintenance schedule should be developed (refer to FDM Chapter 10).

Attachment 1.1 shows a flowchart that describes the various steps of a storm sewer design.

LIST OF ATTACHMENTS

Attachment 1.1	Storm Sewer Design Flow Chart

FDM 13-25-5 Basic Drainage Area Information *August 8, 1997*

5.1 Basic Information Needs

Before the design engineer commences designing a storm sewer system, the following basic information should be collected on the specific area to be drained by the proposed system:

1. Aerial photographs showing the existing land use patterns.
2. A contour map of the area where the storm sewer is to be built. Use a scale of 1" = 100' (1:1200) or 1"= 50' (1:600) with a two foot contour interval.
3. A U.S.G.S. map (scale 1" = 2,000' or 1" = 5,208'), (1:24000 or 1:62500) of the drainage basin containing the area being studied.
4. Soil maps for aid in estimating the runoff potential.
5. Water table data from available borings or maps.
6. A layout of the area where the storm sewer is to be built. This should show the existing or proposed streets, intersections, and development type.
7. Plans of any existing drainage system.
8. Typical street cross section.
9. Street and intersection grades of the area under study.
10. Information on existing and proposed underground utilities.
11. Location, ground elevation, and high-water records for the outfall point of the proposed storm sewer system and/or the existing storm sewer system.
12. Rainfall curves appropriate to the drainage area. Refer to FDM 13-10-5.
13. Local design standards, land use information, and future drainage plans obtained from the appropriate local governmental unit.

14. Field inspection data from the area under study.
15. Locations of sensitive areas and any potential source of pollutants. Refer to Chapter 10.

FDM 13-25-10 Field Drainage Information *August 8, 1997*

10.1 Field Information Needs

Field information must be collected for the drainage area to be served by a storm sewer in order to update the existing land use patterns and to verify the direction of overland flow in the vicinity of the highway. In this field review, the following steps should be taken:

1. Field-walk the drainage area, taking note of any natural waterways, ditches, sinkholes, dry wells, ponds, tiles, or anything else affecting drainage not previously recorded.
2. Record the location of any existing and/or possible outfalls. This will facilitate the mapping of possible drainage easements.
3. Record the location and depth of underground utilities not shown on any maps (water, gas, electric, telephone, sanitary sewers, storm sewers, etc.).
4. Verify the office-estimated runoff coefficients through field-checking the topography of the drainage area.
5. Check with local residents, local maintenance foremen, and municipal officials, etc., for any possible problem areas, such as flooding of lowlands.
6. Identify and locate sensitive areas and any potential source of pollutants. (refer to Chapter 10).

FDM 13-25-15 Preliminary Layout of System *November 26, 1997*

15.1 Background Information

The preliminary layout of a storm sewer system is divisible into two major operations as follows:

1. Locate and space inlets.
2. Prepare a plan layout of the storm sewer system showing the following data:
    - Location of all underground utilities in the plan and profile. Also plot these utilities on cross sections.
    - Location of the main storm sewer line.
    - Direction of flow.
    - Location of inlets.
    - Location of manholes.
    - Location of outfall(s).
    - Shape and type of conduit.

The preliminary plan can be constructed through the application of the design criteria that follow.

15.2 Inlet Locations

Maximum Spacing: Water should normally not travel more than 300 to 600 feet before interception, with the closer spacing employed for flat terrain and for high-speed highways. Moreover, spacing of inlets should be designed to prevent water from spreading over more than one-half of the traveled lane and from overtopping the curb. Since a parking lane or shoulder is not considered a traveled lane, the flow of water can utilize the full parking lane or shoulder width. However, the future expansion of the roadway should be considered before the full parking lane is used for conveyance of water.

At Low Points: In a curb section, at least one inlet must be located at the low point of each sag vertical. However, if there is a possibility of clogging because of high quantities of debris, two inlets should be installed - one at the low point and one where the grade elevation is about 0.20 foot higher than at the low point. Hydraulic Engineering Circular No. 12 “Drainage of Highway Pavements” has a discussion about the use of flanking inlets.

At Bridge Ends: Generally, inlets should be placed to intercept the gutter flow before it reaches the bridge.

At Intersections: Inlets at intersections are to be placed in order to intercept the gutter flow before it reaches a pedestrian crosswalk.

Prevention of Cross Pavement Flow: The flowing of water across pavements should be prevented in order to preclude icing in the winter and hydroplaning during the warmer months. In particular, where pavements are super-elevated, inlets shall be placed to intercept the gutter flow before the pavement becomes too flat for effective pickup.

Driveway Openings: Where driveways have a descending grade from the gutter, the installation of inlets might be necessary to preclude the design storm from overflowing at the driveway openings. However, to minimize the number of inlets, the driveway cross section should be designed with a gutter sufficiently deep to accommodate the design flow.

Side Drainage: Drainage from outlying areas should be intercepted before it reaches the roadway pavement, especially where mud and debris will be carried onto the pavement.

15.3 Conduit Location

The location or lateral placement of a conduit system is dictated by economics, hydraulic requirements, ease of construction and maintenance, and local community preference.

Proposed storm sewer shall be laid at least 8 feet horizontally from any existing or proposed water mains. The distance shall be measured from center to center. In cases where it is not practical to maintain an 8 foot separation, Chapter NR 811 of the Wisconsin Administrative Code shall be consulted for additional guidelines.

Curvilinear and angular alignments in conduits produce hydraulic losses. Therefore, if possible, any change in alignment between structures is to be avoided, especially on trunk or main line segments of a storm sewer system. For pipes with diameters of 30 inches or more, a long radius curve of 100 feet or more is permissible. The radius of curvature specified shall be one of the available standard manufactured curves in the specified type of material.

On lateral lines, an angular change in alignment for conduit of 30 inches or less in diameter is permitted.

15.4 Standards for Storm Drain Pipe

Pipe Diameter: The minimum pipe diameter shall be as follows:

Type of Drain	Minimum Diameter (Inches)
Trunk Line	[1]24"
Trunk Laterals	15"
Inlet Laterals	12"

[1] For short runs, smaller sizes may be specified.

Under special conditions, such as a problem with fine debris or flat grades, the minimum pipe size for laterals should be 18 inches

Pipe Strength: Strength requirements for pipe shall conform to the ASTM designations for the type and class of pipe as given in the approved practice drawing.

Pipe Slope: Refer to FDM 13-25-35, for minimum pipe slopes.

15.5 Manholes

Purpose: The principal purpose of a manhole is to provide maintenance access to a continuous underground conduit.

Types: Refer to Chapter 16 of this manual for standard detail drawings of approved manholes. Special manholes shall be designed when conditions make the use of the above-listed manholes not feasible. When a manhole is used as an inlet, the design criteria for inlets apply.

Location: In general, manholes are to be located as follows:

1. At the end of existing and future lines
2. Where the conduit changes size
3. At sharp curves or angles in the line (10° or over)
4. At points where there is an abrupt change in grade
5. At all intersections

6. At junctions of sewers

If possible, avoid locating manholes in traffic lanes. When manholes must be placed in traffic lanes, care shall be taken to avoid the normal wheel tracks.

Spacing: To facilitate maintenance operations, manhole spacing should be as follows:

Size of Pipe in Inches	Maximum Distance in Feet
12 through 24	350
27 through 36	400
42 through 54	500
60 or larger	1,000

In cases where a municipality has its own policy on spacing of manholes, requiring a lesser maximum spacing, consideration may be given to that policy.

15.6 Outfalls

To preclude an expensive design, care should be taken to avoid placing an outfall underwater or where water might back up into the system. In general, sewer outfalls shall be placed at existing drainage ditches and natural low points.

FDM 13-25-20 Design Discharge

August 8, 1997

20.1 Design Discharge Information

The design discharge used in sizing storm sewer systems should be determined by the Rational Formula. Refer to FDM 13-10-5 for a thorough explanation of the theory and application of the Rational Formula.

When side drainage is collected in the system, the peak discharge must be estimated for the following two separate conditions:

1. Fast runoff from short-duration, high-intensity rainfalls. These peak flows originate only from the connected impervious areas adjacent to the storm sewer, e.g., streets, sidewalks, parking lots, etc.

2. Slow runoff from long-duration, low-intensity rainfalls. These peak flows originate from the total area draining toward the storm sewer.

Since storm sewers are sized for the largest peak flow produced by a specific design frequency, the controlling condition is the one that produces the largest peak flow.

For urban streets, the rainfall intensity should be determined on a 10-year (check 25-year) frequency with a rainfall duration equal to the minimum time of concentration or five minutes, whichever is greater. If the check using the 25-year storm results in unacceptable conditions (highway inundation, flooding, etc.), the sewer shall be sized accordingly to alleviate the effects of backwater associated with the proposed sewer system (refer to FDM 13-25-45 for a design using surcharged full flow).

In cases where the municipality has its own criteria for the design of a sewer system, consideration may be given to those criteria.

Intensity for interstate and freeway projects shall be determined for a minimum time of concentration of five minutes on a 25-year frequency. At sag points, such as roadway underpasses a check of the Hydraulic Gradeline (HGL) should be made using the 50-year storm.

A weighted or composite runoff coefficient shall be determined as explained in FDM 13-10-5.

FDM 13-25-25 Gutter Design

August 8, 1997

25.1 Capacity

The hydraulic design of gutters consists of determining the spacing of inlets to avoid the undue spreading of water over the pavement, or determining the height of water at the face of curb if the spacing is predetermined. The gutter capacity is determined using Attachment 25.1, which applies to triangular channels and other shapes shown in the figure.

25.2 Gutter Types

Refer to FDM 16-5-1 for standard detail drawings of approved curb and gutter cross sections.

25.3 Longitudinal Slopes

Preferable minimum longitudinal gutter grades shall be 0.5 percent, with an absolute minimum of 0.3 percent, except in short runs that will carry no appreciable flow.

Example Problem

Given:

1. Type A Curb and Gutter, 30-Inch.
2. Longitudinal Pavement Slope = 1.0%.
3. Crown of Pavement = 2.0%.
4. Street Width Face-to-Face = 28 feet.

Find:

1. The maximum allowable flow Q_p for the above type of street section.

Solution:

See Attachment 25.2 for a sketch of the street cross section. The process involves the following steps.

1. Determine the maximum allowable flow in combined areas "b" and "c," $Q_{(b+c)}$.

   1.1 From FDM 13-25-15, up to one-half of the traveled lane width may be used to convey storm water flow as long as the curb is not overtopped. In this case half the traveled lane width is 6 feet.

   1.2 The maximum depth, d', in area (b+c) is d' = 6 feet x 0.02 or 0.12 feet.

   1.3 Use Attachment 25.1 and the data below to calculate $Q_{(b+c)}$.

- S= 0.01 ft/ft (given)
- $Z_{(b+c)}$ = 1/.02 = 50 (defined in Attachment 25.1)
- n= .015 (from Attachment 35.1)
- $Z_{(b+c)}$ /n = 3333
- From the nomograph in Attachment 25.1: $Q_{(b+c)}$ = 0.65 cfs

2. Determine the maximum allowable flow in combined areas "a" and "c," $Q_{(a+c)}$.

- From SDD 8d1, the cross slope of Type A curb & gutter is 3/4 inch per ft or 0.0625 ft/ft and the gutter width is 2 ft.
- The depth, d, at the curb is d' + (2 ft x 0.0625 ft/ft) or 0.245 ft. Note the curb is not overtopped so the flow is allowed to extend to half the width of the traveled lane.
- Other data:

  The values for "S" and "n" remain the same as in Step 1 above.

  $Z_{(a+c)}$ = 1/.0625 = 16

  $Z_{(a+c)}$ /n = 1067

  Again, using the nomograph and solving for discharge, $Q_{(a+c)}$ = 1.5 cfs

3. Determine the maximum allowable flow for area "c," Q_c.

- The values for "S," "n" and cross slope remain the same as in Step 2 above so the value for $Z_{c/n}$ is the same as $Z_{(a+c)}$/n above.
- The maximum depth is d' or 0.12 ft.
- From the nomograph, Q_c = 0.25 cfs

4. Determine total maximum allowable flow.

   $Q_p = Q_{(b+c)} + Q_{(a+c)} - Q_c$

   Q_p = 0.65 + 1.5 - 0.25 = 1.90 cfs

In this situation, an inlet must be provided before the volume of flow reaches 1.90 cfs in order to prevent the storm water from infringing too far into the traveled lane.

LIST OF ATTACHMENTS

Attachment 25.1	Gutter Design Nomograph
Attachment 25.2	Gutter Design Example

FDM 13-25-30 Hydraulic Design of Inlets *August 8, 1997*

30.1 Inlet Types

A storm water inlet is a means of admitting storm water into a storm sewer system. Inlets are either constructed on a continuous grade or in a sump condition, and the gutter is either depressed or not depressed. The term "continuous grade" means that the grade of the street is continuous past the inlet. The sump condition exists whenever the water is restricted to the inlet because the inlet is in a low point. The five general types of inlets along with some general information on their use and several examples are listed below. For more thorough discussion, it is recommended that designers obtain a copy of FHWA's Hydraulic Engineering Circular (HEC) #12, Drainage of Highway Pavements. A copy can be obtained for a fee by contacting National Technical Information Services at 1-800-553-6847 and asking for publication FHWA-TS-84-202. Manufacturers, such as Neenah Foundry, should also be contacted when special designs are being considered.

30.1.1 Curb Opening Inlets

A curb opening inlet consists of a vertical opening in the curb through which the gutter flow passes. The major advantage of the curb opening inlet is that it does not clog easily through trapping of debris; instead, the debris readily passes through the curb opening into the storm sewer system.

The capacity of this inlet is increased significantly by depressing the gutter. They perform well where orifice flow may occur, such as in sag or flat conditions. On continuous grades they do not perform well because gutter flow typically bypasses the opening.

30.1.2 Grated Inlets

Basically, a grated inlet consists of an opening in the gutter covered with a grate. The bars may be oriented either longitudinally or transverse to the flow. Although longitudinal grates are more efficient than transverse grates, longitudinal grates are not permitted where bicycle traffic is permitted. However, the efficiency of transverse grates has been improved by designing the bars as vanes slanted at 55 F from the horizontal plane (refer to Chapter 16 for standard detail drawings of these special grates). These grates must be oriented with the slant in the direction of the oncoming flow. Improperly installing the grate will result in virtually 100 percent overflow.

The major disadvantage of grated inlets is that they tend to plug, especially in a sump condition.

Depressing of gutter will significantly increase its capacity; because there is no curb opening to handle possible clogging of the grate, this depression may be undesirable from a traffic standpoint.

30.1.3 Combination Inlets

A combination inlet is composed of both a curb opening inlet and a grated inlet, with the grated inlet usually placed directly in front of the curb opening. In sump conditions, grated inlets in combination with curb openings are advisable since the grate is more apt to plug.

Curb openings on a continuous grade do not efficiently trap water and may not be the best alternative in areas where water quality is a concern. Using curb openings with grates on a continuous grade may be useful in situations where debris is a problem. If debris clogs the grate the curb opening will still handle a minimal amount of the gutter flow.

One alternative to using curb openings is to evaluate grates that have been designed for debris handling efficiency. This information can be best obtained by contacting inlet manufacturers.

Capacity of a combination inlet on grade with a curb opening can be improved if a "sweeper inlet" is used. A sweeper inlet is a combination inlet that has the curb opening upstream of the grate so as to intercept debris and capture some of the gutter flow prior to it entering the grate. Designers should follow H.E.C. #12 for guidance on calculating the inlet capacity of sweeper inlet.

Theoretically, in a sump condition the combination inlet exhibits a high flow capacity; however, this is questionable because of the grate plugging, thus leaving only the curb opening to handle the gutter flow.

In sump conditions, combination inlets are considered advisable where ponding can occur. When in weir flow, the interception capacity of the combination inlet is essentially equal to that of a grate inlet alone, unless the grate becomes clogged. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening (see H.E.C. #12 for examples). Flanking inlets are recommended where significant ponding can occur, such as underpasses and sag vertical curves in depressed conditions.

30.1.4 Multiple Inlets

A multiple inlet consists of two or more closely spaced inlets acting as a unit. The inlets may be any combination of the above-explained types. The characteristics of a specific multiple inlet are the same as the characteristics of each individual inlet type used to construct the multiple inlet. Care should be taken to avoid placement in traffic lanes and interference with bicycle traffic.

Grate inlets are often placed next to each other for this purpose. They perform best when in parallel and adjacent to each other as opposed to adjacent in series along the gutter. However, should placement in series be preferred, inlets should be spaced apart from each other at a distance that allows the bypass flow to return the curb face. If this is not done, the second grate will not provide the capacity it was designed for as most of the water will bypass it.

When inlets are placed in a series, the recommended minimum spacing (a function of the discharge, longitudinal slope, and transverse slope) should be from six to 20 feet to allow the bypass flow to return to the curb face.

30.1.5 Slotted CMP Surface Drains

This type of inlet consists of a corrugated metal pipe with a continuous slot on top. The slot is formed by a pair of angle irons, which serves as a paving bulkhead. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. See discussion and examples presented in this procedure and HEC #12.

30.2 Allowable Inlet Capacities

The allowable inlet capacity is computed as a percentage of the theoretically calculated capacity. This compensates for the decreased capacity of the inlet through debris clogging, pavement overlapping, varying design assumptions, etc. The allowable design capacity for an inlet is determined by applying the reduction factors from Attachment 30.1 to the theoretical capacity calculated by the design process described in this procedure.

Though application of these reduction factors may be needed in a sump or relatively flat condition to account for potential clogging, they may not be needed for inlets on continuous grade. Partial clogging rarely causes major problems for inlets on grade. Therefore, reduction factors should be applied to inlets on grade only "when local experience indicates an allowance is advisable." To help determine whether a reduction factor is needed, designers should do a site investigation and inquire with maintenance staff, and those familiar with the area, to evaluate whether clogging is a problem.

30.3 Capacities of Grate Inlets and Combination Inlets on a Continuous Grade

The capacity of both grate inlets and combination inlets depends on both the length and the depression of the inlet, the depth of flow in the gutter at the curb line, and both the cross slope and the longitudinal slope of the gutter.

The general equation used to determine the capacity of a grate inlet is:

$$Q = KD^{5/3}$$

Where:

Q = grate inlet capacity in cfs.

K = an empirical coefficient for a specific grate, with the appropriate design longitudinal and transverse slopes.

D = curb line flow depth (in feet) upstream from the grate

The Neenah Foundry Company has developed for each grate that they manufacture a chart that gives values of K versus transverse slope S_T (0 to 0.06 ft./ft.)

For longitudinal slopes S_L of one, two, four, and six percent. Charts for most WisDOT-approved grate types may be obtained from a Neenah Foundry publication entitled "Neenah Inlet Grate Capacities" (1).

For unusual situations, such as a depressed inlet, etc., the designer is referred to the research work of John

Hopkins University (2).

Example Problem

Capacity of a Combination Inlet Grate on a Continuous Grade

Given: Type A Curb and Gutter, 30-Inch
Longitudinal Slope SL = 1.0%
Crown of Pavement = 2.0%
Transverse Slope of Gutter ST = 0.0625 ft./ft.
Street Width Face-to-Face = 28 feet
Inlet Type H

Find: The allowable inlet capacity for the above design criteria when the gutter is flowing at the allowable capacity.

Solution:

1. Design equation is $Q = KD^{5/3}$.
2. From the example problem of FDM 13-25-25:

   Allowable D = .245 foot
3. From Neenah Inlet Grate Capacities Manual(1) with S_T = 0.0625 ft./ft., S_L =1%:

   K = 12.5 for a Type H inlet (extrapolated)
4. Therefore, without clogging, the Type H inlet grate capacity is:

   $Q = KD^{5/3}$

   $Q = 12.5\ (0.245)^{5/3}$

   Q = 1.20 cfs
5. From Attachment 30.1, the reduction factor R.F. for a combination grate on a continuous grade is:

   R.F. = 1.10 x .50 = .55
6. Therefore, the allowable inlet capacity is:

   Q(all.)= .55 x 1.20 cfs = .66 cfs

30.4 Capacity of Grate Inlets in a Sag

Depending upon the depth of flow, grate inlets operate under three different conditions of flow:

1. Weir flow (flow depth less than 0.4 foot)
2. Transitional flow, undefinable flow because of vortices and other disturbances (flow depth from 0.4 and 1.4 feet); or
3. Orifice flow (flow depth greater than 1.4 feet).

For transitional flow, the grate inlet capacity is somewhere between the inflows predicted by the weir and orifice flow equations.

The capacity of grate inlets in a sag condition may be determined with the aid of the FHWA H.E.C. #12 publication entitled "Drainage of Highway Pavements" (3).

Specifically, Chart 11 of H.E.C. #12 may be used to graphically solve for a weir or orifice flow condition.

In order to use this chart, the designer must know the effective perimeter and/or inflow area of the selected grate. The effective perimeter may be easily computed from the grate dimensions, while the inflow area may be obtained from a Neenah Foundry publication entitled "Neenah Inlet Grate Capacities" (1).

In addition, the Neenah Foundry publication also contains a nomograph for easy solution of only the orifice flow condition.

30.5 Capacity of Curb Openings in a Sag

As with grate inlets, curb openings operate under three different conditions of flow:

1. Weir flow depth less than the height (h) of the curb opening;
2. transitional flow, undefinable flow because of vortices and other disturbances (flow depth from h and

1.4 h); or

3. orifice flow (flow depth greater than 1.4 h).

Although not empirically correct, for the sake of a design solution it is suggested that designs in the transitional zone be accomplished with the orifice equation. The FHWA publication H.E.C. #12 (3,) contains a thorough discussion along with an example problem for determining the capacity of curb openings in a sag.

30.6 Spacing of Inlets on a Continuous Grade

The spacing of inlets is determined by:

1. The design discharge,
2. the carrying capacity of the gutter, and
3. the allowable spread of water on the pavement. Moreover, the spacing of inlets by hydrologic and hydraulic computations requires a trial and error solution for streets that have varying grades, widths, and cross slopes.

The peak flow rate is determined by the Rational Equation, which is fully explained in FDM 13-10-5. Furthermore, the peak flow rate may result from participation of the entire drainage area or only the pavement surface with all the directly connecting impervious areas. Since the time of concentration in most design cases will be less than five minutes, the rainfall duration will be the standard five-minute minimum.

The economical design of inlets requires that a certain amount of bypass flow be allowed to pass to the next inlet. In certain situations, such as pedestrian crossings, the bypass flow may have to be captured by a second inlet, which should be located a minimum of six to 20 feet downstream from the first inlet. In most cases, standard highway inlets have enough capacity to catch all gutter flow within the width of the inlet.

Procedure

With only pavement runoff, the general procedure for the spacing of inlets on a continuous grade is as follows:

1. Locate the high points on the profile.
2. Estimate the average gutter grade from the high point or the last inlet location to the approximate location of the next inlet.
3. Determine the allowable gutter flow Q_p using FDM 13-25-25.
4. Select a trial inlet and determine the allowable inlet capacity Q_i using the design methodology explained in this procedure under the heading "Capacity of Grate Inlets and Combination Inlets on a Continuous Grade."
5. Determine the five-minute duration, 10-year frequency rainfall intensity "I" from the appropriate intensity-duration-frequency curve, FDM 13-10-5, Attachment 5.4.
6. Determine the pavement-gutter width "W" contributing runoff to the subject inlet.
7. a. First inlet only, the design discharge Q_D equals the allowable gutter flow: $Q_D = Q_p$. Using this design flow in step 8 will give the length of street necessary to produce a gutter flow equal to the allowable gutter flow.

   b. Subsequent inlets, the design discharge equals the allowable gutter flow minus the bypass flow QB from the last inlet or the allowable capacity of the present inlet Qi, whichever flow is less: $Q_D= Q_p – Q_B$ or $Q_D- Q_i$.
8. Estimate the inlet spacing L in feet by substituting the above values into the equation L = 48500 Q_D/IW, with Q_D in cfs, I in./hr., and W in feet.
9. For vertical curved sections, check the assumed grade from step 2 with the actual average grade based on the inlet location of step 8. An error in grades of + or - 10 percent indicates an acceptable solution; however, if the error is greater than + or - 10 percent, repeat procedure starting with step 2.

For curb and gutter sections that carry overland side flow as well as pavement runoff, the above procedure is not applicable. However, if the side flow area is of a uniform width parallel to the street, the above procedure, with modifications of steps 6 and 8, can be used. Step 6 should be modified so that W includes the width of the side flow area as well as the street width. The equation in step 8 has a hidden runoff coefficient of 0.9 for paved areas. With runoff also from impervious areas, this must be replaced by a composite runoff coefficient C, which produces the following equation: L = 43560 Q_D/IWC, with L in feet, Q_D in cfs, I in./hr., and W in feet.

If the side flow area is not of a uniform width, the above equation cannot be employed for the spacing of inlets.

Instead, a trial and error solution of runoff versus inlet capacity, gutter capacity, and bypass flow must be calculated in order to determine the required inlet spacing.

Example Problem Spacing of Inlets on a Continuous Grade

Given: For the design data, see example problem "Capacity of a Combination Inlet Grate on a Continuous Grade," which is located in this procedure.

Assume:

1. Only pavement runoff is intercepted by the inlets.
2. For illustrative purposes only, let the longitudinal slope be a constant one percent between the high and low points of the profile.

Find:

1. The location of the first inlet with respect to the high point of the profile.
2. The spacing of all subsequent inlets.

Solution:

1. Average Gutter Grade = 1%
2. From the example problem of FDM 13-25-25, the allowable gutter flow is:

   Q_p = 1.9 cfs
3. From the Example Problem, "Capacity of a Combination Inlet Grate on a Continuous Grade," the allowable inlet capacity for a Type H inlet is: Q_i = .66 cfs
4. From FDM 13-10-5, Attachment 5.4, the five-minute duration, 10-year frequency rainfall intensity for Milwaukee is: I = 6.4 in./hr.
5. The width of the pavement and gutter, (12' lane, 2' gutter) from FDM 13-25-25 is:

   W = 14 feet
6. The design discharge for the distance to the first inlet is:

   $Q_D = Q_p$ = 1.9 cfs
7. The distance from the high point of the grade to the first inlet is:

   $L = 48500\ Q_D\ /IW$

   L = 48500 (1.9)/(6.4) (14)

   L = 1028 feet

From FDM 13-25-15, the recommended maximum spacing of inlets is 300 to 600 feet. Therefore, the above-calculated value is overridden, and the first inlet is placed 600 feet from the high point of the grade.

8. The bypass flow for the first inlet is: $Q_B = Q_D - Q_i$ = 1.9 - .66 = 1.24 cfs
9. For this particular problem, the gutter grade is the same for all inlets, and hence the values of Q_p and Q_i are the same for all inlets.
10. The design discharge for the spacing of subsequent inlets is the lesser of the following:

    $Q_D = Q_p – Q_B$

    Q_D = 1.9 - 1.24 = .66 cfs, or

    $Q_D = Q_i$ = .66 cfs

These two design discharges are the same for this particular design problem; however, this is not the case when the gutter grade varies from inlet to inlet.

11. The spacing of all subsequent inlets is:

    $L = 48500\ Q_D\ /IW$

    L = 48500 (.66)/(6.4) (14)

    L = 357 feet

    Use L equals 355 feet.

Conclusion:

In conclusion, the first inlet should be placed 600 feet from the high point of the grade and all subsequent inlets placed at 355-foot intervals. Of course, the above-computed spacings in many cases are overridden by the required inlet locations described by FDM 13-25-15.

Slotted CMP Surface Drains

In addition to the previously mentioned conventional inlets, the designer may also use slotted cmp surface drains. The following is a complete discussion of where slotted cmp surface drains can be used, what their benefits are, and how they are designed. Moreover, an example problem is included to illustrate the application of the design procedure.

Problem:

1. Interception of sheet flow before it becomes a problem.
2. Elimination of hazardous curbs and ditches on ramps.
3. Modification of existing drainage systems due to widening or increased runoff.
4. Prevalence of ponding at many flush grate median drop inlets.

Solution:

1. A slotted cmp surface drain that is easy to install and maintain and is capable of supporting occasional heavy loads.

Benefit:

1. Slotted cmp surface drains provide the designer and maintenance engineer with a flexible tool for preventive, corrective, and supplemental drainage applications.
2. Curbs and ditches can often be eliminated from otherwise clear recovery areas.
3. Drainage modifications on widening and safety projects can be installed at less cost than conventional methods.
4. Aesthetics can be improved and maintenance costs reduced.
5. Modification of existing inadequate drop inlets can be made cheaper and faster.

The applications of slotted surface drains are limited only by practicality and the imagination of the designer. Each installation should be economically justified; however, borderline cases on new construction should consider maintenance costs and aesthetic value as well as safety benefits that could result by eliminating curbs from otherwise free recovery areas. The economic advantages are more apparent on widening and safety projects where right-of-way is narrow and existing drainage systems must be supplemented. The elimination of curbs and ditches can significantly simplify maintenance operations.

Design

The slotted drains should be installed only in areas where occasional loads can be expected (shoulders, medians, etc.). The exception to this rule is that more frequent loadings can be tolerated in areas where trucks are prohibited (parkways, parking lots, etc.).

In normal installations where sheet flow is intercepted, simple weir formulas can be used to check inlet capacities. In special cases where slotted drains are used in place of conventional drop inlets to pick up channel flow, the slot acts as an orifice when depths reach two and one-half inches. For rough approximations, 40 feet of 18-inch slotted drain will collect as much water as two 36-inch square drop inlets. A detailed hydraulic analysis would be required when installations are proposed that are other than corrective, preventive, or supplemental in nature. Level or near-level grades should be avoided to prevent silting and clogging. In addition, because of shallow depth, slotted drains need an adequate gradient and/or an unrestricted outlet to ensure against freezing solid with ice in cold weather.

Design Procedure

This discussion will address slotted inlets on grade and in sag locations (1).

The interception of flow by slotted inlets on grade and curb-opening inlets on grade is similar. Each inlet is a side weir and the flow, due to the pavement cross slope, is subjected to lateral acceleration. Therefore, because of the similarities, the equations and charts used for curb opening inlets on grade can be used for the design and analysis of slotted inlets on grade.

Slotted inlets in sag locations act as weirs to depths of about 0.2 ft. depending on length and slot width. At depths greater than 0.4 ft., they act as orifices and can be calculated using the following equation.

$$\mathbf{Q_i = 0.8\ LW\ (2\ gd)^{0.5}}$$

Where:

Q_i = interception capacity, (cfs)

W = width of slot, ft

L = length of slot, ft

d = depth of water at slot, ft $\geq$ .4 ft

g = 32.2 ft/s^2

Typically the width of slot, W = 1.75 inches.

$$Q_i = 0.94\ (L)\ d^{0.5}$$

The interception of flow of slotted inlets at depths between 0.2 ft. and 0.4 ft can be computed by use of the orifice equation but one must remember the orifice coefficient varies with depth, length and slot width of the slotted inlet. Use Attachment 30.1 for weir flow, orifice flow and the transition flow at depths between weir and orifice flow.

Example Problem

Slotted CMP Surface Drains

Given: Q = 1.9 cfs (from FDM 13-25-25)

Find: Length of slotted inlet required to limit the maximum depth at curb to 0.25 ft., assuming no clogging.

Solution: The depth of 0.25 ft means this situation is in the transition between weir and orifice flow. Solving for "L" in the equation above yields a length of 4 ft. Using the chart in Attachment 30.2 yields a length of 6 ft. The greater value should be used.

30.7 Literature on Inlet Design

For unusual inlet designs not covered by this procedure, the designer is referred to this procedure's references as well as the following literature:

1. Tapley, G.S., Hydrodynamics of Model Storm Sewer Inlets Applied to Design, Trans. ASCE, Volume 108, 1943, pp. 409-452.
2. City of Los Angeles, Hydraulic Characteristics of Curb Opening Inlets - Catch Basins and Connecting Pipe as Determined by Experimental Hydraulic Model Studies, Los Angeles, California, 1953-55, 60 pp.
3. City of Los Angeles, Design Charts for Catch Basin Openings as Determined by Experimental Hydraulic Model Studies, Office Standard No. 108, Los Angeles, California, 1965.
4. Conner, N.W., Design and Capacity of Gutter Inlets, Proc. Highway Research Board, Volume 25, 1945, pp. 101-104.
5. Larson, C.L., Experiments on Flow Through Inlet Gratings for Street Gutters, Highway Research Board Research Report 6-B, Washington, D.C., 1948, pp. 17-29.
6. Larson, C.L., Grate Inlets for Surface Drainage of Streets and Highways, Bulletin No. 2, St. Anthony Falls Hydraulic Laboratory, University of Minnesota.
7. Department of the Army, Corps of Engineers, Airfield Drainage Structure Investigation, St. Paul District Suboffice, Hydraulic Laboratory Report No. 54, Iowa City, Iowa, April 1949, 144 pp.
8. Guillov, J.C., The Use and Efficiency of Some Gutter Inlet Grates, University of Illinois, Engineering Experiment Station Bulletin No. 450, University of Illinois Press, 1959.
9. Wasley, R.J., Hydrodynamics of Flow Into Curb-Opening Inlets, Stanford University, Civil Engineering Department, Technical Report No. 6, Stanford, California, November 1960, 146 pp.

10. Karaki, S.S., and Haynie, R.M., Depressed Curb-Opening Inlets - Supercritical Flow - Experimental Data, Colorado State University Research Foundation, Civil Engineering Section, Fort Collins, Colorado, June 1961, 70 pp.
11. Cassidy, J.J., Generalized Hydraulic Characteristics of Grate Inlets, Highway Research Board Record No. 123, Washington, D.C., 1966, pp. 36-48.
12. Bauer, W.J., and Woo, D.C., Hydraulic Design of Depressed Curb-Opening Inlets, Highway Research Board Record No. 58, Washington, D.C., 1964, pp. 61-80.
13. Izzard, C.F., Tentative Results on Capacity of Curb-Opening Inlets, Highway Research Board, Research Report 11-B, Washington, D.C., 1950, pp. 36-54.
14. Water Environment Federation Design and Construction of Sanitary and Storm Sewers, ASCE No. 37 or WPCF No. 9, New York, New York, 1991, 332 pp.

30.8 References

1. Neenah Foundry Company, Neenah Inlet Grate Capacities for Gutter Flow and Ponded Water, Neenah, Wisconsin, 1976.

2. John Hopkins University, The Design of Storm-Water Inlets, Baltimore, Maryland, June 1956, 193 pp.

3. U.S. Department of Transportation, Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, Washington, D.C., 1984, 136 pp.

LIST OF ATTACHMENTS

Attachment 30.1	Reduction Factors for Inlets
Attachment 30.2	Performance Curves for Slotted CMP Surface Drains

FDM 13-25-35 Hydraulic Design of Storm Sewers — *August 8, 1997*

35.1 Background Information

This procedure lists the available design aids and discusses the theoretical concepts needed to hydraulically design a storm sewer system operating under full flow and pressure flow conditions. In addition, criteria for pipe diameter strength, alignment, and flow line depth are discussed under the heading "Standards for Storm Drain Pipes."

Further discussions on storm sewer design may be found in the ASCE book entitled "Design and Construction of Sanitary and Storm Sewers" (1).

35.2 Design Aids

The first flow friction formula used to design closed conduits (partially full, full, or pressure flow) and open channels was published by Kutter in 1869 and is known as Kutter's Formula. Since then, additional flow friction formulas that have gained widely acceptable usage are:

1. The Darcy-Weisbach Equation.
2. The Manning Formula.
3. The Hazen-Williams Formula.

Because of its simplicity, the Manning Formula is used by the Department of Transportation for the design of closed conduits under partially full, full, or pressure flow conditions. For the Manning Formula, the full flow capacity of a specific pipe size is a function of pipe slope and roughness coefficient (Manning's n equals Kutter's n) (see Attachment 35.1).

The design of closed conduits in a partially full flow condition through the direct application of the Manning Formula can be accomplished only through trial and error. However, faster design of closed conduits in partially full flow, full flow, or pressure flow may be accomplished through the use of one or more of the following design aids:

1. Circular pipe flow charts - Bureau of Public Roads, "Design Charts for Open-Channel Flow," Hydraulic Design Series No. 3, Charts 35 to 52 (5).
2. A nomograph for the solution of the Manning Formula in conjunction with a graph of hydraulic elements for a circular section - See Attachment 35.2 and Attachment 35.3, respectively.

3. Nomographs for the direct solution of pipe flow - See Attachment 35.4 and Attachment 35.5.
4. Slide rules - Solution of Manning Formula, copyright 1973, American Concrete Pipe Association; and Solution of Kutter's Formula, copyright 1947, 1961, Irving Goldfein, Civil Engineer, Bureau of Engineers, Municipal Building, Milwaukee, Wisconsin.

35.3 Conduit Design - Full Flow

Tentatively, the pipe gradient is set equal to the pavement gradient, then a pipe size is selected that approximately equals the design flow under full flow conditions. Generally, no standard size pipe will carry the design flow exactly at full depth flow. Therefore, the next larger size pipe must be selected, the pipe gradient modified, or both.

Some publications state that storm sewer pipes should be designed for a 0.8 full flow condition. However, the capacity of a pipe is the same at a 0.8 full flow condition as at a full flow condition, and hence either design method will produce the same required pipe size. Although the capacity of a pipe is largest between a 0.8 full flow condition and a full flow condition, pipes should never be designed for flow in this very unstable, unpredictable flow region.

Under special conditions, such as connecting to an existing undersized storm sewer system, or backwater from a receiving stream, etc., pipes may be allowed to operate under pressure, provided the hydraulic head does not cause any pavement flooding or property damage.

For normal full flow pipe design, no allowance need be or shall be made for energy losses at bends, joints, and transitions, unless anticipated high-energy losses could cause flooding problems. When pressure flow conditions are encountered, the system must be designed for the high-energy losses produced by the pressure-induced high flow velocities. Too large a pressure flow can cause pavement flooding, basement flooding, manhole cover popping, etc.

35.4 Pressure Flow

Storm sewer systems operating under pressure flow must be designed for energy losses (head losses). These energy losses are used to determine the energy grade line and the hydraulic grade line of a storm sewer system. This section briefly explains these hydraulic concepts.

There are six categories of energy loss that should be considered. They are:

- Manhole losses
- Inlet losses
- Entrance losses
- Exit losses
- Bend losses
- Friction losses

35.4.1 Manhole Losses

Manhole losses may be determined by using the procedure presented in the "Urban Drainage Design Manual" (4). The manhole loss coefficient for storm drain design can be evaluated by K x ($V_o^2/2g$) where K can be approximated by:

$$K = K_o C_D C_d C_Q C_p C_B$$

Where:

- K = adjusted loss coefficient.
- K_o=initial head loss coefficient based on relative manhole size.
- C_D=correction factor for pipe diameter (pressure flow only).
- C_d=correction factor for flow depth (non-pressure flow only).
- C_Q=correction factor for relative flow.
- C_B=correction factor for benching.
- C_p=correction factor for plunging flow.

A discussion follows on each of the correction factors.

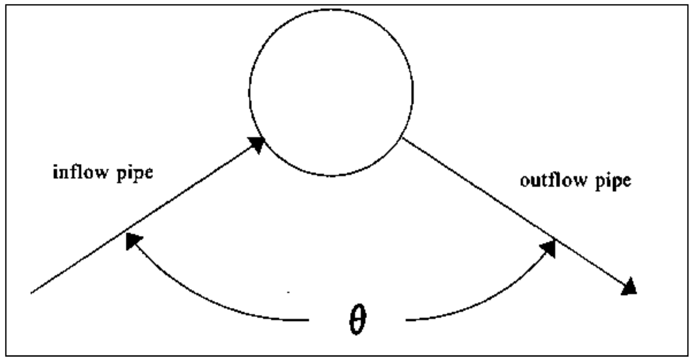
Relative Manhole Size:

K_o is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes.

$K_o = 0.1\ (b/D_o)(1-\sin\theta) + 1.4\ (b/D_o)^{0.15} \sin\theta$

Where:

- θ = the angle between the inflow and outflow pipes
- b = manhole diameter
- D_o = outlet pipe diameter



Pipe Diameter:

The correction factor for pipe diameter is significant only in pressure flow situations when the ratio of the water depth in the manhole(d) to the outlet pipe diameter (D_o), d/D_o, is greater than 3.2.

$C_D = (D_o/D_i)^3$

Where:

- D_i = incoming pipe diameter
- D_o = outgoing pipe diameter

Flow Depth: The correction factor is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases. The water depth in the manhole is approximated as the level of the hydraulic grade line at the upstream end of the outpipe. The correction factor for flow depth, C_d is calculated by:

$C_d = 0.5\ (d/D_o)^{0.6}$

Where:

- d =water depth in manhole above outlet pipe
- D_o = outlet pipe diameter

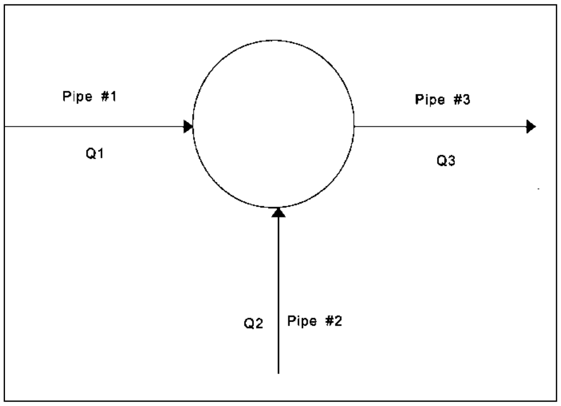
Relative Flow:

The correction factor for relative flow, C_Q, is a function of the percentage of flow coming in through the pipe of interest as well as the angle of the incoming flow versus other incoming pipes. It is calculated by the following:

$C_Q = (1-2\sin\theta) \times [1 - (Q_i/Q_o)]^{0.75} + 1$

Where:

- C_Q =correction factor for relative flow
- θ =the angle between the inflow and outflow pipes.
- Q_i =flow in the inflow pipe
- Q_o =flow in the outflow pipe



The example below illustrates two situations to determine the impact of pipe #2 entering the manhole.

Example 1

Q_1 = 6 cfs, Q_2 = 3 cfs

Q_3 = 9 cfs then

$C_Q = (1\text{-}2 \sin 180°) \times [1 - (6/9)]^{.75} + 1$

$= 1.44$

Example 2

Q_1 = 3 cfs, Q_1 = 6 cfs

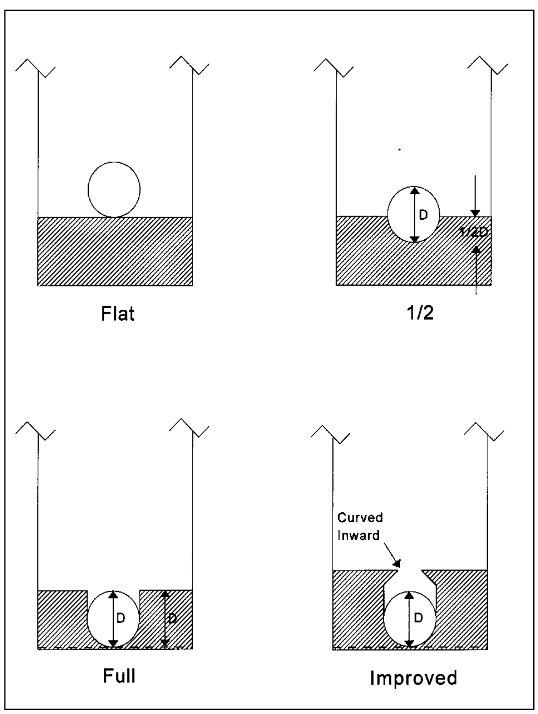
Q_3 = 9 cfs then

$C_Q = (1\text{-}2 \sin 180°) \times [1 - (3/9)]^{.75} + 1$

$= 1.74$

** Free surface flow, $d/D_o < 1.0$

A linear interpolation is performed for flow depths between the submerged and unsubmerged conditions. The following schematic shows each of the four conditions described above.



To estimate the head losses through a manhole from the outlet pipe to a particular inlet pipe, multiply the correction factors together to get the head loss coefficient, K. Then, multiply K by the velocity head in the outflow pipe to estimate the minor loss for the connection.

Manhole losses may also be determined by using the design methodologies, design charts and examples of pressure flow design given in the University of Missouri Bulletin entitled "Pressure Changes at Storm Drain Junctions" (2).

35.4.2 Inlet Losses

Manhole losses Inlet losses may be determined by using the design methodologies, design charts and examples of pressure flow design given in the University of Missouri Bulletin entitled "Pressure Changes at Storm Drain Junctions" (2).

35.4.3 Entrance, Exit, and Bend Losses

The general equation for these losses, expressed as a function of pipe flow velocities, is:

$$H = K\,V^2/2g$$

Where:

- H =head loss
- K =loss coefficient
- V = average pipe velocity
- g = 32.2 ft/sec^2

Entrance losses need only be considered when the storm sewer originates at a culvert. Entrance loss coefficients K_e for various entrance conditions can be obtained from HDS #5 (3), Table 12.

Exit losses for sewer pipe discharging into a receiving stream will produce an energy loss at its outlet equivalent to one velocity head; K equals 1.0.

Bend loss coefficient K_b values for curvilinear and miter bends can be obtained from Attachment 35.6 and Attachment 35.7.

35.4.4 Friction Losses

The largest losses in a storm sewer system are friction losses. They are directly related to the velocity in the pipe and hence the higher the velocity, the greater the friction loss and vice versa. The slope of the friction loss can be estimated by using Attachment 35.4 (corrugated metal pipe) and Attachment 35.5 (concrete).

The total frictional head loss in a given length of pipe can be computed with the following equation:

$$H_f = S_f L$$

Where:

- H_f=head loss for friction
- S_f=slope of the energy grade line
- L =length of the conduit

35.5 Energy and Hydraulic Grade Lines (EGL and HGL)

The energy grade line shows the total energy at any point in a storm sewer, whereas the hydraulic grade line shows the pressure head or the water surface level in open tubes if they are inserted in the pipe. The EGL must always drop in the direction of flow; however, the HGL may rise at hydraulic structures, such as manholes.

The EGL and the HGL might need to be calculated when part of the storm sewer system might be operating under pressure whether or not the outfall is submerged. These computations are made starting at the outfall where the EGL and HGL coincide at the water surface of the discharge pond.

The EGL for a storm sewer is determined by adding the energy losses in a progressive manner from the outfall to the upper end of the system. The elevation of the HGL can be determined by subtracting from the elevation of the EGL the value of the velocity head ($V^2/2g$) for each individual pipe.

For a step-by-step methodology of the determination of the EGL and HGL for surcharged full flow, see FDM 13-25-45.

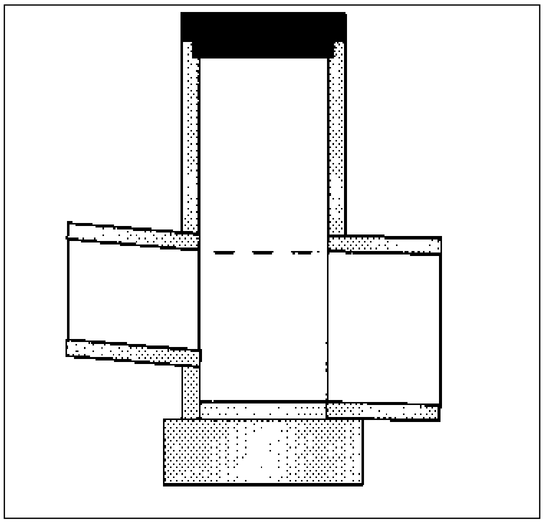
35.6 Hydraulic Standards for Storm Drain Pipe

35.6.1 Minimum Pipe Slope

Minimum full flow velocity shall be 2.5 fps, and preferably 3 fps, to prevent deposition of solids. If the design flow rate based on future conditions is appreciably larger than the present flow rate, it may be advisable that the minimum pipe slope be checked with the present flow rate. Desirable full flow velocity shall be 10 to 15 fps. For some standard size concrete pipe (n = 0.013), the minimum slopes required to maintain a self-cleaning velocity of 2.5 or 3.0 fps at full flow are as follows:

Pipe Diameter (Inches)	Minimum Slope (Ft./Ft.)	
	2.5 fps	3.0 fps
12	.0030	.0044
15	.0023	.0032
18	.0018	.0025
24	.0012	.0017

In the majority of cases, the flow line depth is determined by the conduit size and the slope requirements. However, additional factors, such as hydraulic grade line elevations, lateral connections, vertical clearance of obstructions, etc., may also, in certain cases, control the required flow line depth. Moreover, the flow line depth of the conduit should be set to maintain the calculated hydraulic grade line (water surface elevation) at inlets, junction chambers, and manholes at one foot or more below the grate or cover. If practicable, the crowns of pipes connecting to inlets, junctions, and manholes should be held at the same elevation. See the sketch below.



35.7 References

1. American Society of Civil Engineers and Water Pollution Control Federation, Design and Construction of Sanitary and Storm Sewers, ASCE No. 37 or WPCF No. 9, New York, New York, 1991, 332 pp.

2. Sangster, W.M., Wood, H.W., Smerdon, E.T., and Bossy, H.G., "Pressure Changes at Storm Drainage Junctions," University of Missouri, Engineering Experiment Station Bulletin 41, 1958, 132 pp.

3. U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September 2001.

4. U.S. Department of Transportation, Federal Highway Administration, Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22, FHWA-NHI-01-021, August 2001.

5. U.S. Department of Transportation, Federal Highway Administration, Design Chart for Open - Channel Flow, Hydraulic Design Series No. 3, August 1961.

LIST OF ATTACHMENTS

Attachment 35.1	Manning Roughness Coefficients
Attachment 35.2	Graphic Solution of the Manning Equation
Attachment 35.3	Hydraulic Elements of a Circular Section

FDM 13-25-40 Design Procedure: Full and Partially Full Flow *August 8, 1997*

40.1 Background Information

The following procedure describes the hydrologic and hydraulic design of storm sewers operating under full or partially full flow conditions. For a detailed procedure on the design of storm sewers operating under a surcharged flow condition, see FDM 13-25-45.

40.2 Procedure

1. Prepare a set of prints or maps showing the entire drainage area contributing runoff to the proposed storm sewer.
2. Through the use of FDM 13-25-30, draw on the prints the design locations of all catch basins and inlets.
3. Locate trunk sewer and manholes on the plan; number manholes, catch basins, and inlets beginning from the upper end of the system; and assign a letter designation to each trunk line and lateral line beginning from the upper end of the system.
4. Tentatively sketch on the profile sheet a trunk sewer grade that is approximately parallel to the street profile.
5. The area contributing to each inlet or catch basin should:
    - Be delineated on the map.
    - Be numbered sequentially starting at the upper end of the system.
    - Have its area measured in acres and marked on the map.
    - Have a weighted runoff coefficient C estimated and marked on the map.
    - Have the inlet time (time of concentration) estimated and marked on the map (minimum time of concentration must be five minutes).
6. Complete the work sheet, Attachment 40.1, for storm sewer design as follows:
    a. Under the heading of "Location," enter the following:
        - On the first line, the name of the street containing the sewer line.
        - In column 1, the station of the upstream structure for the pipe run under consideration.
        - In column 2, the structure type (M.H.-manhole, I-inlet, C.B.-catch basin) and the structure number of the upstream structure.
        - In column 3, the structure type and the structure number of the downstream structure.

    b. Under the heading entitled "Tributary Area," enter the following:
        - In column 4, the index number for each subarea.
        - In column 5, the size in acres of each subarea.
        - In column 6, the weighted runoff coefficient C for each subarea.
        - In column 7, the equivalent area (product of columns 5 and 6) of each subarea.
        - In column 8, the sum of all the equivalent areas for the pipe section under consideration.

    c. Under the heading of "Travel Time," enter the following:
        - In column 9, the inlet time for each subarea. For the first inlet of a system, the inlet time is the same as the time of concentration of the system. On subsequent inlets, the inlet time is equal to the time of concentration for each subarea. If the inlet time exceeds the time of concentration from the upstream basin, and the subarea tributary to the inlet is of sufficient magnitude, the inlet time should be substituted for the time of concentration and used for this and subsequent design points.
        - In column 10 or column 11, the appropriate flow time between the upstream structure and the downstream structure. If a significant portion of the flow is carried by the street, the street flow time should be entered in column 10. However, pipe flow volume generally is

more significant than street flow volume, and hence pipe flow time is usually entered in column 11. If there is any question as to which is the controlling time, both times should be computed, entered, and compared (see e.8).

- In column 12, the time of concentration, which is the greater of:
    - The sum of the previous design point (the inlet end of a pipe) time of concentration and the intervening flow time; or,
    - The inlet time for the present design point.

d. Under the heading entitled "Rainfall-Runoff," enter the following:

- In column 13, average rainfall intensity for a rainfall duration equal to the time of concentration (column 12) and the selected design frequency. Obtain the rainfall intensity from the appropriate I-D-F curve, FDM 13-10-5, Attachment 5.4.
- In column 14, the direct runoff (product of columns 8 and 13).
- In column 15, other runoff, such as controlled releases from rooftops, parking lots, base flows from groundwater, and any other source.
- In column 16, the design runoff (summation of columns 14 and 15).

e. Use the columns under the heading of "Flow in Conduit" and the following procedure to design the conduit:

- Enter in column 17 a trial slope for the sewer pipe. Usually the slope of the roadway can be used.
- Determine from either FDM 13-25-35, Attachment 35.4, or FDM 13-25-35, Attachment 35.5 the pipe size by laying a straightedge between the discharge (column 16) and slope (column 17) scales. The appropriate size pipe is read directly above the straightedge. Enter the value in column 18.
- Adjust the straightedge on the nomograph so that it lies on the slope (column 17) and the pipe size (column 18). Read the capacity flowing full on the discharge scale. If this value is 10 percent larger than the design runoff (column 16), then, if feasible, the slope of sewer should be flattened. Through using the discharge (column 16) and pipe size (column 18) scales of the nomograph, a new slope at which the pipe just flows full can be read from the slope scale.
- If a pipe slope adjustment is made, reenter the new value in column 17.
- Enter in column 19 the capacity flowing full for the selected pipe and slope.
- Enter in column 20 the mean velocity flowing full by laying a straightedge on the slope (column 17) and pipe size (column 18) scales of the nomograph. This value should be greater than three fps.
- Enter in column 21 the length of pipe, which is equal to the distance between the center lines of the manholes.
- Enter in column 11 the pipe flow time determined by dividing the length (column 21) by the velocity (column 20). Convert from seconds to minutes.
- Enter in column 22 the fall of pipe.

f. Under the heading of "Vertical Control," enter the following:

- In column 23, the invert elevation for the upper end of the pipe.
- In column 24, the invert elevation for the lower end of the pipe.
- In column 25, the top of structure elevation for the upper end of the pipe.
- In column 26, the top of structure elevation for the lower end of the pipe.

40.2.1 Example Problem

This example problem shows the application of the above-outlined procedure to the design of a storm sewer system operating under full or partially full conditions. See Attachment 40.2 for a plan and profile layout of the proposed storm sewer system. The design computations are also shown in this Attachment and are self-explanatory by following the procedure previously outlined.

Since this is an example problem, only the sewer trunk line is designed for explanation purposes. In an actual problem, the lateral pipes as well as the inlets (see FDM 13-25-30) would have to be designed.

LIST OF ATTACHMENTS

FDM 13-25-45 Design Procedure: Surcharged Full Flow

August 8, 1997

45.1 Background Information

The purpose of this procedure is to determine the effect of backwater on a storm sewer system or the effects of an existing underdesigned storm sewer system that operates in a surcharged condition. All storm sewers that will operate under a submerged condition shall be checked by this section to ensure that low points along the highway will not be inundated during the design storm.

45.2 Procedure

When a sewer system operates under a surcharged condition, the elevation of the water surface (hydraulic grade line - HGL) may be raised high enough to cause the water to bubble out of the manholes and inlets at low points in the highway grade. The elevation of the HGL is equal to the energy grade line (EGL) minus the velocity head. Therefore, the EGL, which is the total energy in the system (potential energy plus kinectic energy), must be calculated first. See Attachment 45.1 for the profiles, EGL's, and HGL's of two different storm sewer systems - one improperly designed (HGL above natural ground) and one properly designed (HGL below natural ground).

A discussion of the energy losses associated with surcharged flow is contained in FDM 13-25-35. For the sake of expediency, the following approximations of loss coefficient at junctions can be used:

1. A 90° turn in main line use 1.5 x velocity head
2. Through flow in main line use 0.2 x velocity head
3. Through flow with large lateral use 0.5 - 1.5 x velocity head
4. First inlet in system use 1.5 x velocity head

Each of the above conditions uses the velocity head of the downstream pipe.

If the calculations show inundation of the pavement during the design storm, the storm sewer system should be redesigned with larger pipe sizes, which reduces the pipe friction loss and hence lowers the HGL.

1. Initially, design the storm sewer system by FDM 13-25-40, "Design Procedure: Full and Partially Full Flow." Assume there is free outfall from the storm sewer.
2. Draw a profile of the proposed sewer showing the highway grade and the location of each manhole and each inlet, along with their cover elevations.
3. Use tabular design sheet "Work Sheet for Storm Sewer Design - Surcharged Flow," Attachment 45.2.
4. Under the heading of "Location," enter the following:
    - In column 1, the station of the sewer outfall or the next structure.
    - In column 2, the structure type (outfall, M.H.-manhole, I-inlet, C.B.-catch basin, etc.) and the structure number.
5. Under the heading entitled "Pipe Data," enter the following:
    - In column 3, the design discharge for the upstream pipe.
    - In column 4, the pipe size of the upstream pipe.
    - In column 5, the pipe length of the upstream pipe.
6. Under the heading of "Velocity Head," enter the following:
    - In column 6, the mean pipe velocity of the upstream pipe.
    - In column 7, the pipe velocity head ($V_1^2/2g$) of the upstream pipe.
    - In column 8, the mean channel velocity component in the outlet channel (which is parallel to the sewer). In most cases this can be considered negligible. This column is used only for the outflow pipe.
    - In column 9, the channel velocity head for the velocity in column 8. In most cases this can be considered negligible. The outlet losses of the outlet pipe should be reduced by this amount.
7. Under the heading of "Pipe Head Losses," enter the following:

- In column 10, the bend loss coefficient K for any bends in this length of upstream pipe.
- In column 11, the bend energy loss (product of columns 7 and 10).
- In column 12, the friction slope as determined from FDM 13-25-35, Attachment 35.4 or FDM 13-25-35, Attachment 35.5 with the discharge (column 3) and the pipe size (column 4). This is the friction slope S_f of the EGL.
- In column 13 the friction head loss is obtained by multiplying (column 5) x (column 12)

8. Under the heading of "Structure Head Losses," enter the following:
   - In column 14, the coefficient K for the appropriate structure losses (outlet, usually 1.00; inlet and manhole as discussed in this procedure and FDM 13-25-35).
   - In column 15, the structure energy losses (product of column 7, pipe velocity head, and column 14, coefficient K structure). Note: Use the upstream pipe velocity head for an outlet structure and the downstream pipe velocity head for all other structure types.
9. Under the heading entitled "Grade Line Elevation at Structure," enter the following:
   - In the upper half of column 16, the downstream EGL elevation. For an outlet structure this is the surface elevation of the receiving body of water. However, for all other structures this elevation is the sum of the upstream EGL elevation of the previous structure (column 16), and the friction head loss (column 13) and the bend energy loss (column 11) of the interconnecting pipe.
   - In the lower half of column 16, the upstream EGL elevation, which is the sum of the downstream EGL elevation (computed under 9a) and the structure energy losses (column 15).
   - In the upper half of column 17, the downstream HGL elevation. For an outlet structure this is the surface elevation of the receiving body of water. However, for a manhole or an inlet this elevation is equal to the downstream EGL elevation (column 16) minus the downstream pipe velocity head (column 7).
   - In the lower half of column 17, the upstream HGL elevation, which is equal to the upstream EGL elevation (column 16) minus the upstream pipe velocity head (column 7). This value is the water surface elevation within the structure.
10. Under the heading of "Vertical Control," enter the following:
    - In column 18, the downstream and upstream invert elevations of the structure. These elevations will usually be the same unless the pipe size changes, and then the upstream elevation is equal to the change in pipe size plus the downstream invert elevation.
    - In column 19, the top of structure elevation, which is the natural ground elevation or the street grade elevation.
    - In column 20, the freeboard height, which is the difference of column 19 (top of structure elevation) and column 17 (upstream HGL elevation).
11. If the freeboard height is negative, the HGL elevation (water surface) is above the highway grade, and larger pipe sizes should be used downstream from the point of inundation in order to reduce the HGL elevation. The pipes should be enlarged sufficiently to allow at least one foot of freeboard at the most critical structure.
12. Repeat the procedure starting with step 4 for the next upstream structure.
13. If a free-water surface is encountered within the conduit, the calculations are normally suspended. However, if a structure further upstream has unusually high energy losses, thus producing surcharged flow, it may be necessary to continue the calculations using the surface of the normal depth of flow for the HGL within the conduit.

Theoretically, the energy losses for partially full flowing conduits should not be computed by the above-listed methods, which are only for full flowing conduits. However, for the sake of expediency it is recommended that the above-cited energy loss methodology for full flowing conduits also be applied to partially full flowing conduits.

Example Problem

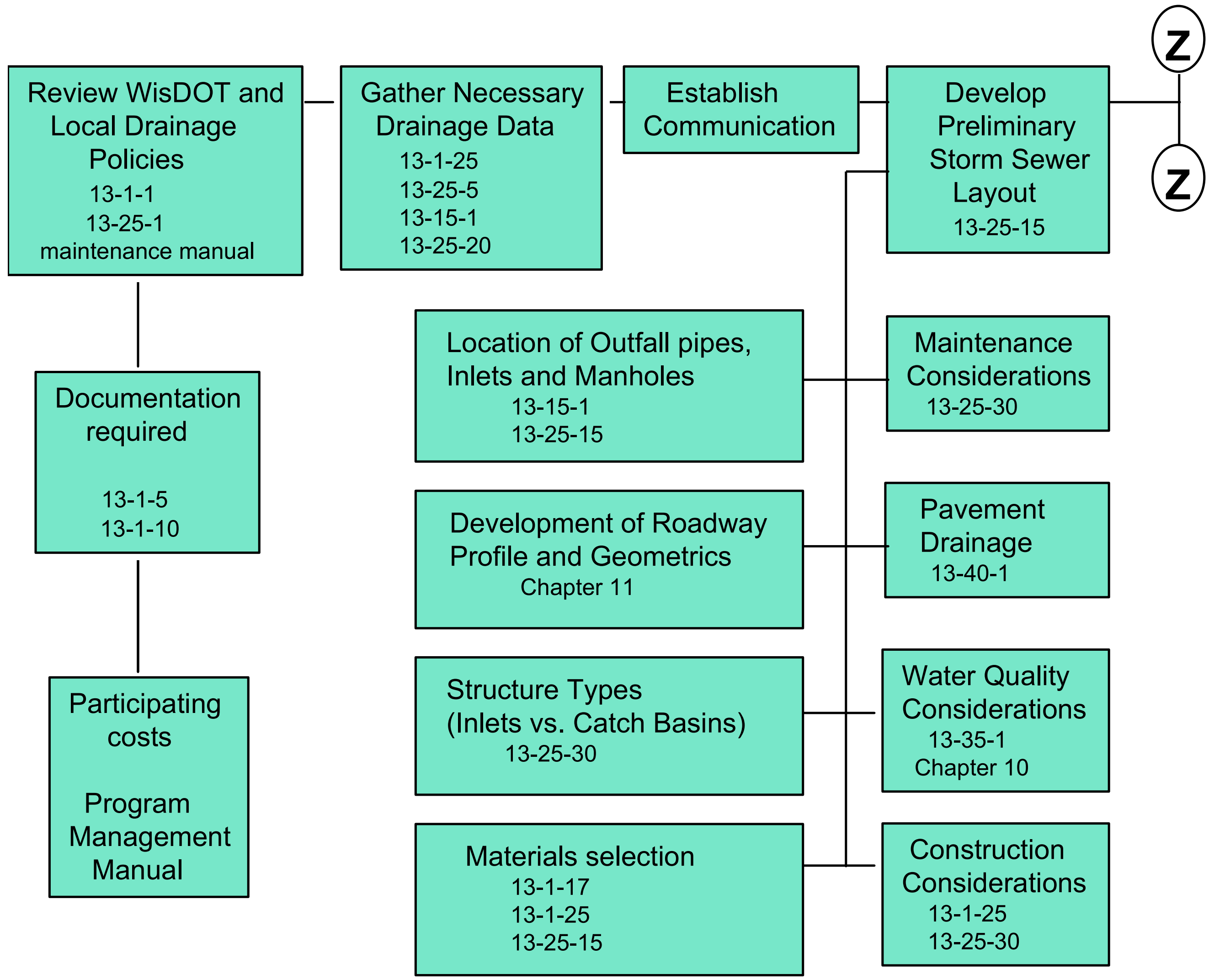
This example problem is a continuance of the example problem from FDM 13-25-40, with the additional design control that the outfall is not a free fall outfall. Instead, a surcharged condition is produced through inundation of the outfall with a water elevation of 990.50. Therefore, the example storm sewer system designed by FDM 13-25-40 must be checked for surcharged flow starting with step 3 of the above-outlined design procedure.

See FDM 13-25-40, Attachment 40.2 for a plan and profile layout of the proposed storm sewer system designed under full and partially full flow conditions. This figure satisfies steps I and 2 of the above-outlined design procedure for surcharged flow. The design computations for surcharged flow are shown in Attachment 45.3 of this procedure and are self-explanatory by following the design procedure outlined above. However, two highlights of this design example problem are:

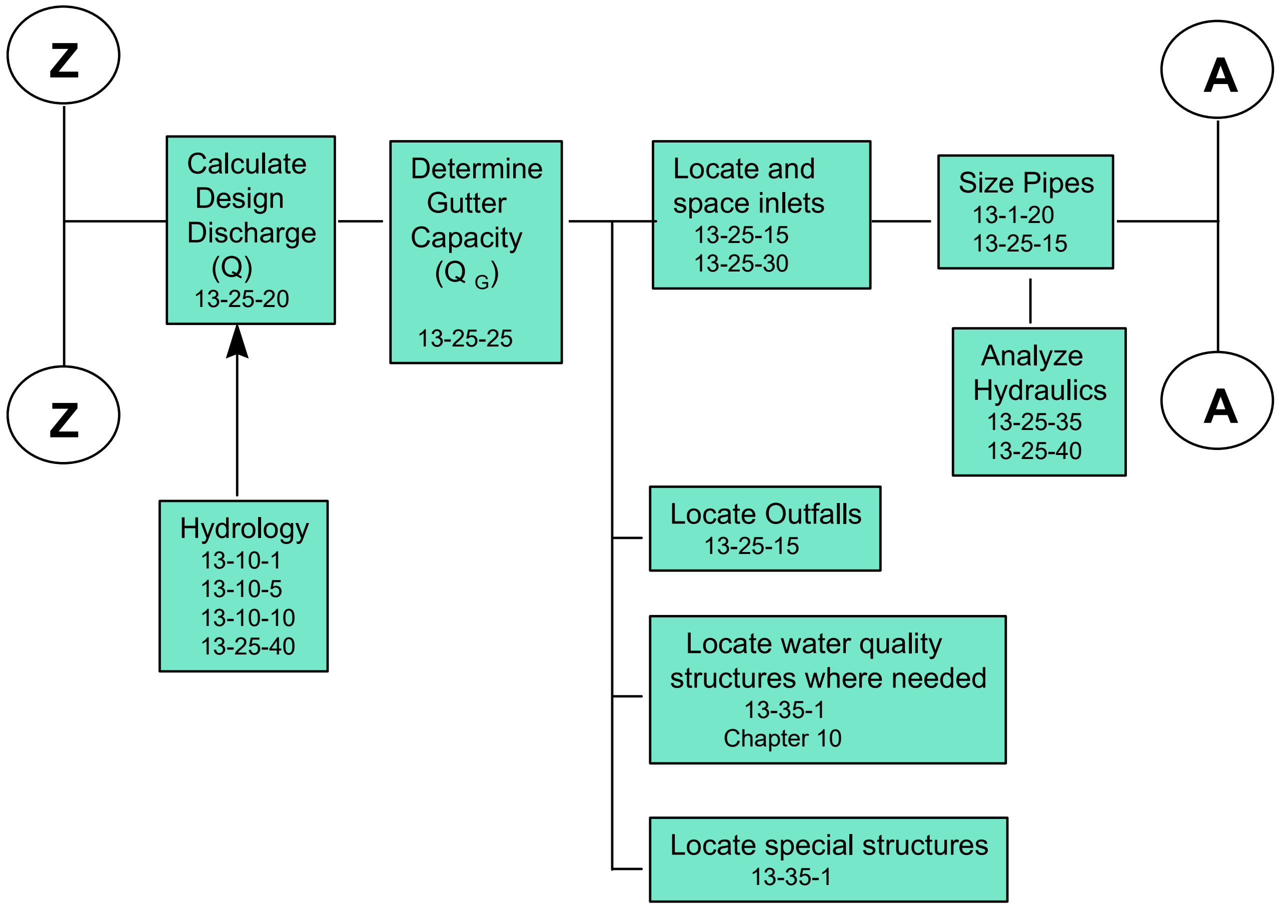
1. The pipe between MH A-3 and A-4 is enlarged from a 36-inch to a 42-inch pipe to eliminate the popping of the cover of MH A-4; and,
2. The computations are discontinued at MH A-2, where a free water surface is encountered.

Since this is an example problem, only the sewer trunk line is checked for surcharged flow. However, in an actual design problem, the lateral inlets must also be checked for surcharged flow.

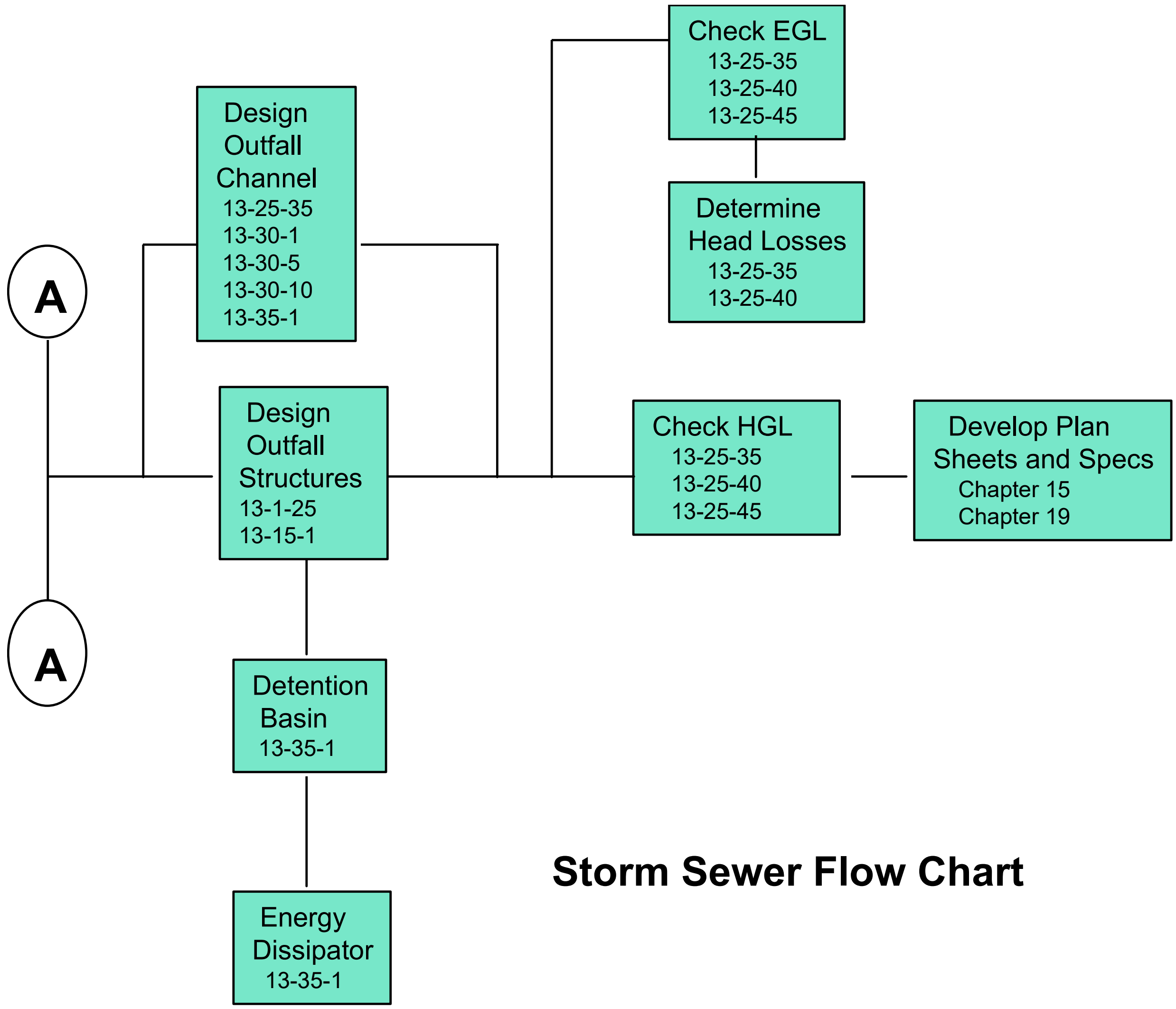
LIST OF ATTACHMENTS



Storm Sewer Flow Chart

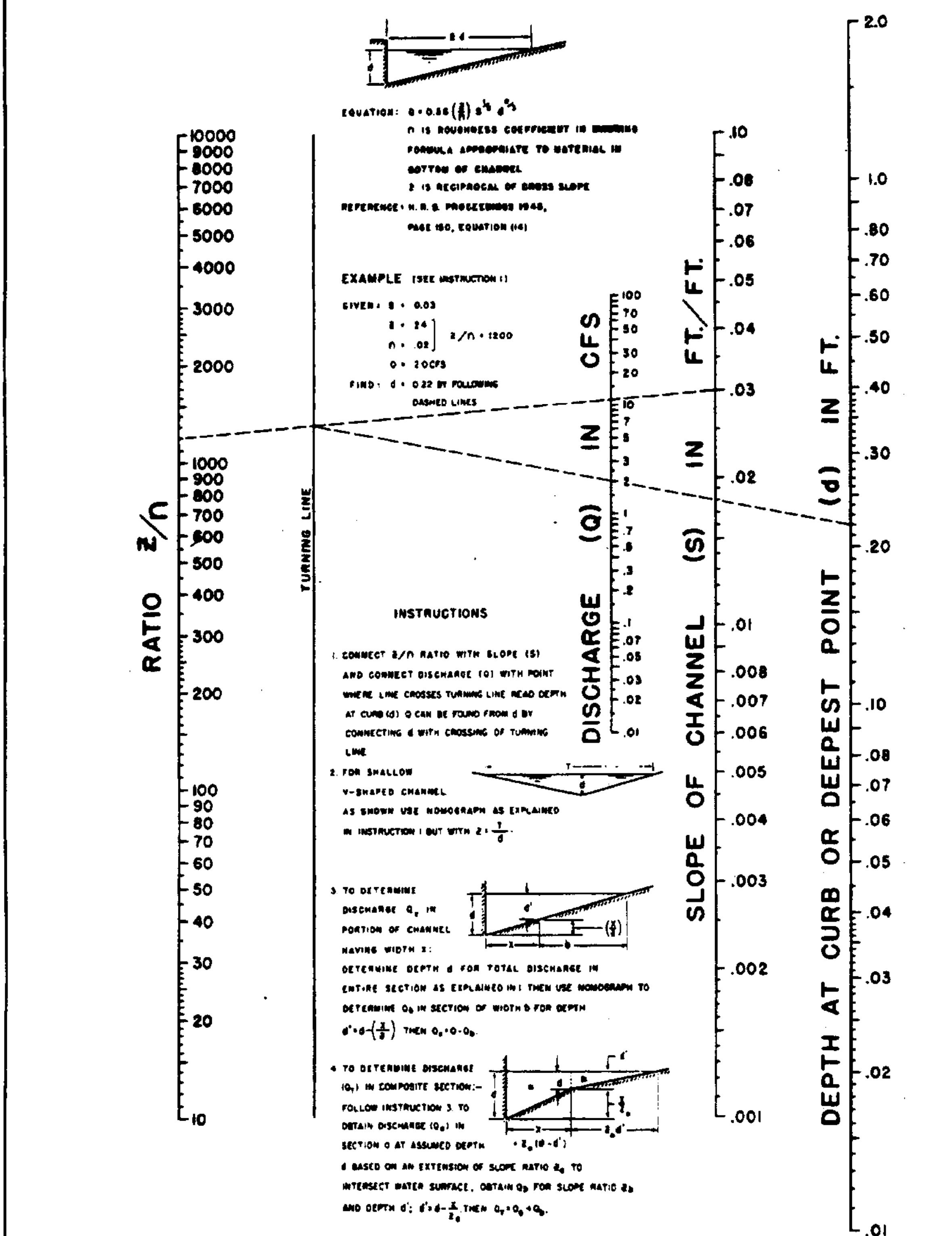


Storm Sewer Flow Chart



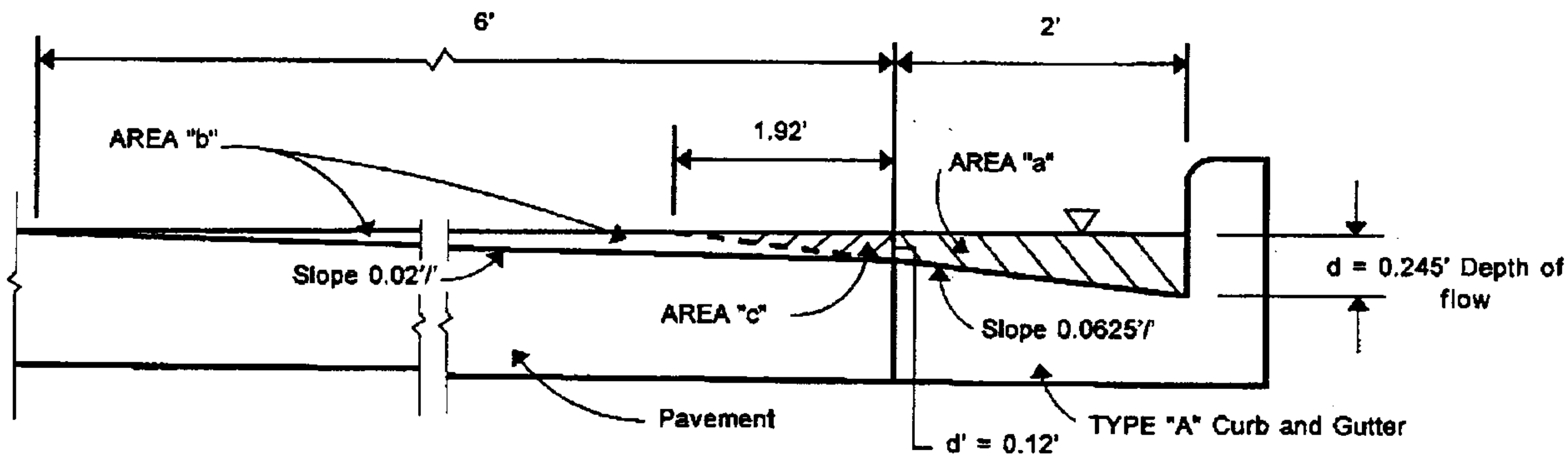
Storm Sewer Flow Chart

Chart 1



NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

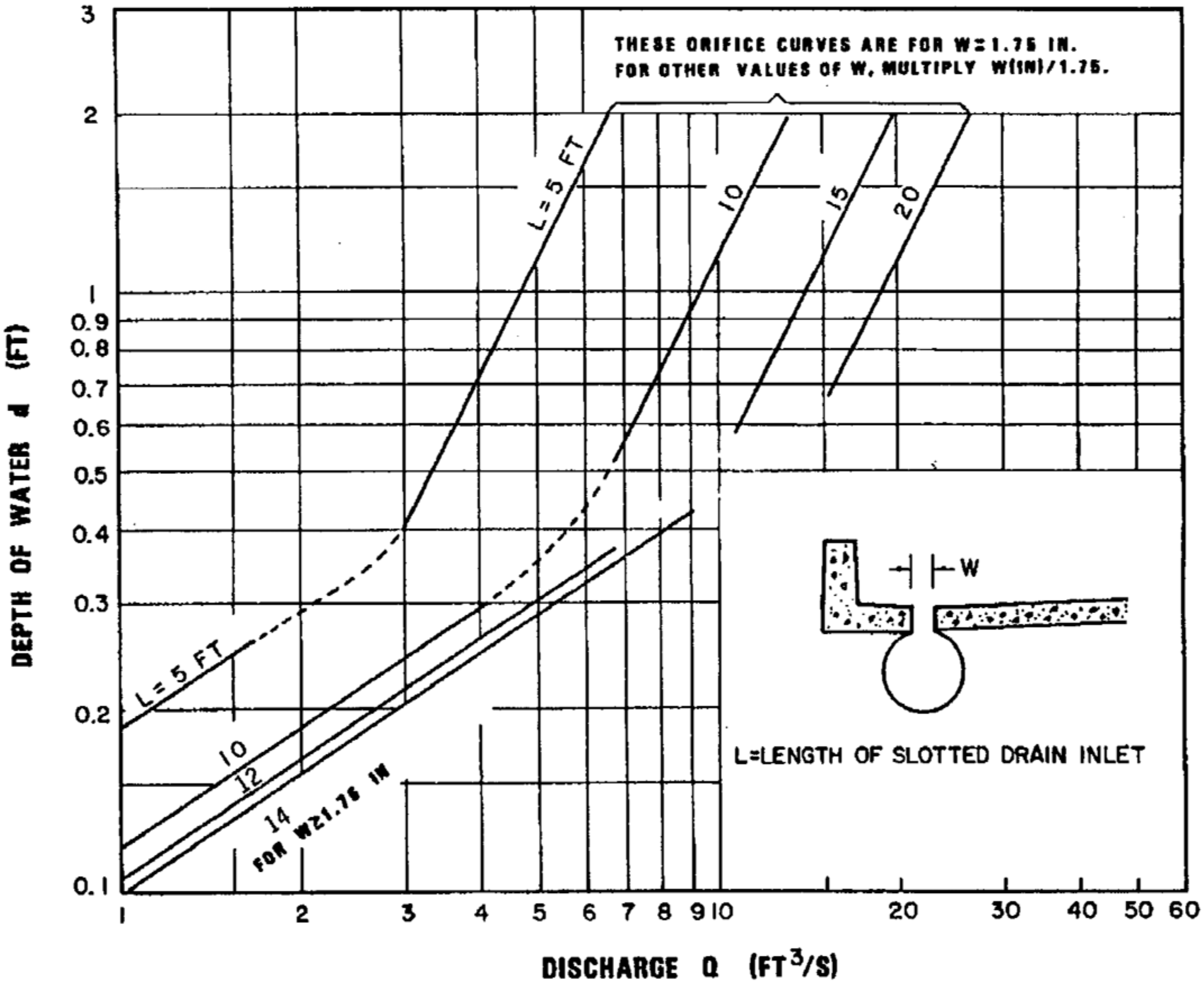
Example Problem
Gutter Design



REDUCTION FACTORS TO APPLY TO INLETS

Condition	Inlet Type	Percentage of Theoretical Capacity Allowed
Sump	Curb Opening	80%
Sump	Grated	50%
Sump	Combination	65%
Continuous Grade	Curb opening	80%
Continuous Grade	Deflector	75%
Continuous Grade	Longitudinal Bar Grated	60%
Continuous Grade	Transverse Bar Grate or Longitudinal Bar Grate incorporating transverse bars	50%
Continuous Grade	Combination	110% of that listed for type of grate utilized

Source: Denver Regional Council of Governments, Urban Storm Drainage-Criteria Manual, Volume 1.



THESE ORIFICE CURVES ARE FOR W=1.75 IN.
FOR OTHER VALUES OF W, MULTIPLY W(IN)/1.75.
L=5 FT
10
15
20
L=5 FT
10
12
14
FOR W=1.75 IN
W
L=LENGTH OF SLOTTED DRAIN INLET
DEPTH OF WATER d (FT)
DISCHARGE Q (FT³/S)
3
2
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60

Table 1.—Manning roughness coefficients, n [1]

	Manning's n range [2]
I. Closed conduits:	
A. Concrete pipe	0.011–0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2⅔ by ½-in. corrugation (riveted pipe): [3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021–0.018
(2) Flow 0.8 depth	0.021–0.016
(3) Flow 0.6 depth	0.019–0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012–0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009–0.011
F. Brick	0.014–0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015–0.017
2. Wood forms, smooth	0.012–0.014
3. Steel forms	0.012–0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017–0.022
2. Natural floor	0.019–0.025
I. Laminated treated wood	0.015–0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined [4] (straight alinement): [5]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013–0.017
2. Trowel finish	0.012–0.014
3. Float finish	0.013–0.015
4. Float finish, some gravel on bottom	0.015–0.017
5. Gunite, good section	0.016–0.019
6. Gunite, wavy section	0.018–0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015–0.017
2. Random stone in mortar	0.017–0.020
3. Cement rubble masonry	0.020–0.025
4. Cement rubble masonry, plastered	0.016–0.020
5. Dry rubble (riprap)	0.020–0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017–0.020
2. Random stone in mortar	0.020–0.023
3. Dry rubble (riprap)	0.023–0.033
D. Brick	0.014–0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011–0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017–0.020
2. Irregular section	0.022–0.027
III. Open channels, excavated [4] (straight alinement,[5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In gravelly soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides clean, gravel bottom	0.025–0.030
5. Sides clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high stage	0.10–0.14

	Manning's n range [2]
IV. Highway channels and swales with maintained vegetation [6] [7] (values shown are for velocities of 2 and 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07–0.045
b. Length 4–6 inches	0.09–0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18–0.09
b. Length about 24 inches	0.30–0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14–0.08
b. Length about 24 inches	0.25–0.13
B. Depth of flow 0.7–1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05–0.035
b. Length 4 to 6 inches	0.06–0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12–0.07
b. Length about 24 inches	0.20–0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10–0.06
b. Length about 24 inches	0.17–0.09
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of n by	*0.002*
VI. Natural stream channels: [8]	
A. Minor streams [9] (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	*0.01–0.02*
2. Irregular sections, with pools, slight channel meander; increase values given in 1a–e about	*0.01–0.02*
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas:	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees: [10]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush: [10]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100–150 per acre:	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited [8] if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028–0.033

from Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"

Footnotes to Table 1 appear on page 2 of this figure

Footnotes to Table 1

[1] Estimates are by Bureau of Public Roads unless otherwise noted.
[2] Ranges indicated for closed conduits and for open channels, lined or excavated, are for good to fair construction (unless otherwise stated). For poor quality construction, use larger values of *n*.
[3] *Friction Factors in Corrugated Metal Pipe*, by M. J. Webster and L. R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HY9, Sept. 1959, Paper No. 2148, pp. 35–67.
[4] For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select *n* by comparison of the specific conditions with the channels tested:
Flow of Water in Irrigation and Similar Channels, by F. C. Scobey, Division of Irrigation, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 652, Feb. 1939; and
Flow of Water in Drainage Channels, by C. E. Ramser, Division of Agricultural Engineering, Bureau of Public Roads, U.S. Department of Agriculture, Tech. Bull. No. 129, Nov. 1929.
[5] With channel of an alinement other than straight, loss of head by resistance forces will be increased. A small increase in value of *n* may be made, to allow for the additional loss of energy.
[6] *Handbook of Channel Design for Soil and Water Conservation*, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station; published by the Soil Conservation Service, U.S. Department of Agriculture, Publ. No. SCS-TP-61, Mar. 1947, rev. June 1954.
[7] *Flow of Water in Channels Protected by Vegetative Linings*, by W. O. Ree and V. J. Palmer, Division of Drainage and Water Control, Research, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 967, Feb. 1949.
[8] For calculation of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey, to obtain data regarding values of *n* applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of *n* tabulated have been derived from data reported by C. E. Ramser (see footnote 4) and from other incomplete data.
[9] The tentative values of *n* cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of *n* must be increased to provide for the additional loss of energy caused by bends. The increase may be in the range of perhaps 3 to 15 percent.
[10] The presence of foliage on trees and brush under flood stage will materially increase the value of *n*. Therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. For trees in channel or on banks, and for brush on banks where submergence of branches increases with depth of flow, *n* will increase with rising stage.

Table 2.—Permissible velocities for channels with erodible linings, based on uniform flow in continuously wet, aged channels [1]

Soil type or lining (earth; no vegetation)	Maximum permissible velocities for—		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	F.p.s.	*F.p.s.*	*F.p.s.*
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	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 (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	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

[1] As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

Table 3.—Permissible velocities for channels lined with uniform stands of various grass covers, well maintained [1] [2]

Cover	Slope range	Permissible velocity on—	
		Erosion resistant soils	Easily eroded soils
	Percent	*F.p.s.*	*F.p.s.*
Bermudagrass	0–5	8	6
	5–10	7	5
	Over 10	6	4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0–5	7	5
	5–10	6	4
	Over 10	5	3
Grass mixture	0–5	5	4
	5–10	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0–5	3.5	2.5
Common lespedeza [3] Sudangrass [3]	[4] 0–5	3.5	2.5

[1] From *Handbook of Channel Design for Soil and Water Conservation* (see footnote 6, table 1, above).
[2] Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.
[3] Annuals, used on mild slopes or as temporary protection until permanent covers are established.
[4] Use on slopes steeper than 5 percent is not recommended.

Table 4.—Factors for adjustment of discharge to allow for increased resistance caused by friction against the top of a closed rectangular conduit [1]

D/B	Factor
1.00	1.21
.80	1.24
.75	1.25
.667	1.27
.60	1.28
.50	1.31
.40	1.34

[1] Interpolations may be made.

Table 5.—Guide to selection of retardance curve

Average length of vegetation	Retardance curve for—	
	Good stand	Fair stand
6–10 inches	C	D.
2–6 inches	D	D.

from Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"

Graphic Solution of the Manning Equation

FIGURE 2 is a nomograph for the solution of the Manning equation:

$$V=\frac{1.49}{n} R^{2/3} S^{1/2}.$$

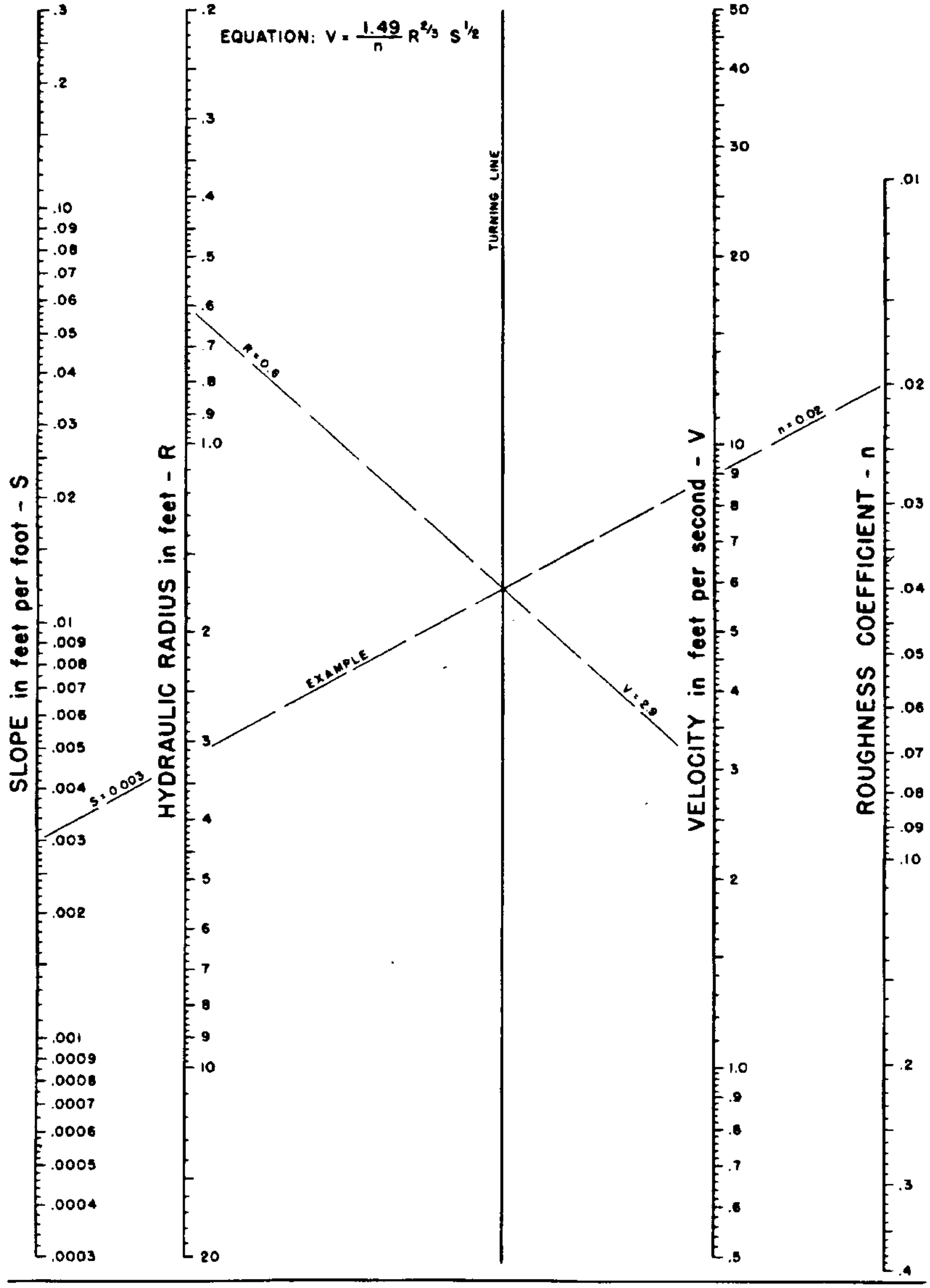
This chart will be found useful when an open-channel flow chart is not available for the particular channel cross section under consideration. Values of n will be found in table 1, and slope S and hydraulic radius $R=A/WP$, where A is the area of cross section and WP is the wetted perimeter, are dimensions of the channel.

Use of the chart is demonstrated by the example shown on the chart itself. Given is a channel with rectangular cross section, 6 feet wide, flowing at a depth of 0.75 foot, with a 0.3-percent slope ($S=0.003$), and $n=0.02$. Area $A=6\times 0.75=4.50$ sq. ft.; wetted perimeter $WP=6+2\times 0.75=7.50$ ft.; then $R=A/WP=4.50/7.50=0.6$.

A straight line is laid on the chart, connecting $S=0.003$ and $n=0.02$. Another straight line is then laid on the chart, connecting $R=0.6$ and the intersection of the first line and the "turning line," and extending to the velocity scale. Reading this scale, $V=2.9$.

The chart may, of course, be used to find any one of the four values represented, given the other three; and may also be used for channels with cross sections other than rectangular.

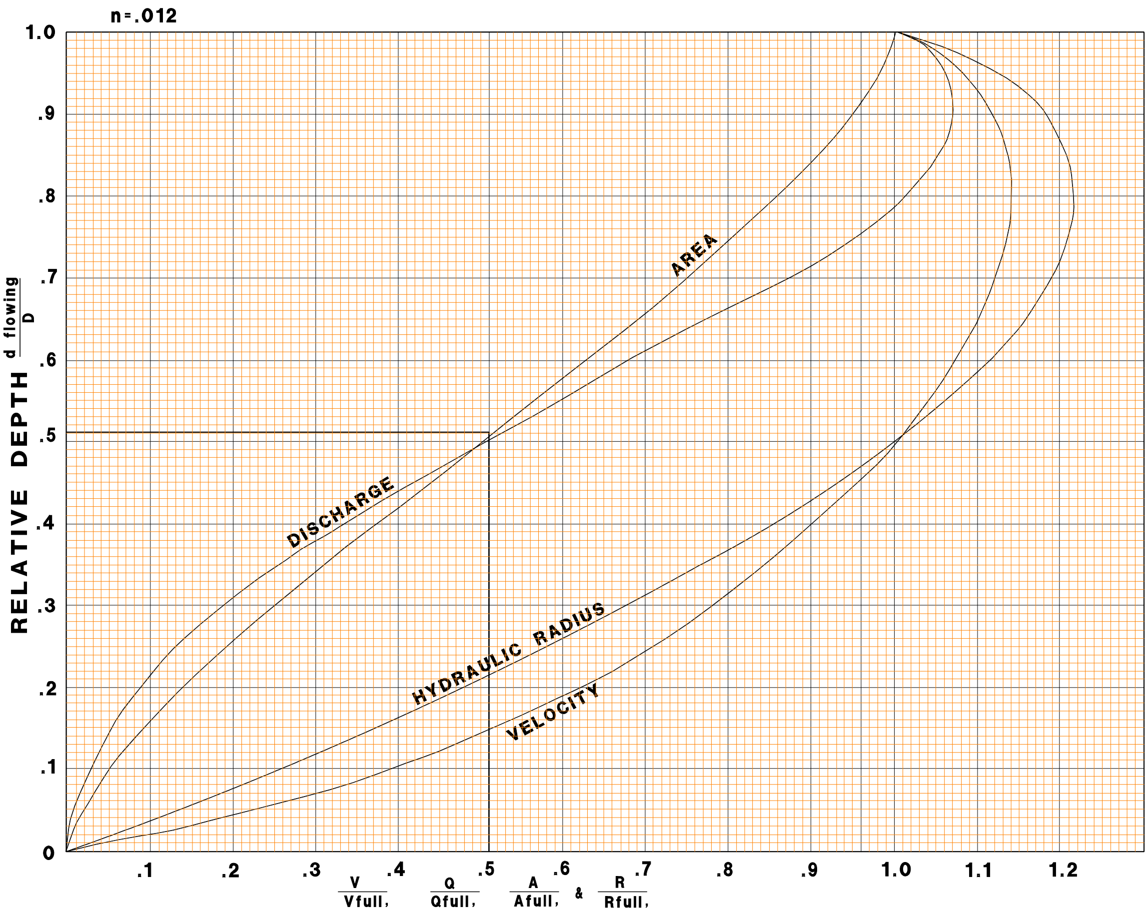
Source: Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow"



NOMOGRAPH FOR SOLUTION
OF MANNING EQUATION

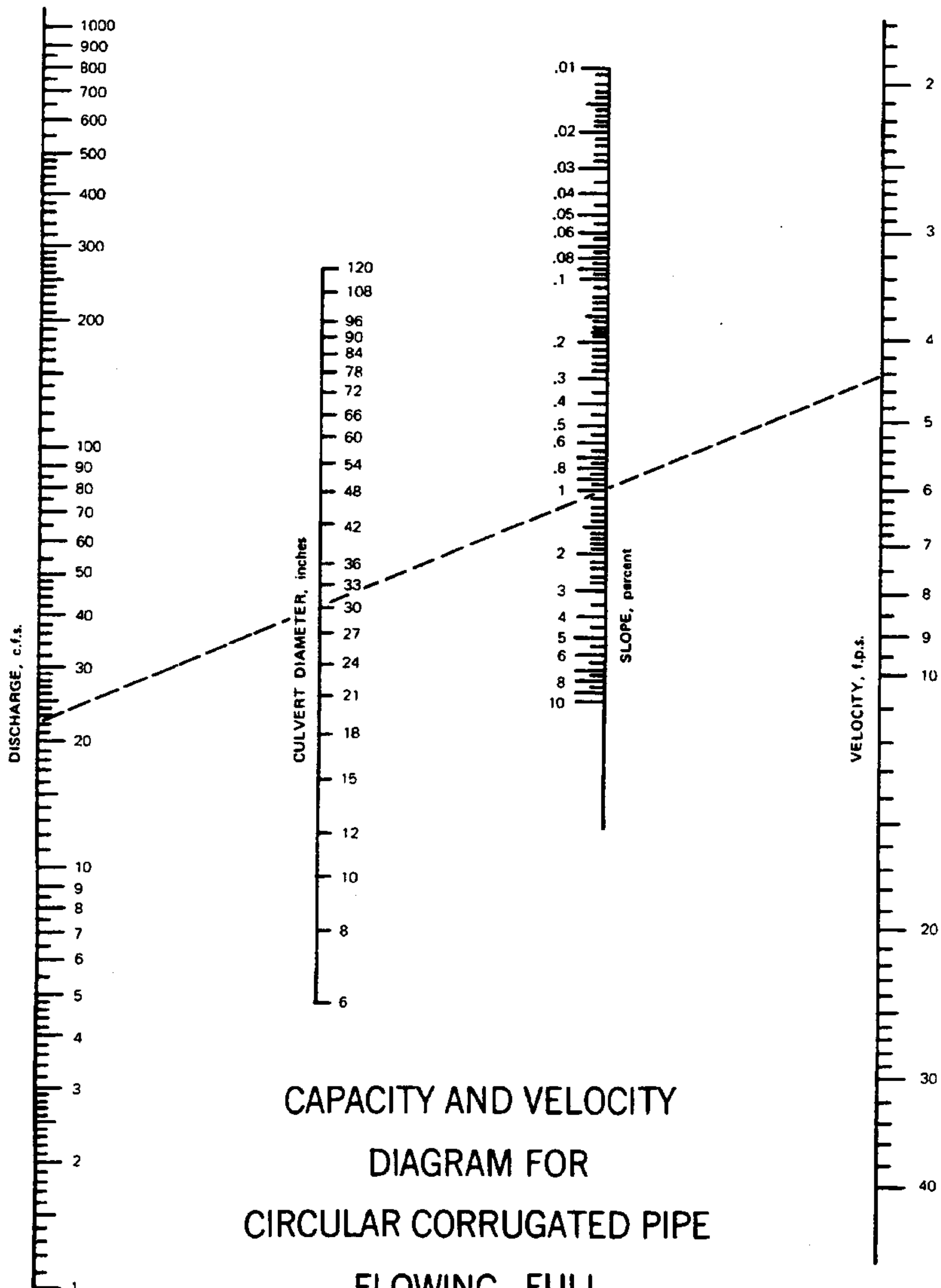
☆ U. S. GOVERNMENT PRINTING OFFICE : 1961 O - 597083

Source: Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow



HYDRAULIC ELEMENTS

CIRCULAR SECTION



CAPACITY AND VELOCITY

DIAGRAM FOR

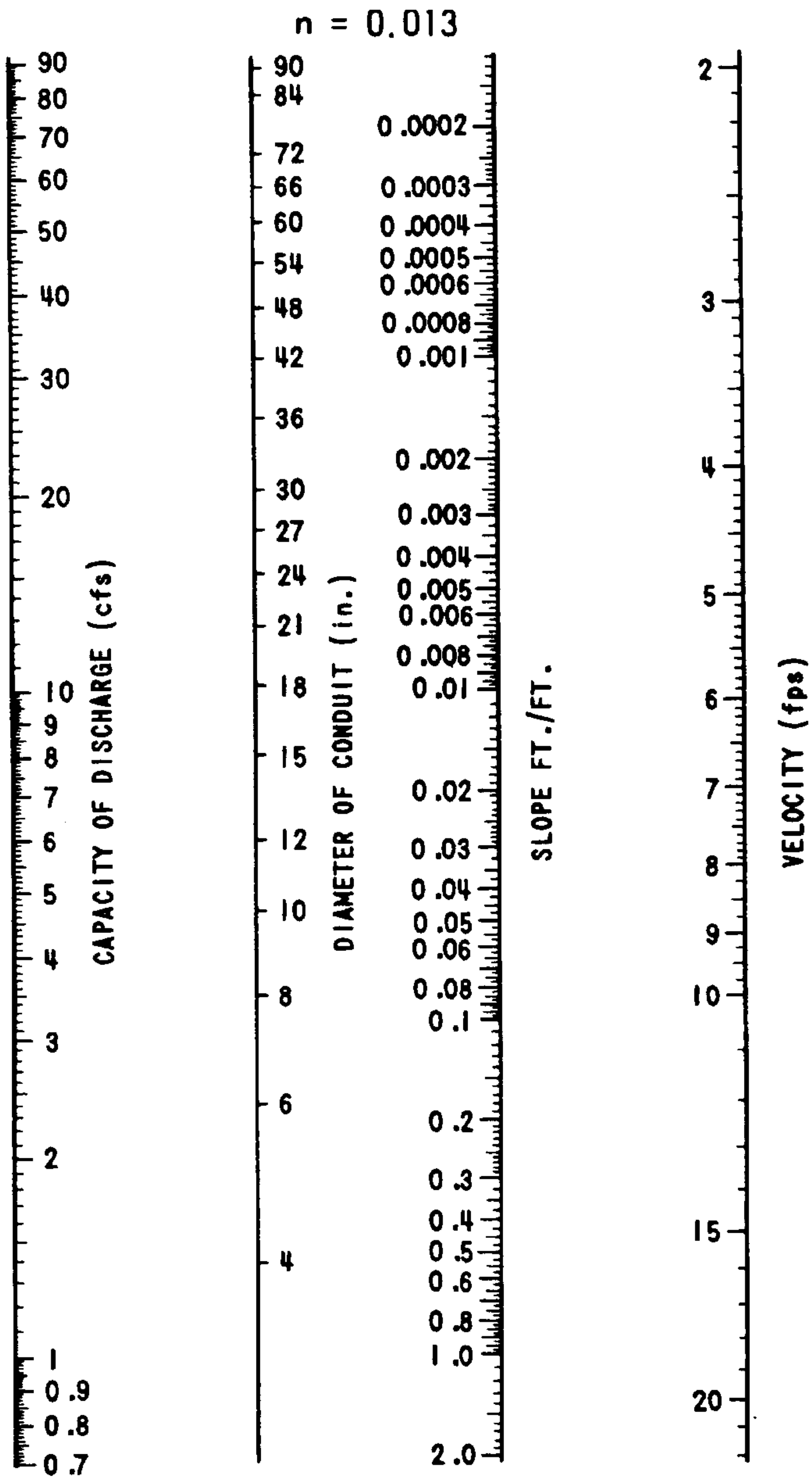
CIRCULAR CORRUGATED PIPE

FLOWING FULL

n = 0.024

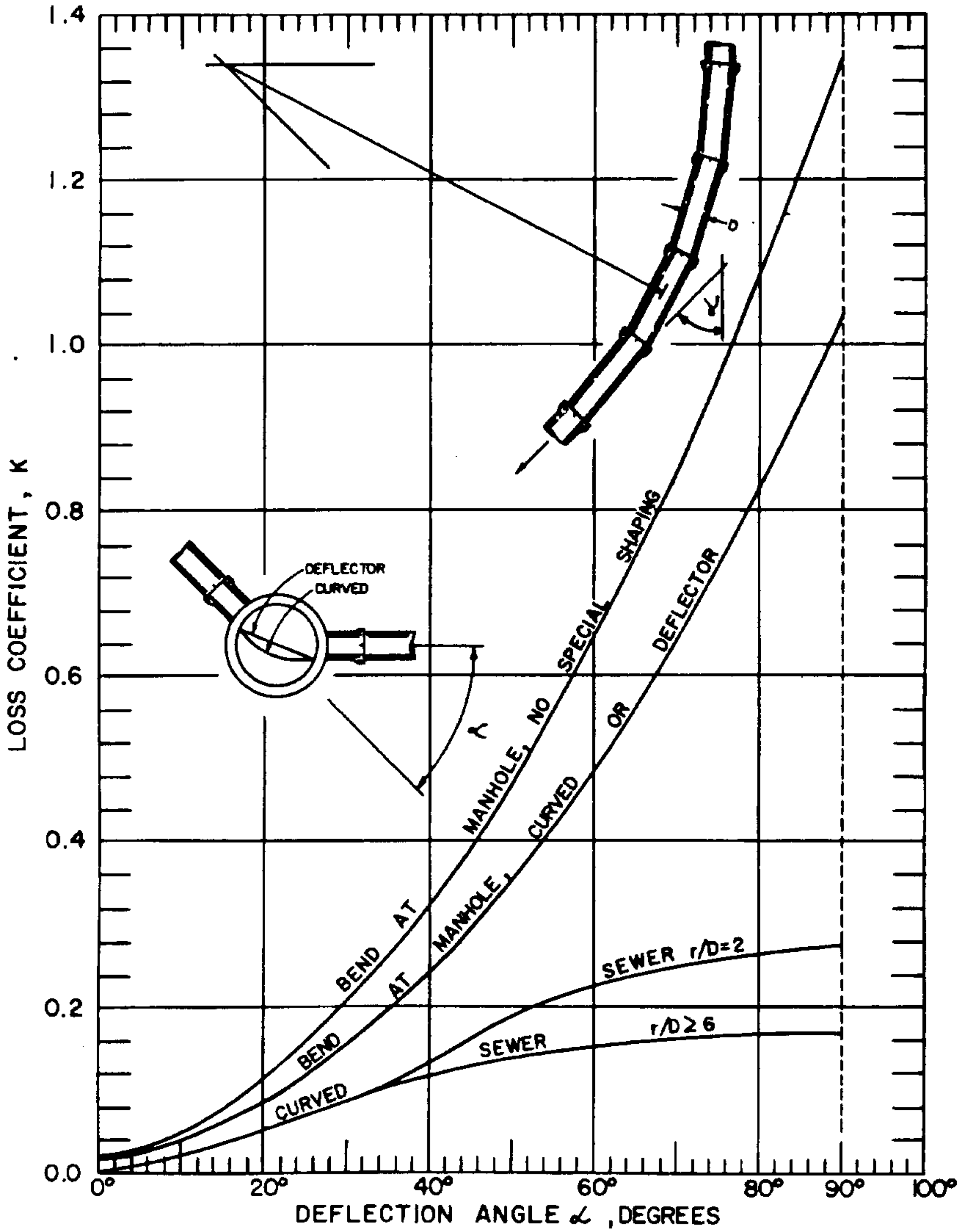
Capacity and Velocity Diagram

For Circular Concrete Pipe Flowing Full

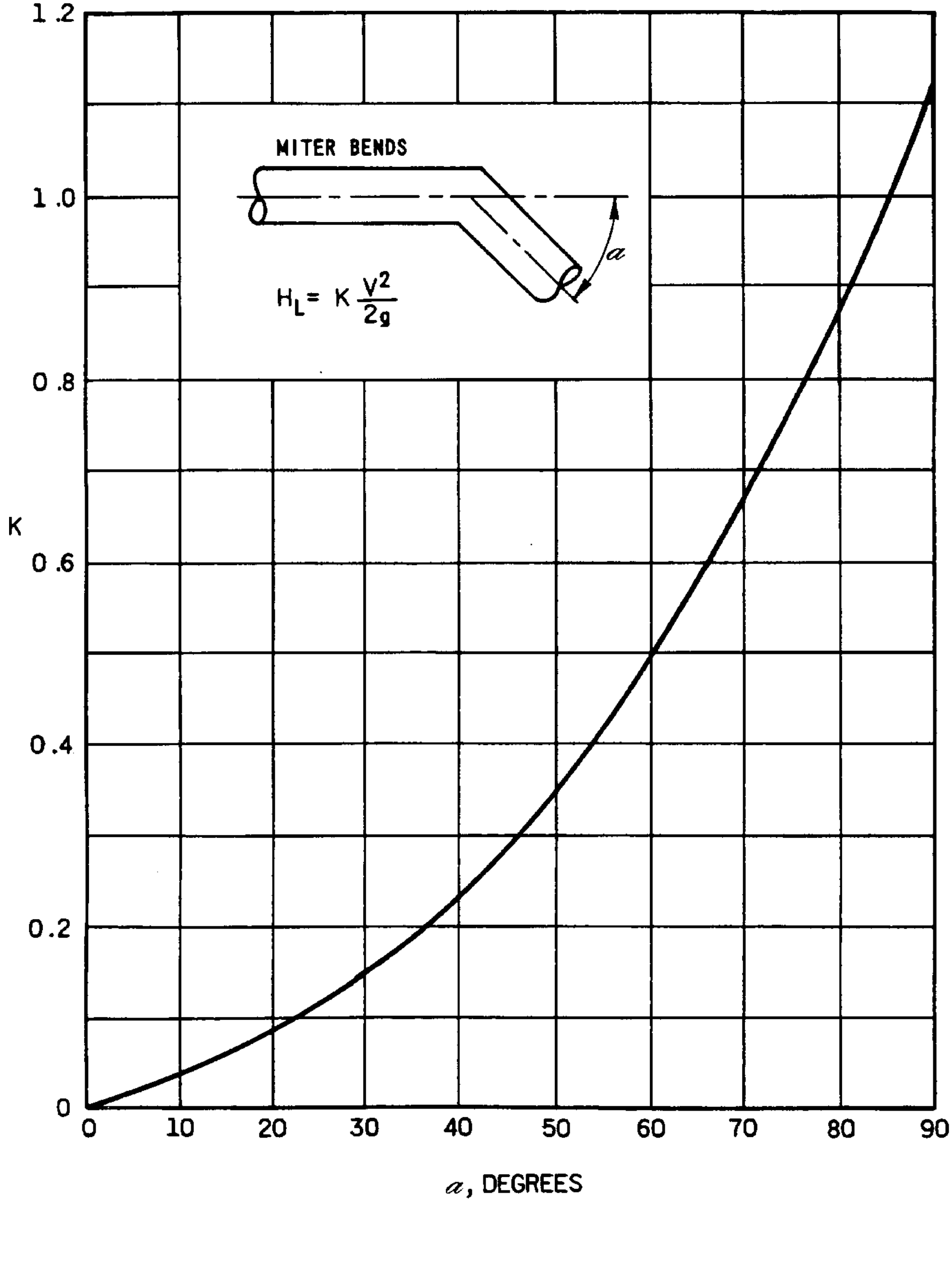


Nomograph based on Manning's formula for circular pipes flowing full in which n=0.013

Sewer Bend Loss Coefficient



Source: Denver Regional Council of Governments, "Urban Storm Drainage"



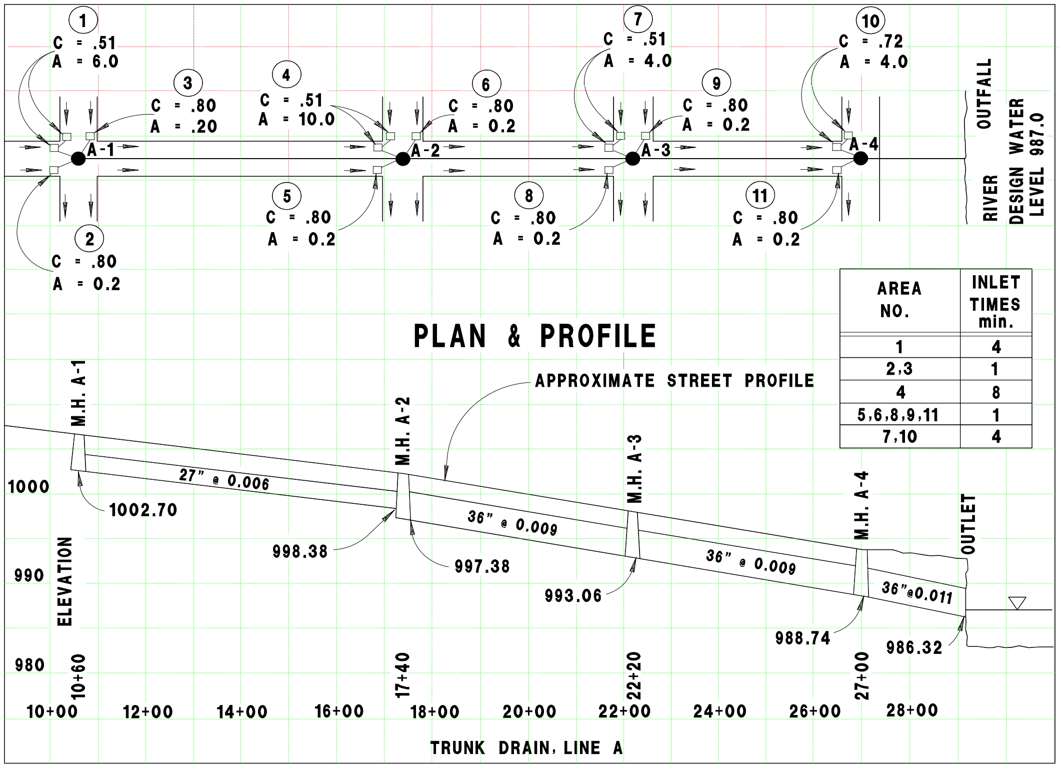
LOSS COEFFICIENTS FOR MITER BENDS

WORK SHEET FOR STORM SEWER DESIGN

PROJECT __________ ROAD __________ COUNTY __________ DESIGN FREQUENCY __________ YR

COMPUTED BY __________ DATE __________ CHECKED BY __________ DATE __________

Location			Tributary Area					Travel Time				Rainfall-Runoff				Flow in Conduit						Vertical Control			
Station of Upstream Structure	Struc- tures		Index No.	Area	Runoff Coeff.	Equlv. Area for 100 Runoff		Inlet Time	Flow Time		Time of Concentration	Ave. Rainfall Intensity	Direct Runoff	Other Runoff	Design Runoff	Slope of Sewer	Pipe Size	Capacity Flowing Full	Mean Velocity Flowing Full	Length of Pipe	Fall of Pipe	Invert Elev.		Top of Structures	
	from	to							Street	Pipe												Upper	Lower	Upper	Lower
				A	C	△CA	ΣCA																		
				Acres		Acres	Acre	Min.	Min.	Min.	Min.	In/Hr	Cfs	Cfs	Cfs	Ft/Ft	In.	Cfs	Fps	Ft.	Ft.	Ft.	Ft.	Ft.	Ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26



Detail A

WORK SHEET FOR STORM SEWER DESIGN

PROJECT 1001-7-00 ROAD OMEGA ROAD COUNTY DANE DESIGN FREQUENCY 10 YR

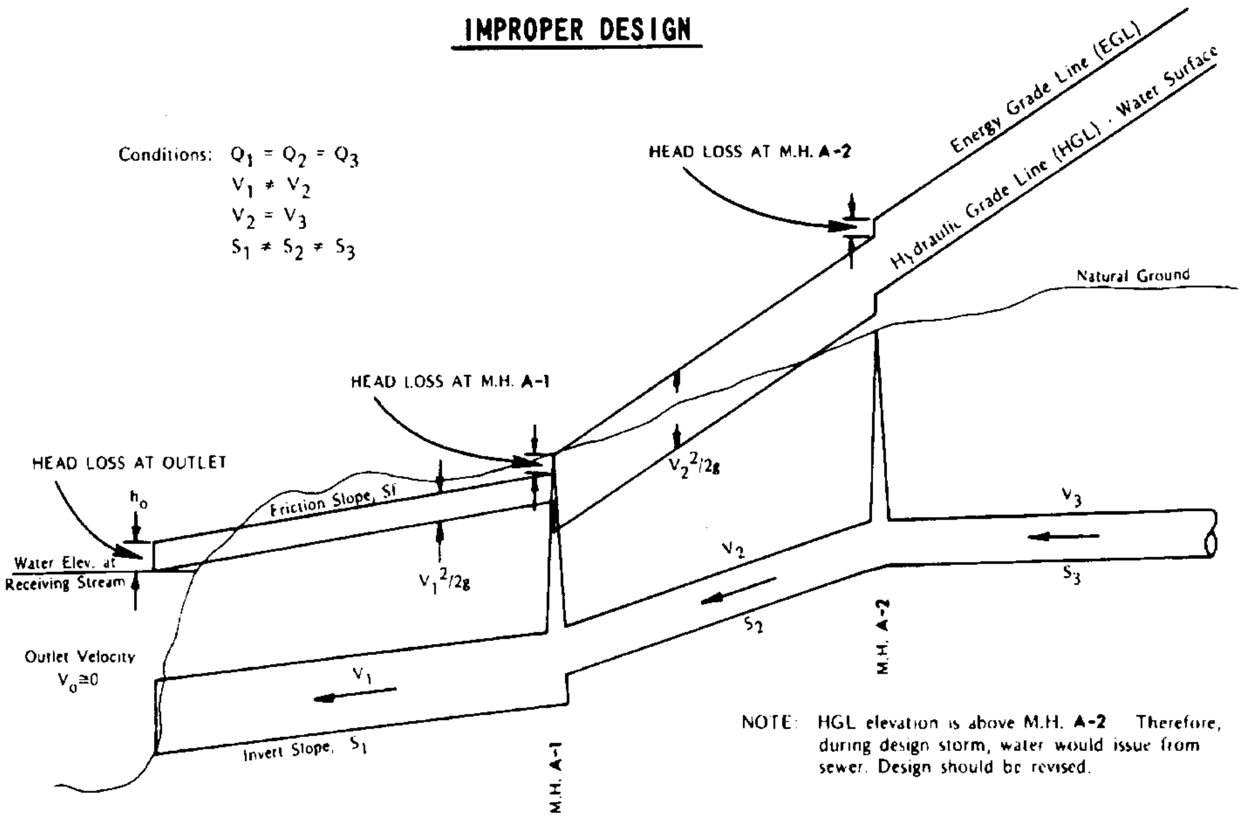
COMPUTED BY ______ DATE 12/24/95 CHECKED BY J.F.K. DATE 12/25/95

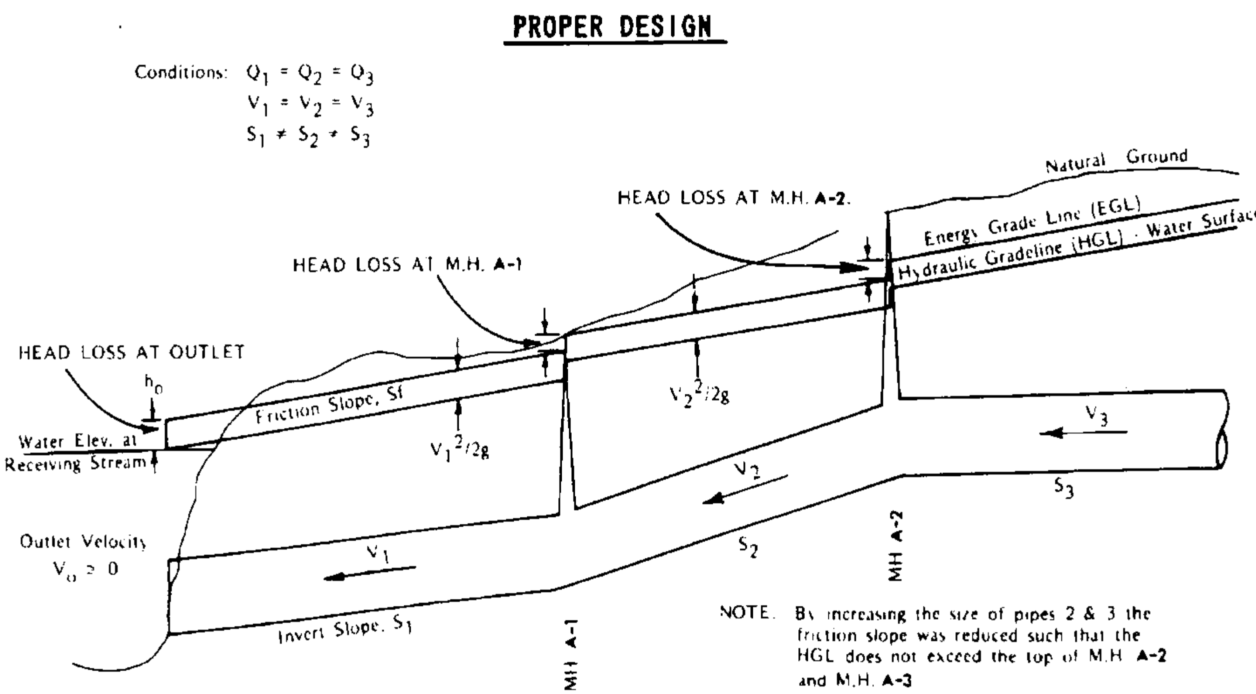
Location			Tributary Area					Travel Time				Rainfall-Runoff				Flow in Conduit						Vertical Control			
Station of Upstream Structure	Structures from	Structures to	Index No.	Area	Runoff Coeff.	Equiv. Area for 100 Runoff		Inlet Time	Flow Time Street	Flow Time Pipe	Time of Concentration	Ave. Rainfall Intensity	Direct Runoff	Other Runoff	Design Runoff	Slope of Sewer	Pipe Size	Capacity Flowing Full	Mean Velocity Flowing Full	Length of Pipe	Fall of Pipe	Invert Elev. Upper	Invert Elev. Lower	Top of Structures Upper	Top of Structures Lower
				A Acres	C	△CA Acres	ΣCA Acre	Min.	Min.	Min.	Min.	In/Hr	Cfs	Cfs	Cfs	Ft/Ft	In.	Cfs	Fps	Ft.	Ft.	Ft.	Ft.	Ft.	Ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
	MH	MH																							
10+60	A-1	A-2	1	6.0	0.51	3.06	3.06	4																	
			2	0.2	0.80	0.16	3.22	1																	
			3	0.2	0.80	0.16	3.38	1	-	1.9	5(1)	6.2	21.	-	21.	.006	27	23	6.1	680	4.32	1002.7	998.38	1006.7	1002.3
	MH	MH																							
17+40	A-2	A-3	4	10.0	0.51	5.10	8.48	8																	
			5	0.2	0.80	0.16	8.64	1																	
			6	0.2	0.80	0.16	8.80	1	-	0.9	8(2)	5.5	48.	-	48.	.009	36	64.0	9.0	480	4.32	997.38	993.06	1002.3	998.20
	MH	MH																							
22+20	A-3	A-4	7	4.0	0.51	2.04	10.84	4																	
			8	0.2	0.80	0.16	11.0	1																	
			9	0.2	0.80	0.16	11.16	1	-	0.9	8.9	5.3	59.	-	59.	.009	36	64.0	9.0	480	4.32	993.06	988.74	998.20	994.00
	MH	out-																							
27+00	A-4	fall	10	4.0	0.72	2.88	14.04	4																	
			11	0.2	0.80	0.16	14.2	1	-	0.8	9.8	5.0	71.	-	71.	.011	36	72.0	10.0	220	2.42	988.20	986.32	994.00	993.00

(1) Minimum design T_c (2) Maximum of (5.0 + 1.9) or 8.0

Detail B

Energy and Hydraulic Grade Lines for a Properly and Improperly Designed Storm Sewer





WORK SHEET FOR STORM SEWER DESIGN

SURCHARGED FLOW

PROJECT____________ ROAD____________ COUNTY____________ DESIGN FREQUENCY ____________YR

COMPUTED BY____________ DATE____________ CHECKED BY____________ DATE____________

Location		Pipe Data			Velocity Head				Pipe Head Losses				Structure Head Losses		Gradeline Elev. at Structure		Vertical Control		
Station of Structure	Structure Type & No.	Discharge	Pipe Size	Pipe Length	Mean Pipe Velocity	Pipe Velocity Head	Mean Channel Velocity	Channel Velocity Head	Coeff. K Bend Loss	Bend Energy Loss	Friction Slope	Friction Head Loss	Coeff. K Structure	Structure Energy Losses	E.G.L. Down-stream / Up-stream	H.G.L. Down-stream / Up-* stream	Invert Elev. Down-stream / Up-stream	Top of Struc. Elev.	Free-board
		Q			V1	$V_1^2/2g$	V_2	$V_2^2/2g$	K_b		S_o		K						
		cfs	in.	ft.	fps	ft.	fps	ft.		ft.	ft/ft	ft.		ft.	ft.	ft.	ft.	ft.	ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

∗ water surface elevation in structure.

SURCHARGED FULL SEWER DESIGN PROBLEM

WORK SHEET FOR STORM SEWER DESIGN

SURCHARGED FLOW

PROJECT 2400-1-00 ROAD ALMA DRIVE COUNTY MILWAUKEE DESIGN FREQUENCY 10 YR

COMPUTED BY D. J. S. DATE 10-16-78 CHECKED BY F. D. S. DATE 10-17-78

Location		Pipe Data			Velocity Head				Pipe Head Losses				Structure Head Losses		Gradeline Elev. at Structure		Vertical Control		
Station of Structure	Structure Type & No.	Discharge	Pipe Size	Pipe Length	Mean Pipe Velocity	Pipe Velocity Head	Mean Channel Velocity	Channel Velocity Head	Coeff. K Bend Loss	Bend Energy Loss	Friction Slope	Friction Head Loss	Coeff. K Structure	Structure Energy Losses	E.G.L. Down-stream Up-stream	H.G.L. Down-stream Up-* stream	Invert Elev. Down-stream Up-stream	Top of Struc. Elev.	Free-board
		Q			V_1	$V_1^2/2g$	V_2	$V_2^2/2g$	K_b		S_o		K						
		cfs	in.	ft.	fps	ft.	fps	ft.		ft.	ft/ft	ft.		ft.	ft.	ft.	ft.	ft.	ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
29+20	River Outlet												1.00	1.55	990.50 992.05	990.50 990.50	986.32 "		
		71	36	220	10.0	1.55	-	-	-	-	.0109	2.40							
27+00	M.H. A-4												0.71	1.10	994.45 995.55	992.90 994.47	988.74 "		
		59	36	480	8.3	1.08	-	-	-	-	.0077	3.70							
22+20	M.H. A-3																		
29+20	River Outlet												1.00	1.55	990.50 992.05	990.50 990.50	986.32 "		
		71	42	220	7.4	0.85	-	-	-	-	.0047	1.30							
27+00	M.H. A-4												0.30	0.26	993.08 993.34	992.23 992.26	988.74 "	994.00	1.74
		59	36	480	8.3	1.08	-	-	-	-	.0077	3.70							
22+20	M.H. A-3												0.71	0.77	997.04 997.81	995.96 997.09	993.06 "	998.20	1.11
		48	36	480	6.8	0.72	-	-	-	-	.0048	2.30							
17+40	M.H. A-2												0.71	0.51	1000.11 1000.62	999.39 999.92	997.38 998.38	1002.3	(3)
		21	24	480	6.7	0.71	-	-	-	-	.0081	3.89							

* water surface elevation in structure.

(1) K Coeff's obtained from Ref. 2, FDM 13-25-35.

(2) Water surface above M H. Cover A-4. Replace downstream pipe with next larger pipe.

(3) Free water surface within conduit. Discontinue calculations or use normal depth of flow within conduit for further calculations.

FDM 13-30-1 Channel Types and Characteristics

October 22, 2012

1.1 Channel Types

Roadside drainage channels perform the vital functions of collecting surface water runoff from the highway and carrying it to natural channels, providing snow storage and filtering sediment from runoff. They should provide the most efficient, stable, and effective disposal system for highway surface runoff, consistent with cost, importance of the road, economy of maintenance, and legal requirements. A standard drainage channel rarely provides the most satisfactory drainage for all sections of the highway, although it is the most efficient for most locations.

1.2 Roadside Ditches

Side ditches are provided in cut sections to remove runoff from cut slopes, pavement, and adjacent areas draining into the highway right-of-way. Side ditches typically are triangular in cross section and dimensioned in accordance with appropriate design standards. The depth may be varied to keep a desirable minimum longitudinal slope of 0.5 percent and to keep the runoff from the design year storm below the top of the highway subgrade (refer to FDM13-10 Attachment 1.1). The minimum depth of a ditch is 1 ft below the subgrade shoulder point to ensure positive drainage of the subgrade. Refer to FDM11-15-1 for cross section elements for rural highways and freeways.

1.3 Median Ditches

The shape, slope, bottom width, and gradient of a median may be varied as required to suit conditions. The following median geometrics are given in Chapter 11:

1. Minimum depth of low point in any depressed median shall be 1.0 foot below subgrade. The desirable median depth should be 1.5 feet. Greater depths are permissible where the median width is more than 60 feet or where dictated by inlet and discharge pipe needs.
2. Side slopes below subgrade should be 6:1 or flatter, with a desirable slope of 10:1 and 20:1 as an absolute minimum.
3. Minimum desirable longitudinal gradient shall be 0.5 percent with 0.3 percent absolute minimum. Where the roadway profile grade is less than the minimum median gradient, the median grade shall be maintained by varying the ditch bottom width, by decreasing the spacing of median drains, and/or by varying the side slopes between 20:1 to 6:1.

Median drains should be spaced so that depth of flow will not rise above the top of the subgrade and so that high erosion-causing velocities will not be reached. In any case, inlet spacing should not exceed 1,000 feet, with 800 feet as desirable maximum. Refer to standard detail drawing for Inlets Type 8, 9, 10 and 11.

1.4 Toe of Slope and Intercepting Embankments

Normally, interceptor ditches are located at the top of a cut slope or along the faces of a cut slope to intercept hillside runoff and prevent the erosion of the cut slope. Consult the region soils engineer to determine whether an interceptor ditch or embankment might cause slope failure. Because the traveling public is not exposed to these ditches safety is not an issue, they may be constructed with 2:1 side slopes, if the slopes are stable and do not erode.

FDM 13-30-5 Channel Characteristics

October 22, 2012

5.1 Introduction

Channels and road ditches are designed for the following channel characteristics: vertical alignment, horizontal alignment, roughness factors, and channel geometry. A general discussion of these channel characteristics is given in this procedure. More specific channel design criteria are covered in FDM 13-30-10.

5.2 Vertical Alignment

The vertical alignment of a new channel should be similar to the profile of the existing channel. Avoid abrupt changes in grade in the new channel. If abrupt changes in grade are designed into the channel, one of the following two effects will occur, depending upon the slope change:

1. Deposition of transported material will occur where the grade changes from steep to flat.
2. Scouring will occur when the grade changes from flat to steep.

Due to topographic features, it is normally impossible to design a channel without areas where deposition of material and/or scouring will occur. Therefore, other means of preventing these conditions from occurring must be part of the final design. Open channel scour may be reduced or prevented by using turf reinforcement mat, riprap or for extreme conditions, grouted riprap, articulated concrete block, drop structures or channel paving. In addition, in areas where vehicles are not likely to travel, vertical grade drops may be used to maintain flat ditch grades that are non-erodible.

5.3 Horizontal Alignment

Horizontal alignments for roadside drainage ditches typically follow the road alignment. Outside of the roadside alignment, the designer should construct meandering channels to reproduce preconstruction condition to imitate nature and to avoid steepening gradients. Changes to alignment should be gradual to minimize erosion. When it is necessary to construct curves that are not erosion resistant, designers should use riprap or other methods to control erosion.

5.4 Roughness Factors

The capacity of a drainage channel depends upon its shape, size, slope, and roughness. A specific channel's capacity will decrease as the roughness factor increases. Erosion potential of a channel on a steep grade may be reduced by increasing the channel roughness which decreases the flow velocity. Conversely, a specific channel's capacity will increase as its roughness decreases. Therefore, if no other options are available, the capacity of channels with flat slopes may be increased by constructing smooth channel walls and bottom that will maintain a higher velocity.

A table of roughness coefficients (n) for Manning's Equation can be found in FDM 13-25-35. With the use of this table and engineering judgment, the approximate roughness coefficients of existing channels should be determined through a field review. Any erosion or deposition of material in the existing channel(s) should be noted and recorded. These facts can be used in the design stage to compare the compatibility of new channels with the existing channels. FDM 13-30-15 and FDM 13-30-25 show the designer how to develop the Manning’s n value for a grass or riprap lined channel that is based upon the channel characteristics, lining, and flow rates.

5.5 Channel Geometry

Channels are usually constructed with a triangular or trapezoidal shape as shown in Figure 1 below. Triangular roadside ditches are typically used for WisDOT projects. Trapezoidal roadside ditches can be used when additional capacity is needed or if velocities need to be reduced. In time, triangular and trapezoidal channels naturally tend to become parabolic in shape because of siltation.

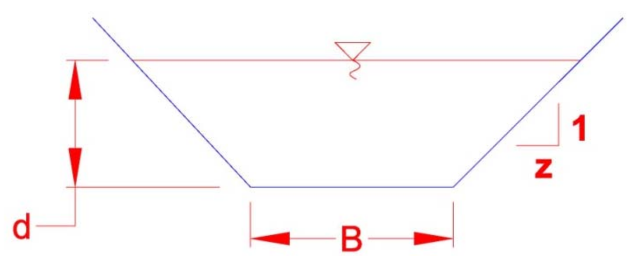


Figure 1.1 Trapezoidal Channel Cross Section

Where:

B = channel base (ft)

d = depth of flow (ft)

Z = side slope, Z horizontal:1 vertical

5.6 Natural Channels

For natural channel reconstruction, the designer should construct meandering channels to reproduce preconstruction conditions, to imitate nature, and to avoid steepening the channel gradients. Any change in alignment should be gradual, to minimize erosion. When it is necessary to construct curves that are not erosion resistant, the erosion may be controlled by using side slope protection, riprap and/or channel paving.

Sometimes it is necessary to connect into or change the alignment of natural channels. Changes to a natural channel should only be considered as a last resort when safety, economic, hydraulic and/or environmental issues warrant it.

A channel change is any alteration in the cross section, slope, alignment, and/or hydraulic capacity of a natural watercourse. Any straightening of a natural channel invariably results in a steeper channel slope and higher velocities, resulting in channel erosion. Channel changes should be designed to duplicate the natural channel's hydraulic conditions. When it is necessary to relocate a meandering channel, the relocated channel should include similar meanders to reproduce existing natural conditions. The stream thread, or channel length, should be maintained.

Regulatory agencies generally discourage alignment changes to natural channels. Typically, significant coordination will be necessary with the WDNR and US Army Corps of Engineers for any proposed changes. It is important that possible channel changes to natural channels be identified early in the design process and brought to the attention of the regulatory agencies for their input and concurrence.

FDM 13-30-10 Hydraulic Design of Open Channels *October 22, 2012*

10.1 Introduction

This procedure presents the theory, design criteria, basic equations, and design methods to hydraulically design open channels. The designer must be knowledgeable about the types of flow, channel design characteristics, and basic hydraulic equations. For additional information, see Hydraulic Design Series No. 4, Introduction to Highway Hydraulics (HDS 4).

With this background information, stable channels can be designed using rigid or flexible lining criteria and design techniques discussed in this procedure and the following three procedures. The design techniques along with example problems presented in this procedure are:

1. General flexible lining design procedure for grass and rock riprap lined channels.
2. Steep side slope protection.
3. Bend protection design.

10.2 Types of Flow

There are several classifications of the flow state in open channels that help determine the appropriate method of analysis for a given design problem.

- *Steady Flow and Unsteady Flow*

  Time is the criterion that distinguishes these two types of flows. The flow at any cross section is said to be steady if the velocity does not vary in magnitude or direction with time. Conversely, the flow is unsteady if the velocity varies with time. In most open channel flow problems, flow conditions are assumed to be steady.

- *Uniform Flow and Varied Flow*

  Space is the criterion that distinguishes these two types of flows. Flow is said to be uniform when the depth of flow and the mean velocities are the same at successive cross sections in any reach. Uniform flow only occurs in a channel with a constant cross section. Varied flow occurs in reaches with uniform or varying cross sections that are affected by other controls or its own changing shape and cause accelerated or decelerated flow conditions. Although steady, uniform flow rarely occurs in natural streams or constructed channels, it provides a check point for open channel design problems

- *Laminar Flow and Turbulent Flow*

  In laminar flow water particles move along well-defined paths, or streamlines. In turbulent flow water particles move in irregular paths that are neither smooth nor fixed; but as an average they represent the forward motion of the entire stream. In general, all practical open channel flow problems exhibit turbulent flow.

- *Critical Flow*

  Critical flow occurs in an open channel when the specific energy (sum of depth and velocity head) is a minimum for a particular discharge, which is also when the Froude number, Fr =1. As flow approaches the critical depth, or minimum specific energy, small changes in energy may cause unstable and excessive water surface undulations. Therefore, when designing channels, avoid the region of instability defined within the range $0.9 \le Fr \le 1.1$.

  The Froude number is defined as:

$Fr = V/(gD)^{0.5}$

Where:

V = average velocity (ft/s)

g = gravitational acceleration, 32.2 ft/s^2

D = hydraulic depth, the area divided by the channel top width (ft)

The channel slope at critical flow conditions is the critical slope. When the channel slope is flatter than the critical slope (mild slope), the flow is subcritical and depth of flow $d > d_c$ = critical depth. Conversely, when the channel slope is steeper than the critical slope (steep slope), the flow is supercritical and $d < d_c$. If the unstable flow region cannot be avoided in design, assume the least favorable type of flow (supercritical or subcritical) for the design and provide sufficient freeboard in the channel so that flow instability will not affect channel stability. For a more detailed discussion of critical flow, see Chow, 1959, Chapters 3 and 4 or HDS 4.

10.3 Uniform Flow

The flow depth or other condition of uniform flow in a channel at a known discharge is computed using the Manning's equation (equation (1)), along with prescribed conditions (e.g., channel shape, material, slope). In uniform flow, $S_f = S_w = S_0$, where:

S_f = (ft/ft) friction slope or energy gradient, which is the elevation of the total head of flow.

S_o = (ft/ft) slope of the channel bed.

S_w = (ft/ft) slope of the water surface.

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2} \qquad (1)$$

Where:

Q = discharge, (ft^3/s)

n = Manning's roughness coefficient, dimensionless

A = cross-sectional area, (ft^2)

R = A/P = hydraulic radius (ft)

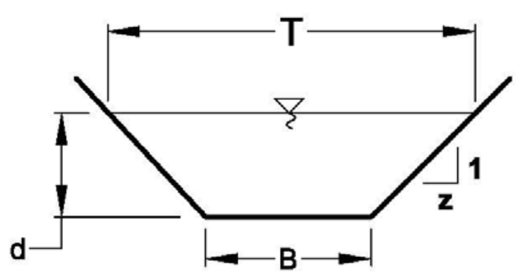


Figure 10.1 ***Trapezoidal Channel Cross Section***

Manning's equation for flow in the trapezoidal channel (Figure 1) is

$$Q = \frac{1.49}{n}(Bd + Zd^2)\left(\frac{Bd + Zd^2}{B + 2d(1 + Z^2)^{1/2}}\right)^{2/3} S_0^{1/2} = \frac{1.49}{n}\left(\frac{[(B + Zd)d]^{5/3}}{[B + 2d(1 + Z^2)^{1/2}]^{2/3}}\right) S_0^{1/2}$$

Where:

B = channel base (ft)

d = depth of flow (ft)

Z = side slope, Z horizontal:1 vertical

A = Bd + Zd^2

10.4 Manning's Roughness Coefficient

The Manning's roughness coefficient is usually considered constant, but often is not constant in shallow flow depth conditions. For a riprap lining, the flow depth in small channels may be only a few times greater than the diameter of the mean riprap size. In this case, use of a constant n value is not acceptable and consideration of the shallow flow depth should be made by using a higher n value. Similarly, the flow depth in grass channels may also be small relative to the height of the grass, so n will also vary with depth. Additional roughness guidance for both types of linings is available in FDM 13-30-15 and FDM 13-30-25, where Manning's n is varied depending upon the depth of flow.

10.5 Shear Stress

10.5.1 Equilibrium Concepts

Most highway drainage channels cannot tolerate bank instability and possible lateral migration. Stable channel design concepts focus on evaluating and defining a channel configuration that will perform within acceptable limits of stability. Methods for evaluation and definition of a stable configuration depend on whether the channel boundaries can be viewed as:

- essentially rigid (static)
- movable (dynamic).

In the first case, stability is achieved when the material forming the channel boundary effectively resists the erosive forces of the flow. Under such conditions the channel bed and banks are in static equilibrium, remaining basically unchanged during all stages of flow.

Because of the need for reliability, static equilibrium conditions and the use of linings to achieve a stable condition are usually preferable to using dynamic equilibrium concepts. Two methods have been developed and are commonly applied to determine if a channel is stable in the sense that the boundaries are basically immobile (static equilibrium):

- the permissible velocity approach
- the permissible tractive force (shear stress) approach.

This design procedure uses the permissible tractive force approach rather than the velocity approach, since two channels may have the same velocity with one stable and the other unstable due to different combinations of depth and slope. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary. By Chow's definition, permissible tractive force is the maximum unit tractive force that will not cause serious erosion of channel bed material from a level channel bed (Chow, 1959 or HDS 4).

10.5.2 Applied Shear Stress

The hydrodynamic force on the channel boundary by water flowing in the channel is the tractive force. The basis for stable channel design with flexible lining materials is that flow-induced tractive force should not exceed the permissible or critical shear stress of the lining materials. In uniform flow, the tractive force equals the effective component of the drag force exerted on the boundary by the water, parallel to the channel bottom (Chow, 1959). The mean boundary shear stress applied to the wetted perimeter for uniform flow is given by the following equation (2).

$$\tau_o = \gamma R S_o \qquad (2)$$

Where:

τ_o = mean boundary shear stress, (lb/ft^2)

γ = unit weight of water, 62.4 lb/ft^3

R = hydraulic radius (ft)

Shear stress in channels is not uniformly distributed along the wetted perimeter (USBR, 1951; Olsen and Florey, 1952; Chow, 1959; Anderson, et al., 1970). A typical distribution of shear stress in a prismatic channel is shown in Figure 10.2. The shear stress is zero at the water surface and reaches a maximum on the centerline of the channel. The maximum for the side slopes occurs at about the lower third of the side.

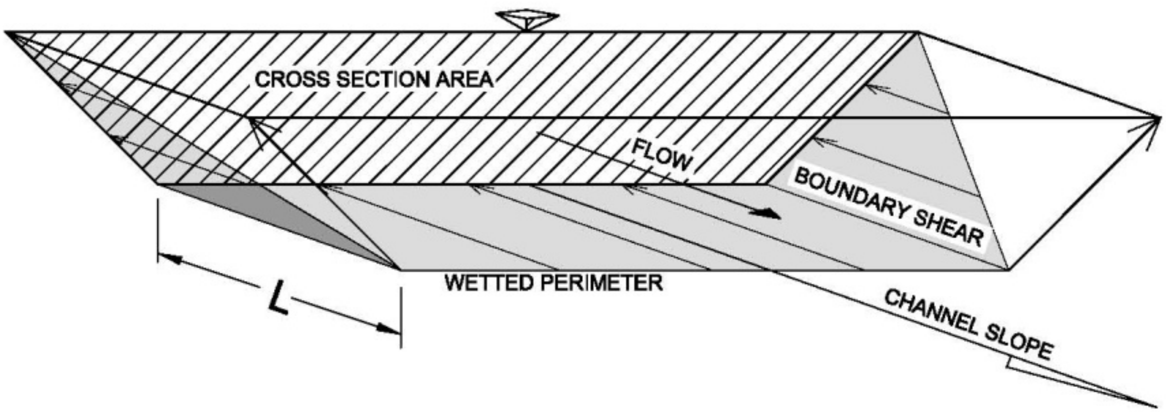


***Figure 10.2* Typical Distribution of Shear Stress**

The maximum shear stress on a channel bottom, τ_d, and on the channel side, τ_s, in a straight channel depends on the channel shape. To simplify the design process, the maximum channel bottom shear stress is taken as:

$$\tau_d = \gamma d S_o \qquad (3)$$

Where:

τ_d = shear stress in channel at maximum depth (lb/ft^2)

d = maximum depth of flow in the channel for the design discharge (ft)

For trapezoidal channels where the ratio of bottom width to flow depth (B/d) is greater than 4, Equation 3 provides an appropriate design value for shear stress on a channel bottom. Most roadside channels are characterized by this relatively shallow flow compared to channel width. For trapezoidal channels with a B/d ratio less than 4, Equation 3 is conservative. For example, for a B/d ratio of 3, Equation 3 overestimates actual bottom shear stress by 3 to 5 percent for side slopes (Z) of 6 to 1.5, respectively. For a B/d ratio of 1, Equation 3 overestimates actual bottom shear stress by 24 to 35 percent for the same side slopes of 6 to 1.5, respectively. In general, Equation 3 overestimates in cases of relatively narrow channels with steep side slopes. In addition, the methods used to analyze riprap stability described in FDM 13-30-25 account for both side slope and bottom slope stability. For more information, see HEC-15, Chapter 2.

10.5.3 Permissible Shear Stress

Flexible linings act to reduce the shear stress on the underlying soil surface. For example, a long-term lining of vegetation in good condition can reduce the shear stress on the soil surface by over 90 percent. Transitional linings act in a similar manner as vegetative linings to reduce shear stress. Performance of these products depends on their properties: thickness, cover density, and stiffness. The erodibility of the underlying soil therefore is a key factor in the performance of flexible linings. The erodibility of soils is a function of particle size, cohesive strength and soil density. The erodibility of non-cohesive soils (defined as soils with a plasticity index of less than 10) is due mainly to particle size, while fine-grained cohesive soils are controlled mainly by cohesive strength and soil density. For most highway construction, the density of the roadway embankment is controlled by compaction rather than the natural density of the undisturbed ground. However, when the ditch is lined with topsoil, the placed density of the topsoil should be used instead of the density of the compacted embankment soil. The in-place topsoil, however, should not be overly compacted to promote vegetation establishment.

For rock linings, the permissible shear stress, τ_p, indicates the force required to initiate movement of the particles. Prior to the movement of rocks, the underlying soil is relatively protected. Therefore permissible shear stress is not significantly affected by the erodibility of the underlying soil. However, if the lining moves, the underlying soil will be exposed to the erosive force of the flow.

Permissible shear stress values for different soil types are based on the methods described in this procedure. Vegetative lining performance relates to how well the lining protects the underlying soil from shear stresses. These linings have permissible shear stresses dependent of soil types.

10.6 Design Parameters

10.6.1 Design Flow Rates

FDM 13-10 Figure 1.1, lists the design flow rate for roadside and median ditches, for flow capacity, that flow with a 25-year recurrence interval. To evaluate the erosion potential of a ditch, use a flow rate with a 10-year recurrence interval. Note that all flows through the ditch, both from the highway right-of-way and from other adjacent properties, outfalls, ditches or channels, must be included in the design flow.

If the permanent lining will be a vegetative lining with a temporary lining during the establishment period, use the 2-year recurrence interval for the temporary lining design. The temporary lining is only required for a short period of time, and if the lining is damaged, repairs are usually inexpensive. Refer to FDM 10-5-30 for a discussion on designing temporary linings.

10.6.2 Channel Cross Section Geometry

Most highway drainage channels are triangular or trapezoidal in shape with rounded corners. For design purposes, a triangular or trapezoidal representation is sufficient. Design of roadside channels should be integrated with the highway geometric and pavement design to ensure proper consideration of safety and pavement drainage needs. If available channel linings are found to be inadequate for the selected channel geometry, it may be feasible to increase the channel area by either increasing the bottom width or flattening the side slopes. Widening the channel will reduce the flow depth and lower the shear stress on the channel perimeter. Limit the ratio of top width to depth to less than 20 (Richardson, Simons and Julien, 1990) because very wide channels have a tendency to form smaller more efficient channels within their banks. This will increase the shear stress above the planned design range, which should be avoided.

It has been demonstrated that if a riprap-lined channel has 3:1 (H:V) or flatter side slopes, there is no need to check the banks for erosion (Anderson, et al., 1970). With side slopes steeper than 3:1, a combination of shear stress against the bank and the weight of the lining may cause erosion on the banks before the channel bottom is disturbed. The design method includes procedures for checking the potential of channel side slopes to erode.

10.6.3 Channel Slopes and Alignment

The longitudinal slope of a roadside channel is usually the same as the roadway profile and so is typically not a design variable option. If channel stability conditions are below the required performance and available linings are nearly sufficient, it may be feasible to reduce the channel slope slightly relative to the roadway profile. For channels outside the roadway right-of-way, there can be more grading design options to adjust channel slope where necessary. Channel slope is one of the major parameters in determining shear stress.

Note that the increased shear stresses created by flow around a bend may produce scour that would not occur in straight channel reaches. To prevent bend scour, it may be necessary to increase the rock riprap size or use a different lining material in the bend. Refer to the section on bend stability for more guidance.

10.6.4 Freeboard

The freeboard of a channel is the vertical distance from the water surface to the top of the channel at the design condition. The importance of this factor depends on the consequence of overflow of the channel bank. At a minimum, the freeboard should be sufficient to prevent water surface waves or fluctuations from washing over the sides of the channel. In a permanent roadway channel 0.5 ft of freeboard above the subgrade shoulder point is the minimum freeboard height, as shown in Figure 10.3. The design channel depth should not be above the subgrade shoulder point. The minimum depth of a ditch is 1 ft below the subgrade shoulder point to ensure positive drainage of the subgrade. Steep gradient channels (supercritical flow) should have a freeboard height equal to the flow depth, which allows for large variations to occur in flow depth caused by waves, jumps, splashing, and surging. Lining materials should extend to the freeboard elevation. Freeboard requirements are shown in Table 10.1.

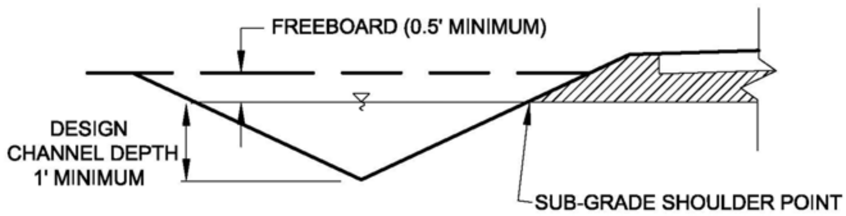


Figure 10.3 Roadside Ditch Freeboard

Table 10.1 Freeboard Requirements

Froude Number Range	Freeboard Requirement
Fr < 0.9	0.5 ft.
0.9 <= Fr <= 1.1	1.0 ft.
Fr > 1.1	Depth of flow

10.6.5 Rigid and Flexible Linings

The design of rigid linings, such as concrete channels, can be accomplished using Manning's formula to determine the required dimension of several channel shapes and chose the one that minimizes cost, maximizes constructability, and meets other roadway specifications. Since there is no erosion, there is no maximum permissible velocity. Table 10.2 outlines the advantages and disadvantages of rigid linings.

Table 10.2 Advantages and Disadvantages of Rigid Linings

Advantages	Disadvantages
Large capacity	Expensive to construct & maintain
Prevent erosion in steep channels	Unnatural appearance
May be constructed within a limited right of way	Prevent or reduce natural infiltration
Underlaying soil completely protected	Scour at downstream end
	Linings may be destroyed by undercutting, channel head cutting or hydrostatic pressure

The flexible linings covered in this discussion are temporary, vegetative, and rock riprap linings. Table 10.3 outline the advantages and disadvantages of flexible linings.

Table 10.3 Advantages and Disadvantages of Flexible Linings

Advantages	Disadvantages
Cheaper than rigid	Limited flow depth because of erosion
Self-healing	Lower capacity
Permit infiltration and exfiltration	Requires more right-of-way than rigid linings
Natural appearance	Riprap may be unavailable
Provide a filtering media for runoff contaminants	May not be able to establish vegetation
Lower velocity	

10.6.6 Composite Linings

Composite linings use two lining types in a single channel rather than one. A more shear resistant lining is used in the bottom of the channel while a less shear resistant lining protects the sides. This type of design may be desirable where the upper lining is more cost-effective and/or environmentally benign, but the lower lining is needed to resist bottom stresses.

Another important use of a composite lining is in vegetative channels that experience frequent low flows. These low flows may kill the submerged vegetation. In erodible soils, this leads to the formation of a small gully at the bottom of the channel. Gullies weaken a vegetative lining during higher flows, causing additional erosion, and can result in a safety hazard. A solution is to provide a non-vegetative low-flow channel lining such as riprap. The dimensions of the low-flow channel are sufficient to carry frequent low flows but only a small portion of the design flow. The remainder of the channel is covered with vegetation.

10.7 General Design Procedures

This section outlines the general procedure for designing flexible channel linings based on the shear stress concepts presented earlier in this section. The simplest case of the straight channel is described first, followed by a discussion of variations to the straight channel including side slope stability, bends, and composite linings. Discussions of specific design approaches for grass channels and riprap channels follow this general procedure.

10.7.1 Straight Channels

The basic design procedure for flexible channel linings is quite simple. The computations include a determination of the uniform flow depth in the channel and determination of the shear stress on the channel bottom at this depth. Both concepts were discussed earlier in this section. Recall that:

$$\tau_d = \gamma d S_o$$

The basic comparison required in the design procedure is that of permissible to computed shear stress for a lining. If the permissible shear stress is greater than or equal to the computed shear stress, including consideration of a safety factor, the lining is considered acceptable. If a lining is unacceptable, a lining with a higher permissible shear stress is selected, the discharge is reduced (by diversion or retention/detention), or the channel geometry is modified. This concept is expressed as:

$$\tau_p \geq SF\,\tau_d \qquad (4)$$

Where:

τ_p = permissible shear stress for the channel lining, (lb/ft^2)

The safety factor provides for a measure of uncertainty, as well as a means for the designer to reflect a lower tolerance for failure by choosing a higher safety factor. A safety factor of 1.0 is appropriate in many cases and may be considered the default. The expression for shear stress at maximum depth (Equation 3) is conservative and appropriate for design as discussed above. However, safety factors from 1.0 to 1.5 may be appropriate, subject to the designer’s discretion, where one or more of the following conditions may exist:

- critical or supercritical flows are expected
- soil types where vegetation may be uneven or slow to establish

- significant uncertainty regarding the design discharge
- consequences of failure are high

The basic procedure for flexible lining design consists of the following steps and is summarized in Figure 10.4.

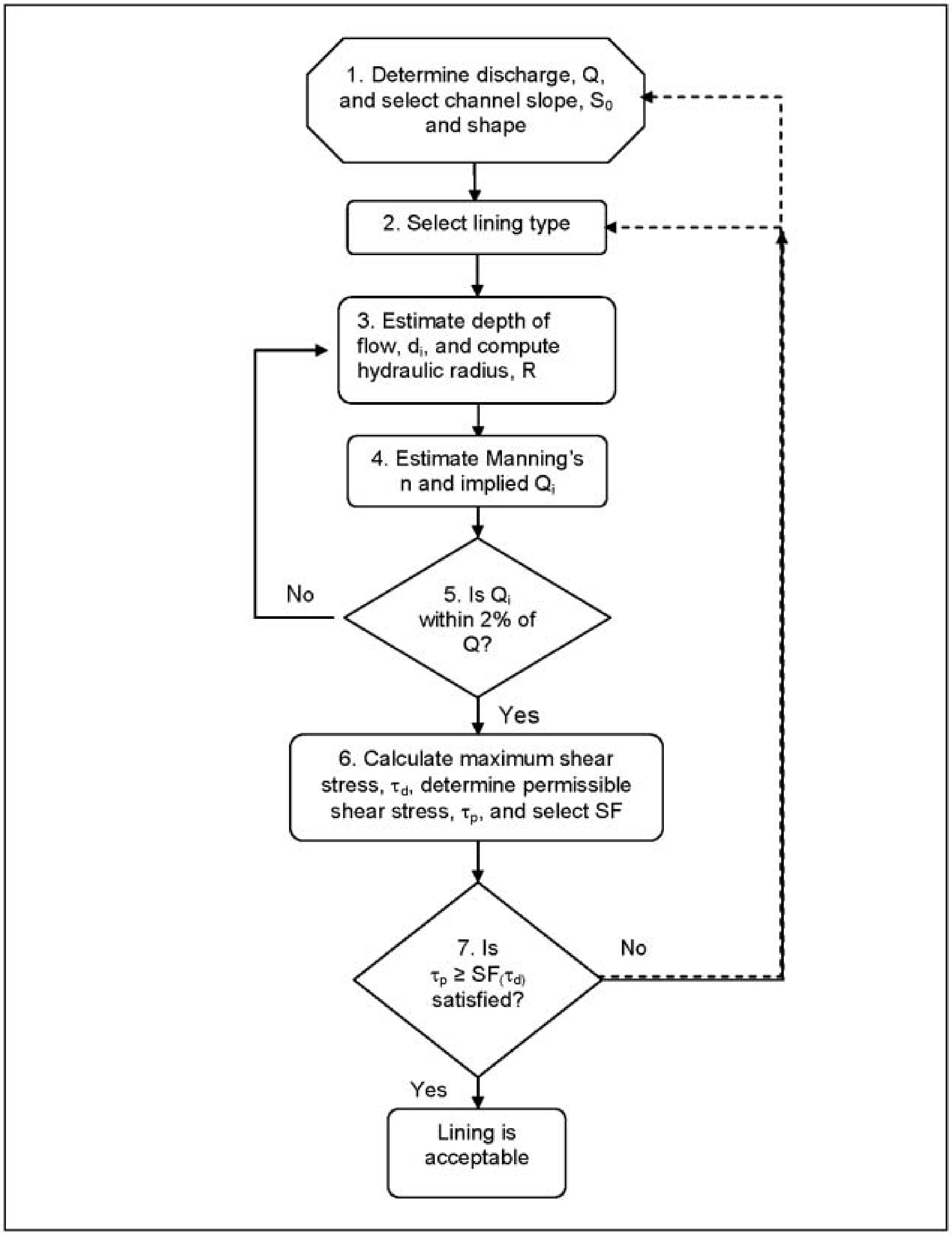


Figure 10.4 Flexible Channel Lining Design Flow Chart

Step 1. Determine a design discharge (Q) and select the channel slope (S_o) and shape.

Step 2. Select a trial lining type. Initially, determine if a long term lining is needed, and whether or not a temporary or transitional lining is required. For the latter, the trial lining type could be chosen as an erosion mat. For example, it may be determined that the bare soil is insufficient for a long-term solution, but vegetation is a good solution. For the transitional period between construction and vegetative establishment, analysis of the bare soil with an erosion control mat will determine if a temporary lining is prudent.

Step 3. Estimate the depth of flow, d_i in the channel and compute the hydraulic radius, R. The estimated depth may be based on physical limits of the channel, but this first estimate is essentially a guess. Iterations on steps 3 through 5 may be required.

Step 4. Estimate Manning's n and calculate the discharge, Q_i, with Manning's equation (using the estimated n) and flow depth values. Manning's n will vary depending upon lining type and flow depth.

Step 5. Compare Q_i with Q. If Q_i is within 2 percent of the design Q then proceed on to step 6. If not, return to step 3 and select a new estimated flow depth, d_{i+1}.

Step 6. Calculate the shear stress at maximum depth, τ_d (Equation 3), determine the permissible shear stress, τ_p ,and select an appropriate safety factor, SF. A safety factor of 1.0 is usually chosen, but may be increased as discussed earlier.

Step 7. Compare the permissible shear stress to the calculated shear stress from step 6 using Equation 3.

- If the permissible shear stress is adequate ($\tau_p \geq SF\,\tau_d$) then the lining is acceptable.

- If the permissible shear is inadequate ($\tau_p < SF\,\tau_d$), return to step 1 and modify the channel slope or shape, or return to step 2 and select an alternative lining type with greater permissible shear stress.

Once the selected lining is stable, the design process is complete. Other linings may be tested, if desired, before specifying the preferred lining.

10.7.2 Side Slope Stability

As described previously, shear stress is less on the channel sides than on the bottom. The maximum shear on the side of a channel is given by the following equation:

$$\tau_s = K_1 \tau_d \tag{5}$$

Where:

τ_s = side shear stress on the channel, (lb/ft2)

K1 = ratio of channel side to bottom shear stress

The value K_1 depends on the size and shape of the channel. For parabolic or V-shape with rounded bottom channels there is no sharp discontinuity along the wetted perimeter and therefore it can be assumed that shear stress at any point on the side slope is related to the depth at that point using Equation 3.

For triangular and trapezoidal channels, K_1 has been developed based on the work of Anderson, et al. (1970). The following equation may be applied.

$K_1 = 0.77$	$Z \leq 1.5$	
$K_1 = 0.066Z + 0.67$	$1.5 < Z < 5$	(6)
$K_1 = 1.0$	$5 \leq Z$	

The Z value represents the ratio of horizontal to vertical dimensions, Z:1 (H:V). Use of side slopes steeper than 3:1 is not encouraged for flexible linings because of the potential for erosion of the side slopes. For riprap, the basic design procedure is supplemented for channels with side slopes steeper than 3:1 as described in FDM 13-30-25.

10.7.3 Bend Stability

As Flow around a bend creates secondary currents that impose higher shear stresses on the channel sides and bottom compared to a straight reach (Nouh and Townsend, 1979) as shown in Figure 10.6, Superelevation Height in a Channel Bend. At the beginning of the bend, the maximum shear stress is near the inside and moves toward the outside as the flow leaves the bend. The increased shear stress caused by a bend persists downstream of the bend.

Equation 7 gives the maximum shear stress in a bend.

$$\tau_b = K_b \tau_d \qquad (7)$$

Where:

τ_b = side shear stress on the channel, (lb/ft2)

Kb = ratio of channel bend to bottom shear stress

The maximum shear stress in a bend is a function of the ratio of the radius of channel curvature to the top (water surface) width, R_C/T (Refer to Figure 10.5). As R_C/T decreases, that is as the bend becomes sharper, the maximum shear stress in the bend tends to increase. K_b can be determined from the following equation from Young, et al., (1996) adapted from Lane (1955):

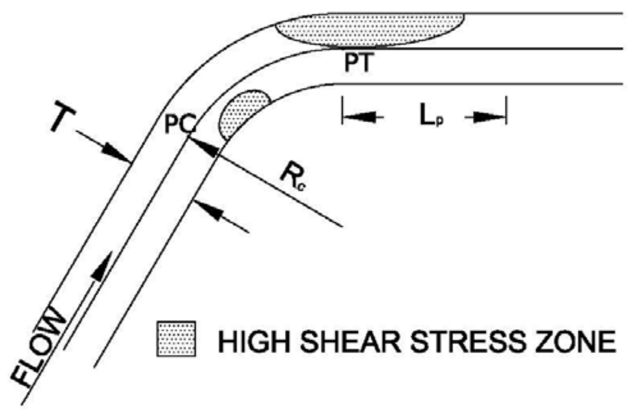


***Figure 10.5* Sheer Stress Distribution in a Channel Bend (Nouh and Townsend, 1979)**

Kb = 2.00	RC/T ≤ 2	
$K_b = 2.38 - 0.206\left(\frac{R_c}{T}\right) + 0.0073\left(\frac{R_c}{T}\right)^2$	2 < RC/T < 10	(8)
Kb = 1.05	RC/T ≥ 10	

Where,

R_c = radius of channel curvature of the bend to the channel centerline, (ft)

T = channel top (water surface) width, (ft)

The added stress induced by bends does not fully attenuate until some distance downstream of the bend. If added lining protection is needed to resist the bend stresses, this protection should continue downstream for a distance given by:

$$L_p = 0.60\left(\frac{R^{7/6}}{n}\right) \qquad (9)$$

Where:

L_p = length of protection, (ft)

R = hydraulic radius of the straight channel, (ft)

n = Manning's roughness for lining material in the bend

A final consideration for channel design at bends is the increase in water surface elevation at the outside of the bend caused by the superelevation of the water surface. Additional freeboard is necessary in bends and can be calculated using the following equation:

$$\Delta d = \left(\frac{V^2 T}{gR_c}\right) \tag{10}$$

Where:

Δd = additional freeboard required because of superelevation, (ft)

V = average channel velocity before the bend, (ft/s)

g = acceleration due to gravity, 32.2ft/s^2

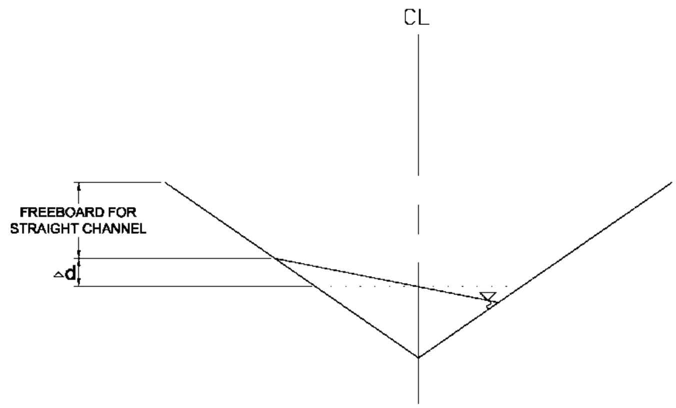


Figure 10.6 Superelevation Height in a Channel Bend

The design procedure for channel bends is summarized in the following steps:

Step 1. Determine the shear stress in the bend and check whether or not an alternative lining is needed in the bend.

Step 2. If an alternative lining is needed, select a trial lining type and compute the new hydraulic properties and bend shear stress.

Step 3. Estimate the required length of protection.

Step 4. Calculate superelevation and check freeboard in the channel.

10.7.3 Composite Linings

It is important that the bottom-lining material cover the entire channel bottom so that adequate protection is provided. To ensure that the channel bottom is completely protected, the bottom lining should be extended a small distance up the side slope. Computation of flow conditions in a composite channel requires the use of an equivalent Manning's n value for the entire perimeter of the channel. To determine the equivalent roughness, the channel area is divided into two parts of which the wetted perimeters and Manning's n values of the low-flow section and channel sides are known. These two areas of the channel are then assumed to have the same mean velocity.

The following equation is used to determine the equivalent roughness coefficient, n_e.

$$n_e = \left[\frac{P_L}{P} + \left(1 - \frac{P_L}{P}\right)\left(\frac{n_S}{n_L}\right)^{3/2} \right]^{2/3} n_L \qquad (11)$$

Where:

n_e = equivalent Manning's n value for the composite channel

P_L = low flow lining perimeter, (ft)

P = total flow perimeter, (ft)

n_s = Manning's n value for the side slope lining

n_L = Manning's n value for the low flow lining

When two lining materials with significantly different roughness values are adjacent to each other, erosion may occur near the boundary of the two linings. Erosion of the weaker lining material may damage the lining as a whole. In the case of composite channel linings with vegetation on the banks, this problem can occur in the early stages of vegetative establishment. A transitional lining should be used adjacent to the low-flow channel to provide erosion protection until the vegetative lining is well established.

The procedure for composite lining design is based on the design procedure for straight channels with additional sub-steps to account for the two lining types. The procedure is:

Step 1. Determine design discharge and select channel slope and shape. (No change.)

Step 2. Need to select both a low flow and side slope lining.

Step 3. Estimate the depth of flow in the channel and compute the hydraulic radius. (No change.)

Step 4. After determining the Manning's n for the low flow and side slope linings, use Equation 11 to calculate the effective Manning's n.

Step 5. Compare implied discharge and design discharge. (No change.)

Step 6. Determine the shear stress at maximum depth, τ_d ,(Equation 3), and the shear stress on the channel side slope, τ_s ,(Equation 5).

Step 7. Compare the shear stresses, τ_d and τ_s, to the permissible shear stress, τ_p, for each of the channel linings. If τ_d or τ_s is greater than the τ_p for the respective lining, a different combination of linings should be evaluated.

10.8 References

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FDM 13-30-15 Grass Lined Channels

October 22, 2012

15.1 Introduction

Vegetation is one of the most common long-term channel linings. Most roadside channels receive highway runoff from rainfall and snowmelt events and remain dry most of the time. For these conditions, upland species of vegetation (typically grasses) provide good erosion protection and can trap sediment and related contaminants in the channel section. Thus, the vegetated liner, or “grass swale” can be used to meet the post construction total suspended solids (TSS) reduction requirements listed in Wisconsin Administrative Code TRANS 401.106. However, grasses are not suited to sustained flow conditions or long periods of submergence. Common design practice for vegetative channels with sustained low flow and intermittent high flows is to provide a composite lining with riprap, grouted riprap, articulated concrete block or concrete providing a low flow section.

Vegetative linings consist of seeded or sodded grasses placed in and along the channel. Grasses are seeded and fertilized according to the requirements of the particular WisDOT seed mix and soil type.

Between seeding and vegetation establishment, the channel is vulnerable to erosion. Erosion mats provide erosion protection during the vegetation establishment period. These linings are typically degradable and do not provide ongoing stabilization of the channel after vegetation is established. Turf reinforcement mats enhance the performance of the vegetation by permanently reinforcing the turf root structure, which increases the permissible shear stress of the channel.

The behavior of established grass in an open channel lining is complicated. Grass stems flex as flow depth and shear stress increase, which reduces the roughness height and increases velocity and flow rate. As a result, a grass-lined channel cannot be described by a single roughness coefficient.

For channels where the design shear exceeds that of vegetation alone, but where vegetation is desirable from a cost or water quality standpoint, a turf reinforcement mat may be appropriate.

Kouwen and Unny (1969) and Kouwen and Li (1981) developed a model of the biomechanics of vegetation in open-channel flow. This model provides a general approach for determining the roughness of vegetated channels based upon the retardance classification. The resulting resistance equation (refer to HEC-15, Appendix C.2) uses the same vegetation properties as the Soil Conservation Service (SCS), now known as the NRCS, retardance approach but is more adaptable to the requirements of highway drainage channels. The design approach for grass-lined channels described in this procedure was developed from the Kouwen resistance equation, as found in HEC-15 Section 4

15.2 Grass Lining Properties

Vegetative linings are classified as Retardance A, B, C, D, or E according to a certain group of grasses of given heights as defined by the SCS. The SCS Retardance Table is in HEC-15, Table 4-1. Retardance A refers to grasses of high hydraulic resistance while Retardance E refers to grasses of low hydraulic resistance. WisDOT has established typical grass heights for selected retardance classification as identified in Table 15.1.

The density, stiffness, and height of grass stems are the main biomechanical properties of grass that relate to flow resistance and erosion control. The stiffness property (product of elasticity and moment of inertia) of grass is similar for a wide range of species (Kouwen, 1988) and is a basic property of grass linings. These properties are combined to describe the cover condition of the grass.

For design purposes, good cover of a well established grass channel is the typical reference condition. Use the fair condition when you anticipate difficulty establishing or maintaining a good stand of grass. The retardance classifications for WisDOT seed mixes are listed in Table 15.1. Note that native seed mixes (mixes 70 - 80) take about three years to develop and tend to be bunched, leaving bare soil between the clumps.

Table 15.1 Typical Height and Retardance Classification of Vegetal Covers by WisDOT Seed Mix

WisDOT Highway Seed Mixture Number	Retardance Classification	Typical grass height	Cover Factor
70,70A, 75, 80	B	2 ft (24 in)	Mixed
10, 20, 30, 40, 60	C(1)	0.67 ft (8 in)	Turf

1.) Use Retardance Classification Factor D for continuously mowed conditions of 4 inches or less.

15.3 Manning's Roughness

Manning's roughness coefficient for grass linings varies with the grass roughness parameter, C_n, and the shear force, τ_o , exerted by the flow. This is because the applied shear on the grass stem causes the stem to bend, which reduces the stem height relative to the depth of flow and reducing the roughness. The Manning's n roughness coefficient is represented by:

$$n = 0.213 C_n \tau_o^{-0.4} \qquad (12)$$

Where:

C_n = grass roughness coefficient

τ_o = mean boundary shear stress, lb/ft^2

Equation (12) is derived in Appendix C.2, HEC-15.

15.4 Permissible Shear Stress

The permissible shear stress of a vegetative lining is determined both by the underlying soil properties as well as those of the vegetation. Grass linings move shear stress away from the soil surface. The remaining shear at the soil surface is termed the effective shear stress. When the effective shear stress is less than the allowable shear stress for the soil surface, then erosion of the soil surface will be controlled. Grass linings provide shear reduction in two ways. First, the grass stems absorb a portion of the shear force within the canopy before it reaches the soil surface. Second, the grass plant (both the root and stem) stabilizes the soil surface against turbulent fluctuations. Hence, the effective shear at the soil surface is a function of the design shear stress τ_d the grass cover factor C_f, the soil grain roughness n_s, and the overall lining roughness n (refer to section 4.3.1, HEC-15 for more details).

15.5 Grass Cover Factor

The grass cover factor, C_f, varies with cover density and grass growth form (turf, bunch, or mixed). Bunchgrass is the general name for perennial grass species that tend to grow in discrete tufts or clumps (i.e., bunches) and does not spread by stolons or rhizomes like turf. Turf is grass and the part of the soil beneath it held together by the roots that forms a mat. Turf grasses may also be referred to as sod (i.e. in HEC 15). Mixed is a combination of the two types, individual clumps of bunchgrasses intermixed with turf grasses. The selection of the cover factor is a matter of engineering judgment since limited data are available. Cover factors are better for turf grasses than bunch grasses. In all cases a uniform stand of grass is assumed. Non-uniform conditions include wheel ruts, animal trails and other disturbances that run parallel to the direction of the channel. Estimates of cover factor are best for good uniform stands of grass; there is more uncertainty in the estimates of fair and poor conditions. Cover factor values are provided in the WisDOT grass swale design spreadsheet.

15.6 Permissible Soil Shear Stress

Soil boundary erosion occurs when the effective shear stress exceeds the permissible soil shear stress. Permissible soil shear stress is a function of particle size, cohesive strength, and soil density. The erodibility of coarse non-cohesive soils (defined as soils with a plasticity index less than 10) is due mainly to particle size, while fine-grained cohesive soils are controlled mainly by cohesive strength and soil density.

New ditch construction includes the placement of topsoil in the channel. Salvaged topsoil is typically gathered from locations on the project and stockpiled for revegetation work. Therefore, the important physical properties of the soil can be determined during the design by sampling surface soils from the project area. Since these soils are likely to be mixed together, average physical properties are acceptable for design.

15.6.1 Non-Cohesive Soils

For non-cohesive soils, the permissible soil shear stress depends upon the particle size D_{75}, which is the particle size for which 75% of the material by weight is smaller than that size. For fine-grained, non-cohesive soils with D_{75} < 0.05 in, the permissible shear stress is relatively constant and is conservatively estimated at 0.02 lb/ft^2. For coarse-grained non-cohesive soils (0.05 in < D_{75} < 0.6 in), the permissible shear stress varies with the particle size. For more detail, refer to HEC-15 section 4.3.2.1. Non-cohesive permissible soil shear stress values are calculated in the WisDOT Grass Swale design spreadsheet. Typical values for non-cohesive soils are listed in Table 15.2

Table 15.2 Typical Particle Sizes of Native Sands at 75 Percent Passing (D_{75})

Material Description (1)	USCS Classification	D75 (in)		
		Minimum	Average	Maximum
Silty and Clayey Fine Sand with a trace to no gravel	SM, SC	0.004	0.008	0.012
Silty and Clayey Fine to Coarse Sand with a trace to little gravel	SM, SC	0.012	0.031	0.12
Silty and Clayey Fine to Coarse Sand with gravel (>20%)	SM, SC	0.08	0.24	0.51
Fine Sand with a trace to no gravel, silt, and clay	SP, SW	0.008	0.016	0.02
Fine to Coarse Sand with a trace to little gravel and a trace to no silt and clay	SP, SW	0.024	0.063	0.12
Fine to Coarse Sand with gravel (>20%) and a trace to little silt and clay	SP, SW	0.16	0.28	0.39

Notes: (1) Percentage Descriptions: Trace = 1-9%, Little = 10-19%

15.6.2 Cohesive Soils

For cohesive soils, the permissible soil shear stress depends upon the ASTM soil classification, the soil plasticity index and the void ratio of the soil. Cohesive permissible soil shear stress values are calculated in the WisDOT Grass Swale design spreadsheet based upon these variables. Typical cohesive soils are listed below; more detail is given in HEC-15, section 4.3.2.2.

GM Silty gravels, gravel-sand silt mixtures
GC Clayey gravels, gravel-sand-clay mixtures
SM Silty sands, sand-silt mixtures
SC Clayey sands, sand-clay mixtures
ML Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
MH Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
CH Inorganic clays of high plasticity, fat clays

15.6.3 Combined Grass and Soil Shear Stress

The combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining results in a permissible shear stress for a vegetative lining. Equation 2 is used to calculate the permissible shear stress for the vegetative lining. For more detail, go to HEC-15, section 4.3.3.

$$\tau_p = \frac{\tau_{p,soil}}{(1-C_f)}\left(\frac{n}{n_s}\right)^2 \tag{13}$$

Where:

τ_p = permissible shear stress on the vegetative lining, (lb/ft^2)

$\tau_{p,soil}$ = permissible soil shear stress, (lb/ft^2)

C_f = grass cover factor

n_s = soil grain roughness

n = overall lining roughness

15.7 Grass Lined Channel Design Example

15.7.1 Grass Lined Channel Design Using HEC-15

A grass lined channel design example using the HEC-15 method is provided as Attachment 15.1.

15.7.2 Grass Lined Channel Design Using WisDOT Spreadsheet

WisDOT has prepared a spreadsheet that incorporates all the design guidelines and equations described in HEC-15 to design grass lined channels. The spreadsheet is divided into three sections:

1. A data entry section,
2. A results section, and
3. An intermediate calculations section.

An link to the spreadsheet and an example spreadsheet is provided as Attachment 15.2. Be sure to enable the spreadsheet Macros by clicking on the security warning "options" box on the top of the spreadsheet and then highlight the "enable this content" button. A step by step example problem is provided as Attachment 15.3.

15.8 References

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LIST OF ATTACHMENTS

Attachment 15.1	Grass Lined Channel Design Example (Using HEC-15)
Attachment 15.2	Grass Lined Channel Design WisDOT Spreadsheet Worksheet
Attachment 15.3	Grass Lined Channel Design Example (Using WisDOT Spreadsheet)

FDM 13-30-25 Rock Riprap Lined Channels

May 17, 2022

25.1 Introduction

Vegetation is one of the most common long-term channel linings. Most roadside channels receive highway runoff from rainfall and snowmelt events and remain dry most of the time. For these conditions, upland species HEC-15 (FHWA, 2005) contains design methods to design rock riprap channel linings, riprap on side slopes and channel protection in bends. The methods described in HEC-15 should not be used for rapidly varied flow, which often occurs at, for example, bridge abutments. For the design of riprap around bridge structures, consult with the Bureau of Structures Hydraulic Engineer.

Riprap, cobble, and gravel linings are considered permanent flexible linings. They are described as a noncohesive layer of stone or rock with a characteristic size D_{50}. The WisDOT rock riprap sizes are found in Standard Spec 606, Rock Riprap and Standard Spec 312, Select Crushed Material. Selected design characteristics are listed in Table 25.1. Refer to the standard specification for the required size distribution of each riprap type.

***Table 25.1* Typical Particle Sizes of Native Sands at 75 Percent Passing (D_{75})**

Riprap Type	D50 (inches)	D50 (feet)	Riprap Thickness (in)	Geotextile Type
Select Crushed Material	2.2	0.18	5	Type R
Extra Light (EX-LIGHT) Riprap	6	0.5	9	Type R
Light Riprap	10	0.83	12	Type R
Medium Riprap	12.5	1.04	18	Type HR
Heavy Riprap	16	1.33	24	Type HR
Extra-Heavy Riprap	20	1.67	30	Type HR

A geotextile must be placed beneath the riprap, as described in Table 25.1, to prevent the washout of the underlying soil. The fabric should be installed per the manufacturer's specifications. The riprap should be placed to form a well-graded interlocking mass with a minimum of voids. Rocks should be hard, durable, preferably angular in shape (see additional design considerations at the end of this section), and free from overburden, shale, and organic material. The rock should be resistant to disintegration from chemical and physical weathering.

The procedures in this section are applicable to uniform prismatic channels with rock sizes within the ranges given in Table 25.1. If the channel slope is less than or equal to 20%, then the designer should use the HEC-15 rock riprap design method described in this Procedure. If the channel slope is greater than 20%, then the designer should use the NRCS Design of Rock Chutes methodology described in FDM 13-30-30. For situations not satisfying the uniform prismatic channel condition such as stream banks or lakeshores, the designer is referred to HEC-11, "Design of Riprap Revetment" (FHWA, 1989).

25.2 Analysis of Slopes Less than or Equal to 20 Percent

For channel slopes that are less than or equal to 20%, the designer should use the design methodology described in HEC-15 Chapter 6, and repeated in this Procedure. This approach uses the permissible shear stress method to determine the appropriate size riprap for a drainage channel. The process includes a procedure to determine Manning's n for the channel followed by the approach necessary to determine the permissible shear stress for the given channel and flow. If the rock size from this analysis appears excessive, use the design approach for rock chutes described in FDM 13-30-30 and select the most appropriate design for the given site conditions using engineering judgment.

For runoff from bridge decks, review SDD 8D2 and SDD 8D3.

25.3 Manning's Roughness (for Rock Riprap Lined Channels)

Manning's roughness is a key parameter needed for determining the relationships between depth, velocity, and slope in a channel. However, for gravel and riprap linings, roughness has been shown to be a function of a variety of factors including flow depth and the particle size D50, which is the particle size for which 50% of the material by weight is smaller than that size. A partial list of roughness relationships includes Blodgett (1986a), Limerinos (1970), Anderson, et al. (1970), USACE (1994), Bathurst (1985), and Jarrett (1984). For the conditions encountered in roadside and other small channels, the relationships of Blodgett and Bathurst are adopted for this procedure.

Blodgett (1986a) proposed a relationship for Manning's roughness coefficient, n, that is a function of the flow depth and the relative flow depth (da/D50) as follows:

$$n = \frac{0.262 d_a^{1/6}}{2.25 + 5.23 \log\left(\frac{d_a}{D_{50}}\right)} \tag{14}$$

Where:

n = Manning's roughness coefficient, dimensionless

d_a = average flow depth in the channel, (ft)

D_{50} = median riprap/gravel size, (ft)

Equation 14 is applicable for the range of conditions where $1.5 \le d_a/D_{50} \le 185$. For small channel applications, relative flow depth should not exceed the upper end of this range.

Some channels may experience conditions below the lower end of this range where protrusion of individual riprap elements into the flow field significantly changes the roughness relationship. This condition may be experienced on steep channels, but also occurs on moderate slopes. The relationship described by Bathurst (1991) addresses these conditions and can be written as follows (See HEC-15, Appendix D for the original form of the equation):

$$n = \frac{1.49 d_a^{1/6}}{\sqrt{g}\, f(Fr)\, f(REG)\, f(CG)} \tag{15}$$

Where:

g = acceleration due to gravity, 32.2 ft/s^2

Fr = Froude number, $Q/\left(A/\sqrt{32.2 d_a}\right)$

REG = roughness element geometry (dimensionless)

CG = channel geometry (dimensionless)

Q = design flow (ft^3/s)

A = area of flow in channel (ft^2)

Equation 15 is a semi-empirical relationship applicable for the range of conditions where $0.3 < d_a/D_{50} < 8.0$. The three terms in the denominator represent functions of Froude number, roughness element geometry, and channel geometry given by the following equations:

$$f(Fr) = \left(\frac{0.28 Fr}{b}\right)^{\log(0.755/b)} \tag{16}$$

$$f(REG) = 13.434\left(\frac{T}{D_{50}}\right)^{0.492} b^{1.025(T/D_{50})^{0.118}} \tag{17}$$

$$f(CG) = \left(\frac{T}{d_a}\right)^{-b} \tag{18}$$

Where:

T = channel top width (ft)

b = parameter describing the effective roughness concentration (dimensionless)

The parameter b describes the relationship between effective roughness concentration and relative submergence of the roughness bed. This relationship is given by:

$$b = 1.14\left(\frac{D_{50}}{T}\right)^{0.453}\left(\frac{d_a}{D_{50}}\right)^{0.814} \qquad (19)$$

Equations 14 and 15 both apply in the overlapping range of $1.5 \le d_a/D_{50} \le 8$. For consistency and ease of application over the widest range of potential design situations, use the Blodgett equation (15) when $1.5 \le d_a/D_{50}$. The Bathurst equation (15) is recommended for $0.3 < d_a/D_{50} < 1.5$. As a practical problem, both methods require average depth to estimate n, while n is needed to determine average depth - setting up an iterative design process.

25.4 Permissible Shear Stress

Values for permissible shear stress for riprap and gravel linings are based on research conducted at laboratory facilities and in the field. The values developed in HEC-15 are judged to be conservative and appropriate for design use. Permissible shear stress is a function of the Shield's parameter, the specific weight of the rock and the water, and the mean riprap size. The Shields parameter is a dimensionless variable that embodies the factors that interact to initiate particle motion on a sediment bed. These factors include the Reynolds number, which is in turn a function of the shear velocity, the kinematic viscosity, and the mean riprap size. The details of these equations are described in Section 6.2 of HEC-15. Permissible shear stress is given by the following equation:

$$\tau_\rho = F^*(\gamma_s - \gamma)D_{50} \qquad (20)$$

Where:

τ_ρ = permissible shear stress (lb/ft^3)

F* = Shield's parameter (dimensionless)

γ_s = specific weight of the rock (lb/ft^3)

γ = specific weight of the water, 62.4 lb/ft^3

Typically, a specific weight of rock of 165 lb/ft^3 is used, but if the available rock is different from this value, the site-specific value should be used.

Recalling Equation 4 from FDM 13-30-10,

$$\tau_p \ge SF\,\tau_d$$

And Equation 3 from FDM 13-30-10,

$$\tau_d = \gamma d S_o$$

Equation 20 can be written in the form of a sizing equation for D_{50} as shown below:

$$D_{50} \ge \frac{SFdS_o}{F^*(SG-1)} \qquad (21)$$

Where:

d = maximum channel depth (ft)

SG = specific gravity of rock (γ_r/γ) (dimensionless)

τ_d = shear stress in channel at maximum depth (lb/ft^2)

SF = safety factor (greater than or equal to one) (dimensionless)

S_o = average bottom slope (equal to energy slope for uniform flow) (ft/ft)

Changing the inequality sign to equality gives the minimum stable riprap size for the channel bottom. Additional evaluation for the channel side slope is given in Section 6.3.2 of HEC-15.

Equation 21 is based on assumptions related to the relative importance of skin friction, form drag, and channel slope. However, skin friction and form drag have been documented to vary resulting in reports of variations in Shield's parameter by different investigators, for example Gessler (1965), Wang and Shen (1985), and Kilgore and Young (1993). This variation is usually linked to particle Reynolds number as defined below:

$$R_e = \frac{V_* D_{50}}{\upsilon} \qquad (22)$$

Where:

R_e = particle Reynolds number, dimensionless

V*= shear velocity (ft/s)

υ = kinematic viscosity, 1.217×10^{-5} ft^2/s at 60 degrees F

The shear velocity if defined as:

$$V_* = \sqrt{gdS} \qquad (23)$$

Where:

g = gravitational acceleration, 32.2 ft/s^2

S = channel slope (ft/ft)

Higher Reynolds number correlates with a higher Shields parameter as is shown in Table 25.2. For many roadside channel applications, Reynolds number is less than 4×10^4 and a Shields parameter of 0.047 should be used in Equations 20 and 21. In cases for a Reynolds number greater than 2×10^5, for example, with channels on steeper slopes, a Shields parameter of 0.15 should be used. Intermediate values of Shields parameter should be interpolated based on the Reynolds number.

Table 25.2 Selection of Shield's Parameter and Safety Factor

Reynolds Number	F *	SF
$\le 4 \times 10^4$	0.047	1.0
$4 \times 10^4 < R_e < 2 \times 10^5$	Linear interpolation	Linear interpolation
$\ge 2 \times 10^5$	0.15	1.5

Higher Reynolds numbers are associated with more turbulent flow and a greater likelihood of lining failure with variations of installation quality. Because of these conditions, it is recommended that the Safety Factor be increased with Reynolds number as shown in Table 25.2. Depending on site-specific conditions, safety factor may be further increased by the designer, but should not be decreased to values less than those in Table 25.2.

As channel slope increases, the balance of resisting, sliding, and overturning forces is altered slightly. Simons and Senturk (1977) derived a relationship that may be expressed as follows:

$$D_{50} \ge \frac{SFdS\Delta}{F^*(SG-1)} \qquad (24)$$

Where:

Δ = function of channel geometry and riprap size

The parameter Δ can be defined as follows (see HEC-15, Appendix D for the derivation):

$$\Delta = \frac{K_1(1+\sin(\alpha+\beta))\tan\phi}{2(\cos\phi\tan\phi - SF\sin\theta\cos\beta)} \tag{25}$$

Where:

K_1 = ratio of channel side to bottom shear stress

K_1 = 0.77 $\quad Z \leq 1.5$

K_1 = 0.066Z + 0.67 $\quad 1.5 < Z < 5$

K_1 = 1.0 $\quad 5 \leq Z$

Z = side slope, Z horizontal:1 vertical

α = angle of the channel bottom slope

β = angle between the weight vector and the weight/drag resultant vector in the plane of the side slope

Finally, β is defined by:

$$\beta = \tan^{-1}\left(\frac{\cos\alpha}{\frac{2\sin\theta}{\eta\tan\phi} + \sin\alpha}\right) \tag{26}$$

Where:

The stability number is calculated using:

$$\eta = \frac{\tau_s}{F^*(\gamma_s - \gamma)D_{50}} \tag{27}$$

Where:

τ_s = side shear stress on the channel, (lb/ft^2)

γ_s = specific weight of the rock, (lb/ft^3)

Riprap stability on a steep slope depends on forces acting on an individual stone making up the riprap. The primary forces include the average weight of the stones and the lift and drag forces induced by the flow over the rock. On a steep slope, the weight of a stone has a significant component in the direction of flow. Because of this force, a stone within the riprap will tend to move in the flow direction more easily than the same size stone on a milder gradient. As a result, for a given discharge, steep slope channels require larger stones to compensate for larger forces in the flow direction and higher shear stress.

The size of riprap linings increases quickly as discharge and channel gradient increase. Equation 24 is recommended when channel slope is greater than 10 percent and provides the riprap size for the channel bottom and sides. Equation 21 is recommended for slopes less than 5 percent. For slopes between 5 percent and 10 percent, it is recommended that both methods be applied and the larger size used for design. Values for safety factor and Shields parameter are taken from Table 25.2 for both equations.

25.5 Rock Riprap Design Procedure

This design procedure addresses the approach for selecting riprap for the bottom and sides of a channel, and for channel bends.

The riprap and gravel lining design procedure for the bottom of a straight channel is described in the following steps. It is iterative by necessity because flow depth, roughness, and shear stress are interdependent. The procedure requires the designer to specify a channel shape and slope as a starting point and is outlined in the eight-step process identified below. In this approach, the designer begins with a design discharge and calculates an acceptable D_{50} to line the channel bottom. The following steps are recommended for the standard design.

Step 1. Determine channel slope, channel shape, and design discharge.

Step 2. Select a trial (initial) D_{50}, perhaps based on available sizes for the project. Change the default specific weight if appropriate.

Step 3. Estimate the depth. For the first iteration, select a channel depth, d_i. For subsequent iterations, a new depth can be estimated from the following equation or any other appropriate method.

$$d_{i+1} = d_i \left(\frac{Q}{Q_i} \right)^{0.4} \tag{24}$$

Where:

Q_i = Flow estimate from previous iteration, (ft^3/s)

Determine the average flow depth, d_a in the channel, $d_a = A/T$

Step 4. Estimate Manning's n and the implied discharge. First, calculate the relative depth ratio, d_a/D_{50}. If d_a/D_{50} is greater than or equal to 1.5, use Equation 14 to calculate Manning's n. If d_a/D_{50} is less than 1.5, use Equation 15 to calculate Manning's n. Calculate the discharge using Manning's equation.

Step 5. If the calculated discharge is within 2 percent of the design discharge, then proceed to step 6. If not, go back to step 3 and estimate a new flow depth.

Step 6. Calculate the particle Reynolds number using Equation 22 and determine the appropriate Shields parameter and Safety Factor values from Table 2. If channel slope is less than 5 percent, calculate required D_{50} from Equation 21. If channel slope is greater than 10 percent, use Equation 24. If channel slope is between 5 and 10 percent, use both Equations 21 and 24 and take the largest value.

Step 7. If the D_{50} calculated is greater than the trial size in step 2, then the trial size is too small and unacceptable for design. Repeat procedure beginning at step 2 with new trial value of D_{50}. If the D_{50} calculated in step 6 is less than or equal to the previous trial size, then the previous trial size is acceptable. However, if the D_{50} calculated in step 6 is sufficiently smaller than the previous trial size, the designer may elect to repeat the design procedure at step 2 with a smaller, more cost effective, D_{50}.

25.6 Design Example (Using Equations): Riprap Channel (Mild Slope)

Attachment 25.1 provides an example design example solved using equations for a riprap channel that has a mild slope.

25.7 Example Riprap Lined Design for Channel Slopes ≤ 20% Using the WisDOT Spreadsheet

Attachment 25.2 is a blank example sheet and provides a link to a working copy of a WisDOT Spreadsheet for riprap lined design channel with slope less than or equal to 20%. Be sure to enable the spreadsheet Macros by clicking on the security warning "options" box on the top of the spreadsheet and then highlight the "enable this content" button. Attachment 25.3 is a step by step example using the WisDOT spreadsheet for riprap lined channel that have a mild slope.

25.8 Additional Design Considerations

25.8.1 Water Surface Profiles, HEC RAS

In situations where it is necessary to determine the water surface profile of a channel with varying channel characteristics and flow rates, a program that analyzes gradually varied flow must be employed. One such computer program, entitled "HEC-RAS River Analysis System," was developed and first published by the U. S. Army Corps of Engineers (USACE) in 1968. The current version 4.1 (USACE, 2010) is available from the USACE web site: http://www.hec.usace.army.mil/software/hec-ras/. For further information on this subject, refer to the discussion in FDM 13-20-1 under "Water Surface Profiles, (HEC-2) and (HEC-RAS)."

25.8.2 Angular vs. Rounded Riprap

The riprap design methods described in this procedure assume that the contractor will construct riprap channels with angular riprap. If the designer does not expect that angular riprap will be available for the project and that the contractor will be using rounded riprap, then the design riprap size should be increased by 40% (Ullmann and Abt, 2000). To apply this to a channel design, for channel slopes less than or equal to 10% increase the size of the riprap to the next highest size. Refer to Attachment 25.5 for a map of those areas of the state where rounded riprap is predominantly available.

25.8.3 Inflow from the Sides

The riprap design methods Channels that intercept surface flow from the sides must incorporate into their design the following criteria:

1. The lining shall be carried to an elevation slightly below the ground level.
2. A cut-off wall must be placed at the top of the lining to prevent undermining.
3. Pipes discharging into the channel shall be flush with the channel lining.

25.8.4 Drainage

If hydrostatic pressure is foreseen behind the sidewalls of an apron endwall discharging into a channel, install both weep holes and a subsurface drainage system behind the sidewalls.

25.8.5 Bulking

At supercritical velocities, air entrainment occurs causing increases in the depth of flow (bulking effect). With concrete-lined channels, determine the normal depth of flow with a bulking condition by setting Manning's "n" equal to 0.018 instead of 0.014. For other lining types, multiply the n values calculated using the appropriate design process by 1.3.

25.9 References

1. Bathurst, J.C., 1985. "Flow Resistance Estimation in Mountain Rivers," Journal of Hydraulic Engineering, ASCE, Vol. 111, No. 4.

2. Bathurst, J.C. R.M. Li, and D.B. Simons, 1981. "Resistance Equation for Large-Scale Roughness." Journal of the Hydraulics Division, ASCE, Vol. 107, No. HY12, Proc. Paper 14239, December, pp. 1593-1613.

3. Blodgett, J.C., 1986. "Rock Riprap Design for Protection of Stream Channels Near Highway Structures, Volume 1 - Hydraulic Characteristics of Open Channels," USGS Water Resources Investigations Report 86-4127.

4. U.S. Department of Transportation, Federal Highway Administration, Design of Riprap Revetment, Hydraulic Engineering Circular (HEC) No. 11, Washington, D.C. March 1989, 182 pp.

5. U.S. Department of Transportation, Federal Highway Administration, Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular #15, Washington, D.C., September 2005.

6. U.S. Army Corps of Engineers Hydraulic Engineering Center, "HEC-RAS River Analysis System Applications Guide, Version 4.0, March 2008.

7. Ullmann, Craig M. and Abt, Steven R., “Stability of Rounded Riprap in Overtopping Flow”, 2000 Joint Conference on Water Resource Engineering and Water Resouce Planning and Management, Minneapolis, MN, 2000.

LIST OF ATTACHMENTS

Attachment 25.1	Design Example (Using Equations):Riprap Channel (Mild Slope)
Attachment 25.2	Riprap Channel (Mild Slope) WisDOT Spreadsheet Worksheet
Attachment 25.3	Instructions and Example for Riprap Lined Design for Channel Slopes ≤ 20% Using the WisDOT Spreadsheet
Attachment 25.4	Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone
Attachment 25.5	Map of Areas in Wisconsin where Rounded Riprap is Predominantly Available

FDM 13-30-30 Rock Riprap Lined Chutes — May 17, 2022

30.1 Introduction

The Natural Resource Conservation Service (NRCS) has developed a design procedure for rock chute drainageways that WisDOT has modified to account for WisDOT specific riprap sizes and design requirements. A rock chute is defined as a channel with a slope that is 5:1 or steeper and that has an energy dissipation structure at the toe of the chute. This method should not be used for rapidly varied flow, which often occurs at, for example, bridge abutments.

The WisDOT rock riprap sizes are found in the Standard Spec 606, Rock Riprap and Standard Spec 312, Select Crushed Material. Design characteristics are listed in Table 30.1

Table 30.1 WisDOT Riprap Size Classifications

Riprap Type	D50 (inches)	D50 (feet)	Riprap Thickness (in) SF=1.2(1)	Riprap Thickness (in) SF=2.0 1	Geotextile Lining Type
Select Crushed Material	2.2	0.18	6	9	Type R
Extra Light (EX-LIGHT) Riprap	6	0.5	14	24	Type R
Light Riprap	10.	0.83	24	40	Type R
Medium Riprap	12.5	1.04	30	50	Type HR
Heavy Riprap	16	1.33	38	64	Type HR
Extra-Heavy Riprap	20	1.67	48	80	Type HR

Note: (1) Riprap thickness must equal 2 x D_{50} x SF, where SF is the selected factor of safety for the riprap design. This thickness requirement must be noted in the special provisions.

Riprap requires a filter material between the rock and the underlying soil to prevent soil washout. WisDOT requires a geotextile lining beneath the riprap, as described in Table 30.1, which is used as the filter. The lining should be installed per the manufacturer’s specifications. Riprap is placed on the geotextile to form a well-graded mass with a minimum of voids. Rocks should be hard, durable, preferably angular in shape, and free from overburden, shale, and organic material. The rock should be resistant to disintegration from chemical and physical weathering.

The procedures in this section are applicable to uniform prismatic channels (as would be characteristic of roadside channels) with rock sizes within the range in Table 30.1. If the channel slope is less than or equal to 10%, then the designer should use the HEC-15 rock riprap design method described in FDM 13-30-25. If the channel slope is greater than 10%, then the designer should use the NRCS Design of Rock Chutes methodology described in this procedure. For situations not satisfying the uniform prismatic channel condition such as stream banks or lakeshores, refer to HEC-11, “Design of Riprap Revetment” (FHWA, 1989).

30.2 Steep Slope Analysis

If channel slopes are greater than 10%, then the designer should use the design procedures for a rock-lined chute. This approach is based upon the paper "Design of Rock Chutes" by Robinson, Rice, and Kadavy, ASAE Vol. 41(3), pp. 621-626, 1998. The design procedure was developed by the Iowa NRCS design staff, who developed a design spreadsheet based upon the rock chute design procedures. This spreadsheet was modified by NRCS WI and by WisDOT.

The tests that were used to develop this approach focused on rock slope stability, roughness, and outlet stability. The relationship to predict rock size for the highest stable unit discharge uses rock size as a function of the discharge and channel slope. Rice et al. (1997) developed empirical relationships to predict the Manning roughness coefficient as a function of stone size and bed slope. These roughness relationships allow calculation of the flow depth in a rock chute.

Rock size for chutes shall be expressed by the D_{50} size (50 percent passing by weight). To determine the D_{50} size for slopes (S_{ch}) greater than 10% (10:1), use the following equation:

$$D_{50} = \left[\frac{q_t (S_{ch})^{0.58}}{0.0395} \right]^{1/1.89} \qquad (28)$$

Where:

D_{50} = median rock size in inches

q_t = equivalent unit discharge in (ft^3/sec) per foot of channel width

S_{ch} = chute slope (ft/ft)

The roughness value of the rock lining varies according to the rock size (D_{50}) and the slope of the chute's bed S_{ch}. Manning's n is found using the following equation:

$$n = 0.047(D_{50}\, S_{ch})^{0.147} \qquad (29)$$

Where:

n = Manning's n for given rock size and chute slope

The riprap design methods described in this procedure assume that the contractor will construct riprap channels with angular riprap. If the designer does not expect that angular riprap will be available for the project and that the contractor will be using rounded riprap, then use the safety factor to compensate for any unexpected variables in the flow, rock shape, or the n value. The minimum SF allowed on any chute is 1.2. For rounded riprap, set SF = 2.0. Refer to FDM 13-30 Attachment 25.5 for a map of those areas of the state where rounded riprap is predominantly available Use your engineering judgment to determine an appropriate safety factor for riprap that combines angular and rounded rock. The SF is used in Equation 3 to determine the D_{50}* that shall be used in the design.

$$D_{50}^* = D_{50} \times SF \qquad (30)$$

Where:

D_{50}* = Minimum design rock size (in)

To use the design spreadsheet for rock chutes, enter values for the following variables, which are highlighted as blue, bold numbers in a box.

30.2.1 Input Geometry - Upstream Channel

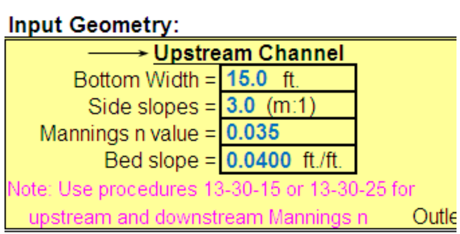


Figure 30.1 Riprap Lined Chute - Input Geometry

Bottom Width - The bottom width of the upstream channel (ft).

Side slopes - The side slope of the upstream channel (ft.Horizontal:1 ft. Vertical).

Mannings n Values - The Mannings n value of the upstream channel. Use either FDM 13-30-15 or FDM 13-30-25 to determine these values if they are not known.

Bed slope – The bed slope of the upstream channel (ft./ft.).

30.2.2 Input Geometry - Chute

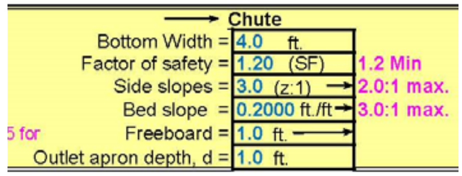


Figure 30.2 Riprap Lined Chute - Chute Input

Bottom Width - The bottom width of the upstream channel (ft).

Factor of Safety - The safety factor for the chute design, selected using engineering judgment and the guidelines described about equation (3) in this section. The minimum value is 1.2.

Side slopes - The side slope of the chute (ft.Horizontal:1 ft. Vertical). The steepest allowable side slope is 2:1.

Bed slope - The bed slope of the chute (ft./ft.). The steepest allowable side slope is 3:1, or 0.333 ft/ft.

Freeboard - The required distance from the top of the water surface in the chute and the top edge of the chute (ft).

Outlet apron depth - The distance between the bottom of the discharge apron and the downstream channel bottom (ft).

30.2.3 Input Geometry - Downstream Channel

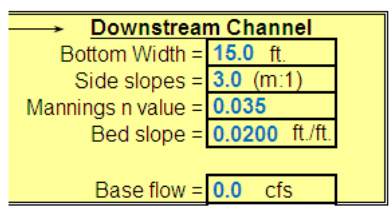


Figure 30.3 Riprap Lined Chute - Downstream Channel

Bottom Width - The bottom width of the downstream channel (ft).

Side slopes - The side slope of the downstream channel (ft.Horizontal:1 ft. Vertical).

Manning's n value - The Manning's n value of the downstream channel. Use either FDM 13-30-15 or FDM 13-30-25 to determine these values if they are not known.

Bed slope - The bed slope of the downstream channel (ft./ft.).

Base flow - The constant flow rate due to non-runoff related discharges that is added to the design flow (cfs).

30.2.4 Flow and Elevation Data

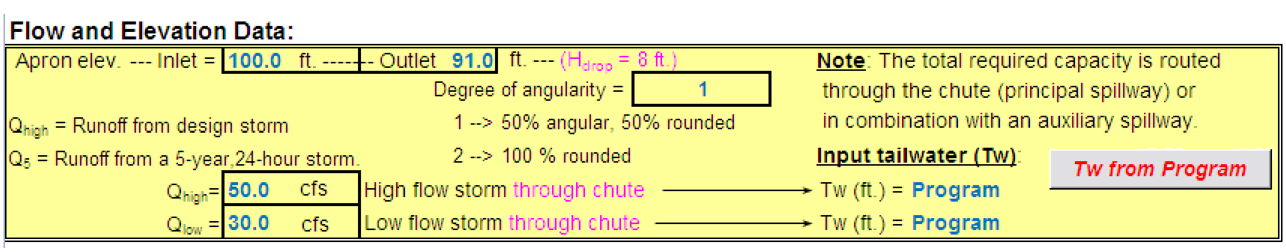


Figure 30.4 Flow and Elevation Data

Apron elevation - inlet - The inlet elevation of the chute apron (ft)

Apron elevation - outlet - The outlet elevation of the chute apron (ft). The H_{drop} value is the difference between the inlet and outlet elevations less the outlet elevation depth.

Degree of angularity - Enter a ‘1’ if the rock is at least 50% angular rock or a ‘2’ if the riprap is less than 50%

angular rock. If the rock is less than 50% angular and the safety factor is less than 1.7 a warning message will appear below the apron elevation data.

Q_{high} - The peak design storm flow value (cfs).

Q_{low} - The low flow design storm flow value (cfs), which is typically a 5-year design flow rate.

Input Tailwater - The user has the option of allowing the program to calculate the tailwater values used in the analysis or to enter specific values for the tailwater elevation. For the former option, press the ‘Tw from Program’ button and the word ‘Program’ will appear in place of a tailwater value.

30.2.5 Profile and Cross Section (Output)

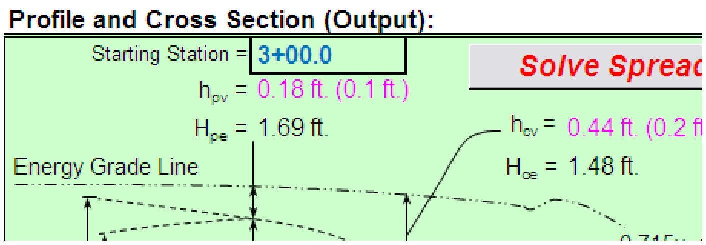


Figure 30.5 Profile and Cross Section (Output)

Starting Station - The station at the upstream edge of the rock chute inlet apron. This value is used to calculate the stationing for the Stakeout Notes listed on the Plan Sheet.

30.3 Rock Chute Design Spreadsheet

30.3.1 Spreadsheet Overview

There are five tabs included on the "Rock Chute Design Data" spreadsheet:

1. Design Data Tab (refer to example included as Attachment 30.1) - Enter the project name, designer and date as well as the upstream and downstream chute and flow data listed above. You will also initiate the calculations to determine the appropriate size riprap from this tab. Print this tab to document your work.
2. Plan Sheet Tab (refer to example included as Attachment 30.2) - Review the spreadsheet output and enter unit cost information on this tab.
3. Construction Detail Tab (refer to example included as Attachment 30.3) - This tab provides a printout of the proposed design. Print this tab to document your work.
4. Calculations Tab - This tab contains the formulas needed to calculate the appropriate riprap size for the channel or rock chute.
5. Help Tab - The variable definitions and instructions for using the spreadsheet are on this tab.

Be sure to enable the spreadsheet Macros by clicking on the security warning "options" box on the top of the spreadsheet and then highlight the "enable this content" button.

30.3.2 Spreadsheet Directions

Follow these steps to evaluate a steep slope for riprap protection:

1. Enter your project information and data on the ‘Design Data’ sheet. Begin with the ‘Input Geometry and Flow’ section, which is the major input area for setting channel geometry. All blue values and text can be entered (or changed) by the user.
2. Changing any value (with the exception of Freeboard under the inlet channel column, Outlet apron depth, d, and the Factor of safety (multiplier) under the chute column) will clear the output values in the Profile and Cross Section area.
3. Enter the Inlet and Outlet apron elevation.
4. Enter the high and low frequency storm (in cfs) flowing through the chute portion of the structure (this program does not design the auxiliary spillway).
5. Select the ‘Tw from Program’ button if you want the spreadsheet to calculate the tailwater for both high and low flow events, or enter your own values. If you select the ‘Tw from Program’ button, the

spreadsheet will enter the word "Program" in the tailwater cells indicating that the spreadsheet will calculate the tailwater.

6. Select the 'Solve Spreadsheet' button when finished entering data.

30.3.3 Spreadsheet Notes

Designers need to be aware of the following spreadsheet notes;

1. The program sets a limit on the steepest side slope allowed in the chute (2:1) and the steepest bed slope (3:1). Values steeper than these will blank the output area and the program cannot be solved or printed (just to the right of these cells a note will indicate "Too Steep").
2. Enter a 1.0-foot "suggested" minimum for d, and always make sure that T_w + d is greater than or equal to z_2.
3. Select the 'Solve Spreadsheet' button if you change Q_{high} or Q_{low}.

The link to the spreadsheet that WisDOT has revised that incorporates all the design guidelines and equations described in Robinson, Rice and Kadavy (1998) to design steep rock lined channels is located on the top of Attachment 30.1.

30.3.4 Riprap Lined Chute Example Spreadsheets

The following example illustrates the design procedure. Enter the data into the design spreadsheet on the appropriate row. Assume the following:

Table 30.2 Riprap Lined Chute Design Example Data

Upstream Channel		**Downstream Channel**	
Bottom Width =	4.0 ft	Bottom Width =	4.0 ft
Side Slopes =	3.0	Side Slopes =	3.0
Mannings n value =	0.040	Mannings n value =	0.040
Bed Slope =	0.01	Bed Slope =	0.01 ft
Chute		Base Flow =	0 cfs
Bottom Width =	4.0 ft	**Flow and Elevation Data**	
Factor of Safety =	1.2	Apron Elevation – Inlet =	100.0 ft
Side Slopes =	3.0	Apron Elevation – Outlet =	91.0 ft
Bed Slope =	0.10	High Flow Rate =	50 cfs
Freeboard =	0.5 ft	Low Flow Rate =	30 cfs
Outlet Apron Depth =	1.0 ft	Tailwater from Program? =	YES
Degree of angularity =	1		

To evaluate the data, press the "Solve Spreadsheet" button. Refer to Attachments 30.2and 30.3for the example output. Review the critical slope information (Design Data tab, cell A41) and the High Flow Storm Information on the Design Data Tab. The design riprap gradation is listed on the Plan Sheet tab, cell A42, as is the overall design. If the channel or chute does not function adequately, you can flatten the side slopes and/or widen the channel bottom to decrease the flow depth.

Additional information is available from the 'Help' tab of the spreadsheet.

30.3.5 Additional Design Considerations

Water Surface Profiles, HEC-RAS: In situations where it is necessary to determine the water surface profile of a channel with varying channel characteristics and flow rates, use a program that analyzes gradually varied flow.

One such computer program, entitled "HEC-RAS River Analysis System," was developed and first published by the U. S. Army Corps of Engineers (USACE) in 1968. The current version 4.0 (USACE, 2008) is available from the USACE web site: http://www.hec.usace.army.mil/software/hec-ras/. For further information on this subject, see the discussion in FDM 13-20-1 under "Water Surface Profiles, (HEC-2) and (HEC-RAS)."

Inflow from the Sides: Channels that intercept surface flow from the sides must incorporate into their design the criteria that follow:

1. The lining shall be carried to an elevation slightly below the ground level.
2. A cut-off wall must be placed at the top of the lining to prevent undermining.
3. Pipes discharging into the channel shall be flush with the channel lining.

Drainage: If hydrostatic pressure is foreseen behind the sidewalls of an apron endwall discharging into a channel, install both weep holes and a subsurface drainage system behind the sidewalls.

Bulking: At supercritical velocities, air entrainment occurs, causing increases in the depth of flow (bulking effect). With concrete-lined channels, determine the normal depth of flow with a bulking condition by setting Manning's "n" equal to 0.018 instead of 0.014. For other lining types, multiply the n values calculated using the appropriate design process by 1.3.

30.4 References

1. Rice, C.E., K.C. Kadavy , and K.M. Robinson, “Roughness of Loose Rock Riprap on Steep Slopes”, Journal of Hydraulic Engineering, American Society of Civil Engineers, Volume 124(2), 1998.

2. Lorenz, E.EA, and K.M. Robinson, “An Excel Program to Design Rock Chutes for Grade Stabilization”, 2000 ASAE Annual International Meeting, Milwaukee, WI,, July 2000, ASAE Meeting Presentation Paper No. 002008.

3. Robinson, Rice and Kadavy, “Design of Rock Chutes”, ASAE Volume 41(3), pgs 621-626, 1998.

4. U.S. Army Corps of Engineers Hydraulic Engineering Center, "HEC-RAS River Analysis System Applications Guide, Version 4.0, March 2008.

5. U.S. Department of Transportation, Federal Highway Administration, Design of Riprap Revetment, Hydraulic Engineering Circular (HEC) No. 11, Washington, D.C. March 1989, 182 pp.

6. Ullmann, Craig M. and Abt, Steven R., “Stability of Rounded Riprap in Overtopping Flow”, 2000 Joint Conference on Water Resource Engineering and Water Resouce Planning and Management, Minneapolis, MN, 2000.

LIST OF ATTACHMENTS

GRASS LINED CHANNEL DESIGN EXAMPLE USING HEC-15

Given the following channel geometry, soil conditions, grade, and design flow, determine if a grass lining that will be maintained in good condition in the spring and summer months (the main storm seasons) is an acceptable channel liner.

Given:

Geometry: Triangular (V-ditch), B = 0.0 ft, Z = 4

Soil: Clayey sand (SC classification)

PI = 16

e = 0.35

Soil Type: Cohesive

Grass: Vegetation Type: Turf

Retardance Class: Type C

Grass Height, h = 0.67 ft

Grass Condition: Good

Grass Density-Stiffness Coefficient, C_s, = 9.0

Slope: 2.0 percent

Flow: 15 ft^3/s

Solution:

Use the procedure in Section 3.1, HEC-15 for a straight channel.

Step 1. Channel slope, soil type, geometry, and discharge are given.

Step 2. Assume an initial depth = 1.0 ft.

From the geometric relationship of a trapezoid (see HEC-15, Appendix B):

A (Area) = $Bd + Zd^2 = 0.0(1.0) + 4(1.0)^2 = 4.00$ ft^2

P (Wetted Perimeter) = $B + 2d(Z^2 + 1)^{1/2} = 0.0 + 2(1.0)(4^2 + 1)^{1/2} = 8.25$ ft

R (Hydraulic Radius) = A/P = (4.00)/(8.25) = 0.485 ft

Step 3. To estimate n, determine the applied shear stress on the grass lining and the discharge:

$\tau_o = \gamma R S_o$ = 62.4(0.485)(0.02) = 0.61 lb / ft^2 (Refer to FDM 13-30-10, Equation 2)

Determine a Manning's n from FDM 13-30-15, Equation 12 with

$C_n = 0.237(C_s)^{0.1}(h)^{0.528} = 0.237(9.0)^{0.1}(0.67)^{0.528} = 0.238$ (HEC-15, Section 4.1)

$n = 0.213 C_n \tau_o^{-0.4}$ = $0.213(0.238)(0.61)^{-0.4} = 0.062$

The discharge is calculated from Manning's equation (Refer to FDM 13-30-10, Equation 1):

$$Q = \frac{1.49}{n} A R^{2/3} S_0^{1/2}$$ = $1.49/0.062(4.00)(0.485)^{2/3}(0.02)^{1/2} = 8.4$ ft^3/s

Step 4. Since this calculated flow value is more than 2 percent different from the design flow, repeat step 2 with a new, estimated flow depth.

Step 2 (2nd iteration). Estimate a new depth more than the initial estimate as the calculated Q < given Q

d = 1.21 ft, and calculate the new geometric channel properties:

$A = Bd + Zd^2 = 0.0(1.21) + 4(1.21)^2 = 5.86$ ft^2

$P = B + 2d\,(Z^2 + 1)^{1/2} = 0.0 + 2(1.21)\,(4^2 + 1)^{1/2} = 9.98$ ft

R = A/P = (5.86)/(9.98) = 0.587 ft

Step 3 (2nd iteration). To estimate n, determine the applied shear stress on the grass lining and the discharge:

$\tau_o = \gamma R S_o$ = 62.4(0.587)(0.02) = 0.73 lb / ft (Refer to FDM 13-30-10, Equation 2)

Determine Manning's n from FDM 13-30-15, Equation 12:

$C_n = 0.237(C_s)^{0.1}(h)^{0.528} = 0.237(9.0)^{0.1}(0.67)^{0.528} = 0.238$ (HEC-15, Section 4.1)

$n = 0.213 C_n \tau_o^{-0.4}$ = $0.213(0.238)(0.73)^{-0.4} = 0.057$

The discharge is calculated from Manning's equation:

$$Q = \frac{1.49}{n} A R^{2/3} S_0^{1/2} = 1.49/0.057(5.86)(0.587)^{2/3}\,(0.02)^{1/2} = 15.2\ ft^3/s$$

Step 4 (2nd iteration) Since this value is within 2 percent of the design flow, proceed to step 5.

Step 5. The maximum shear on the channel bottom is.

$$\tau_d = \gamma d S_o = 62.4(1.21)(0.02) = 1.51\ lb/ft^2$$

The permissible soil shear stress is given by Equation 4.6., HEC-15 Section 4.3.3.2

$$\tau_{p,soil} = (c_1 PI^2 + c_2 PI + c_3)(c_4 + c_5 e)^2 c_6 = (1.07(16)^2 + 14.3(16) + 47.7)(1.42 - 0.61(0.35))^2(0.00010) = 0.08\ lb/ft^2$$

Equation 2 gives the permissible shear stress on the vegetation. The value of C_f is found in Table 4.5, HEC-15 Section 4.3.1. The soil grain roughness, n_s, equals 0.016 unless the D_{75} of the soil is greater than 0.05 in.

$$\tau_p = \frac{\tau_{p,soil}}{(1-C_f)}\left(\frac{n}{n_s}\right)^2 = (0.08/(1-0.9))(0.057/0.016)^2 = 10.2\ lb/ft^2$$

The safety factor for this channel is taken as 1.0. (Refer to FDM 13-30-10, General Design Procedures)

Step 6. The grass lining is acceptable since the maximum shear on the vegetation (1.51 lb/ft^2) is less than the permissible shear of 10.2 lb/ft^2.

Grass Lined Channel Design WisDOT Spreadsheet Worksheet

Download a zipped working copy of the spreadsheets at:

http://wisconsindot.gov/rdwy/fdm/files/WisDOT-Stormwater-Drainage-WQ-Channel-Spreadsheets.zip

1	**Lining Type: Vegetation**								
2	**Project ID:**								
3	**Location:**								
4	**Designer/Checker:**								
5	**Date:**								
6									
7	**STA**	**10+00**	**10+00**	**12+00**	**13+00**	**14+00**	**15+00**	**16+00**	**17+00**
8	**Left, Center or Right**	**R**	**R**	**R**	**R**	**R**	**R**	**R**	**R**
9	***Channel/Ditch Geometry***								
10	Channel Slope, S_o (ft/ft)	**0.02**	**0.02**						
11	Channel Bottom Width, B (ft)	**0**	**0**						
12	Channel Side Slope, z_1	**4**	**4**						
13	Channel Side Slope, z_2	**4**	**4**						
14	Flow Depth, d (ft) Solve iteratively	**1.00**	**1.21**						
15	Safety Factor, SF	**1.0**	**1.0**						
16	***Vegetation/Soil Parameters***								
17	Vegetation Retardance Class	**C**	**C**						
18	Vegetation Condition	**good**	**good**						
19	Vegetation Growth Form	**turf**	**turf**						
20	Soil Type	**cohesive**	**cohesive**						
21	D_{75} (in) (Set at 0.00 for cohesive soils)								
22	ASTM Soil Class	**SC**	**SC**						
23	Plasticity Index, PI	**16**	**16**						
24	***Results Summary***								
25	Design Q (ft^3/s)	**15.0**	**15.0**						
26	Calculated Q (ft^3/s)	8.4	15.2	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
27	Difference Between Design & Calc. Flow (%)	-44.0%	1.2%	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
28	**Stable (Yes or No)**	**YES**	**YES**	**#VALUE!**	**#VALUE!**	**#VALUE!**	**#VALUE!**	**#VALUE!**	**#VALUE!**
29	***Channel Parameters***								
30	Vegetation Height, h (ft)	0.67	0.67	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
31	Grass Roughness Coefficient, C_n	0.238	0.238	undefined	undefined	undefined	undefined	undefined	undefined
32	Cover Factor, C_f	0.90	0.90	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
33	Noncohesive Soil								
34	Soil Grain Roughness, n_s	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016
35	Permissible Soil Shear Stress, τ_p (lb/ft^2)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
36	Cohesive Soil								
37	Porosity, e	0.35	0.35	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
38	Soil Coefficient 1, c_1	1.0700	1.0700	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
39	Soil Coefficient 2, c_2	14.30	14.30	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
40	Soil Coefficient 3, c_3	47.700	47.700	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
41	Soil Coefficient 4, c_4	1.42	1.42	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
42	Soil Coefficient 5, c_5	-0.61	-0.61	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
43	Soil Coefficient 6, c_6	0.00010	0.00010	FALSE	FALSE	FALSE	FALSE	FALSE	FALSE
44	Permissible Soil Shear Stress, τ_p (lb/ft^2)	0.080	0.080	N/A	N/A	N/A	N/A	N/A	N/A
45	Total Permissible Shear Stress, τ_p (lb/ft^2)	0.080	0.080	0.000	0.000	0.000	0.000	0.000	0.000
46	Cross Sectional Area, A (ft^2)	4.000	5.856	0.000	0.000	0.000	0.000	0.000	0.000
47	Wetted Perimeter, P (ft)	8.25	9.98	0.00	0.00	0.00	0.00	0.00	0.00
48	Hydraulic Radius, R (ft)	0.485	0.587	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
49	Top Width, T (ft)	8.00	9.68	0.00	0.00	0.00	0.00	0.00	0.00
50	Hydraulic Depth, D (ft)	0.500	0.605	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
51	Froude Number (Q design)	0.523	0.587	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
52	Channel Shear Stress, τ_o (lb/ft^2)	0.61	0.73	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
53	Actual Sheer Stress, τ_d (lb/ft^2)	1.25	1.51	0.00	0.00	0.00	0.00	0.00	0.00
54	Mannings n	0.062	0.057	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
55	Average Velocity, V (ft/s)	3.75	2.56	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
56	Calculated Flow, Q (ft^3/s)	8.4	15.2	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
57	Difference Between Design & Calc. Flow (%)	-44.0%	1.2%	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
58	Effective Shear on Soil Surface, τ_e (lb/ft^2)	0.008	0.012	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
59	Total Permissible Shear on Veg., $\tau_{p,veg}$ (lb/ft^2)	12.03	10.17	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!
60	Stable (Y or N)	YES	YES	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!	#VALUE!

GRASS LINED CHANNEL DESIGN EXAMPLE (USING WisDOT SPREADSHEET)

To use this spreadsheet, enter data into all the grey cells. The spreadsheet is designed so that you can select the cells in column C from rows 5 to 52 and drag them across to create additional columns for additional swale segments.

Enter the following information into the spreadsheet, as shown in the example to the right. Note that many of the variables are selected using a drop-down list box. In Figures 1 to 3, the first data column represents values from the first iteration of the above example, and the second column displays values from the second iteration of the above example.

Lines 2 – 5: Enter the project information as described.

Lines 7 – 8: For each channel segment, enter the highway station and whether the channel is on the left, center or right (L, C or R) of the highway.

Line 10: Enter the channel bottom slope, S_o, (ft/ft).

Line 11: Enter the channel bottom width, B, (ft).

1	**Lining Type: Vegetation**		
2	**Project ID:**		
3	**Location:**		
4	**Designer/Checker:**		
5	**Date:**		
6			
7	**STA**	**10+00**	**10+00**
8	**Left, Center or Right**	**R**	**R**
9	***Channel/Ditch Geometry***		
10	Channel Slope, S_o (ft/ft)	**0.02**	**0.02**
11	Channel Bottom Width, B (ft)	**0**	**0**
12	Channel Side Slope, z_1	**4**	**4**
13	Channel Side Slope, z_2	**4**	**4**
14	Flow Depth, d (ft) Solve iteratively	**1.00**	**1.21**
15	Safety Factor, SF	**1.0**	**1.0**
16	***Vegetation/Soil Parameters***		
17	Vegetation Retardance Class	**C**	**C**
18	Vegetation Condition	**good**	**good**
19	Vegetation Growth Form	**turf**	**turf**
20	Soil Type	**cohesive**	**cohesive**
21	D_{75} (in) (Set at 0.00 for cohesive soils)		
22	ASTM Soil Class	**SC**	**SC**
23	Plasticity Index, PI	**16**	**16**

Figure 15.1 Grass Lined Channel Design Spreadsheet Data Entry Section

Lines 12-13: Enter the side slopes of the channel (z_1,z_2) (z Horizontal: 1 Vertical)

Line 14: Enter the initial estimate of the water depth, d, in the swale (ft).

Line 15: SF, the selected safety factor.

Line 17: Vegetation Retardance Class (A, B, C, D, or E) selected from a drop-down menu.

Line 18: Vegetation condition (good, fair or poor), selected from a drop-down menu

Line 19: Vegetation growth form (bunched, mixed or turf), selected from a drop-down menu. Select “bunched” for seed mixes 70, 70A, 75 and 80.

Line 20: Soil Type (cohesive or non-cohesive) of the soil you are constructing the swale on, selected from a drop-down menu.

Line 21: If non-cohesive, enter the D_{75} of the soil. Typical values are listed on Table 2. Set this value to zero for cohesive soils.

Line 22: If cohesive, enter ASTM Soil Class (CH, CL, GC, GM, MH, ML, SC, SM) of the soil, selected from a drop-down menu.

Line 23: If cohesive, enter the PI (Plasticity Index) of the soil. This value can be obtained from the soils engineer or from the NRCS website: http://websoilsurvey.nrcs.usda.gov/app/ and begin by pressing the ‘Start WSS’ button. To determine the PI:

1. Draw an outline around the geographic area of interest (AOI) of your project.
2. Select the ‘Soil Data Explorer’ tab.
3. Select the ‘Soil Properties and Qualities’ tab.

4. Select 'Plasticity Index' from the 'Soil Physical Properties' menu.
5. Select the 'Layer Option' you want and enter a depth range, if appropriate.
6. Press the 'View Rating' button to view the plasticity index of your area of interest. Use the value in the Ratings (percent) column of the Plasticity Index Table for the PI value of the soil types in your AOI.

Grass Lined Channel Design Results Summary

24	**Results Summary**		
25	Design Q (ft^3/s)	**15.0**	**15.0**
26	Calculated Q (ft^3/s)	8.4	15.2
27	Difference Between Design & Calc. Flow (%)	-44.0%	1.2%
28	**Stable (Yes or No)**	**YES**	**YES**

Figure 15.2 Grass Lined Channel Design Spreadsheet Results Summary Section

Line 25: Enter the design flow for the channel section, in ft^3/s.

Line 26: The calculated flow for the channel, based upon the depth of flow you entered, the channel geometry, and the grass swale parameters.

Line 27: The percent difference between the design flow and the calculated flow.

Line 28: Channel bottom lining stability status (Yes or No).

Analysis Process:

Once you have entered all relevant information, you must change the channel depth, d, in Line 13 until the difference in the Design Q and the Calculated Q is less than 2%. The spreadsheet will indicate, in Line 27, whether the grass lined channel is stable or unstable.

If the channel is unstable, you have the following options:

1. Flatten the longitudinal slope by either modifying the slope grade or putting in permanent ditch checks that are not in the clear zone. If you use properly spaced permanent ditch checks, assume a longitudinal slope of 1%. Ditch checks should be spaced such that the base of the upstream check is at the same elevation as the top of the downstream check
2. Flatten the side slopes and/or widen the ditch bottom to decrease the flow depth
3. Select a different liner, such as a TRM (Turf Reinforcement Mat) or riprap

Grass Lined Channel Spreadsheet Intermediate Calculations

The intermediate calculations are values determined by the spreadsheet, based upon the procedures described in HEC-15. The user does not enter any of these values into the spreadsheet.

29	**Channel Parameters**		
30	Vegetation Height, h (ft)	0.67	0.67
31	Grass Roughness Coefficient, C_n	0.238	0.238
32	Cover Factor, C_f	0.90	0.90
33	Noncohesive Soil		
34	Soil Grain Roughness, n_s	0.016	0.016
35	Permissible Soil Shear Stress, τ_p (lb/ft^2)	N/A	N/A
36	Cohesive Soil		
37	Porosity, e	0.35	0.35
38	Soil Coefficient 1, c_1	1.0700	1.0700
39	Soil Coefficient 2, c_2	14.30	14.30
40	Soil Coefficient 3, c_3	47.700	47.700
41	Soil Coefficient 4, c_4	1.42	1.42
42	Soil Coefficient 5, c_5	-0.61	-0.61
43	Soil Coefficient 6, c_6	0.00010	0.00010
44	Permissible Soil Shear Stress, τ_p (lb/ft^2)	0.080	0.080
45	Total Permissible Shear Stress, τ_p (lb/ft^2)	0.080	0.080
46	Cross Sectional Area, A (ft^2)	4.000	5.856
47	Wetted Perimeter, P (ft)	8.25	9.98
48	Hydraulic Radius, R (ft)	0.485	0.587
49	Top Width, T (ft)	8.00	9.68
50	Hydraulic Depth, D (ft)	0.500	0.605
51	Froude Number (Q design)	0.523	0.587
52	Channel Shear Stress, τ_o (lb/ft^2)	0.61	0.73
53	Actual Sheer Stress, τ_d (lb/ft^2)	1.25	1.51
54	Mannings n	0.062	0.057
55	Average Velocity, V (ft/s)	3.75	2.56
56	Calculated Flow, Q (ft^3/s)	8.4	15.2
57	Difference Between Design & Calc. Flow (%)	-44.0%	1.2%
58	Effective Shear on Soil Surface, τ_e (lb/ft^2)	0.008	0.012
59	Total Permissible Shear on Veg., $\tau_{p\,veg}$ (lb/ft^2)	12.03	10.17
60	Stable (Y or N)	YES	YES

Figure 15.3 Channel Parameters Section Spreadsheet Data

Channel Parameters:

Line 30: The vegetation height, from Table 1 in this procedure and from HEC-15 Table 4-1.

Line 31: The grass roughness coefficient, from HEC-15, Equation 4.1.

Line 32: The cover factor, from HEC-15, Table 4-5.

Lines 34 - 35: Calculations, described in HEC-15, to determine the permissible soil shear stress for noncohesive (or sandy) soils.

Line 37 - 45: Calculations and coefficients, described in HEC-15, to determine the permissible soil shear stress for cohesive soils.

Line 46: The cross section area, based upon the depth of flow (line 14) and channel geometry.

Line 47: The wetted perimeter, based upon the depth of flow (line 14) and channel geometry.

Lines 48: The hydraulic radius, based upon the depth of flow (line 14) and channel geometry.

Line 48: The top width, based upon the depth of flow (line 14) and channel geometry.

Line 50: The hydraulic depth, which is the area (line 45) divided by the top width (line 48).

Line 51: The Froude number, which is a function of the calculated flow, the cross sectional area, the gravitational constant, g, and the hydraulic depth.

Line 52: The channel shear stress, which is a function of the hydraulic radius (line 47), the channel slope (line 9), and the density of water. This value is used to calculate Manning's n.

Line 53: The actual shear stress, which is a function of the flow depth (line 13), the channel slope (line 9), and the density of water. This value is used to determine the effective shear on the soil, and thus the stability of the channel.

Line 54: The Manning's n value selected by the spreadsheet, which is determined in the Manning's n spreadsheet section below.

Line 55: The average velocity, which is found by dividing the calculated flow (line 25) by the cross section area (line 45).

Line 56: The calculated flow, which is calculated using Manning's equation.

Line 57: The percent difference in flow between the calculated flow and the design flow.

Line 58: The effective shear stress of the grass lining, which is due to the shear force dissipation due to the grass stems and the grass plant stabilization (both root and stem) against turbulent fluctuations.

Line 59: Total permissible shear of the vegetative lining, which includes the combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining.

Line 60: The channel is stable if the permissible shear stress (line 58) is greater than the effective shear stress (line 52) times the safety factor (line 14).

DESIGN EXAMPLE (USING EQUATIONS): RIPRAP CHANNEL (MILD SLOPE)

Design a riprap lining for a trapezoidal channel.

Given:

$Q = 40\ ft^3/s$

B = 0.0 ft (bottom width)

Z = 4

$S_o = 0.02$ ft/ft

Solution:

Step 1. Channel characteristics and design discharge are given above.

Step 2. The WisDOT standard riprap sizes are listed in Table 25.1. Assume that γ_s =165 lb/ft3 for all classes. Try light riprap for the initial trial. D50 = 0.83 ft

Step 3. Assume an initial trial depth, di, of 1.5 ft. Using the geometric properties of a trapezoid, the maximum and average flow depths are found:

$A = Bd+Zd^2 = 0.0(2)+4(2)^2 = 16.0$ ft2

$P = B + 2d\sqrt{Z^2+1} = 0 + 2(2)\sqrt{4^2+1} = 16.5$ ft (wetted perimeter)

R = A/P = 16.0/16.5 = 0.97 ft (hydraulic radius)

T = B+2dZ = 0.0+2(2)(4) = 16.0 ft

d_a = A/T = 16.00/16.0 = 1.00 ft

Step 4. The relative depth ratio, d_a/D_{50} = 1.0/0.83 = 1.20. Therefore, use Equation 15 to calculate Manning's n.

$$b = 1.14\left(\frac{D_{50}}{T}\right)^{0.453}\left(\frac{d_a}{D_{50}}\right)^{0.814} = 1.14\left(\frac{0.83}{16.0}\right)^{0.453}\left(\frac{1.00}{0.83}\right)^{0.814} = 0.347$$

$$f(Fr) = \left(\frac{0.28Fr}{b}\right)^{\log(0.755/b)} = \left(\frac{0.28\left(40/\left(16\sqrt{32.2(1)}\right)\right)}{0.347}\right)^{\log(0.755/0.347)} = 0.705$$

$$f(REG) = 13.434\left(\frac{T}{D_{50}}\right)^{0.492} b^{1.025(T/D_{50})^{0.118}} = 13.434\left(\frac{16.0}{0.83}\right)^{0.492} 0.347^{1.025(16.0/0.83)^{0.118}} = 12.4$$

$$f(CG) = \left(\frac{T}{d_a}\right)^{-b} = \left(\frac{16.0}{1.0}\right)^{-0.347} = 0.382$$

$$n = \frac{1.49d_a^{1/6}}{\sqrt{g}f(Fr)f(REG)f(CG)} = \frac{1.49(1.00)^{1/6}}{\sqrt{32.2}(0.705)(12.4)(0.382)} = 0.079$$

Calculate Q_i using Manning's equation:

$$Q_i = \frac{1.49}{n}AR^{2/3}S^{1/2} = \frac{1.49}{0.079}(16.0)(0.97)^{2/3}(0.02)^{1/2} = 4 \quad ft^3/s$$

Step 5. Since this estimate is more than 2 percent from the design discharge, estimate a new depth in step 3.

Step 3 (2nd iteration). Estimate a new depth:

$$d_{i+1} = d_i\left(\frac{Q}{Q_i}\right)^{0.4} = 2.0\left(\frac{40}{41.8}\right)^{0.4} = 1.96 \quad \text{ft}$$

Using the geometric properties of a trapezoid, the maximum and average flow depths are found:

$A = Bd+Zd^2 = 0.0(2)+4(1.96)^2 = 15.37 \text{ ft}^2$

$P = B + 2d\sqrt{Z^2+1} = 0 + 2(1.96)\sqrt{4^2+1} = 16.16$ ft

$R = A/P = 15.37/16.16 = 0.951$ ft

$T = B+2dZ = 0.0+2(1.96)(4) = 15.7$ ft

$d_a = A/T = 15.37/15.7 = 0.98$ ft

Step 4. (2^{nd} iteration)

$$b = 1.14\left(\frac{D_{50}}{T}\right)^{0.453}\left(\frac{d_a}{D_{50}}\right)^{0.814} = 1.14\left(\frac{0.83}{15.7}\right)^{0.453}\left(\frac{0.98}{0.83}\right)^{0.814} = 0.345$$

$$f(Fr) = \left(\frac{0.28Fr}{b}\right)^{\log(0.755/b)} = \left(\frac{0.28\left(40/\left(16\sqrt{32.2(0.98)}\right)\right)}{0.345}\right)^{\log(0.755/0.345)} = 0.717$$

$$f(REG) = 13.434\left(\frac{T}{D_{50}}\right)^{0.492} b^{1.025(T/D_{50})^{0.118}} = 13.434\left(\frac{15.7}{0.83}\right)^{0.492} 0.345^{1.025(15.7/0.83)^{0.118}} = 12.2$$

$$f(CG) = \left(\frac{T}{d_a}\right)^{-b} = \left(\frac{15.7}{0.98}\right)^{-0.345} = 0.385$$

$$n = \frac{1.49 d_a^{1/6}}{\sqrt{g}f(Fr)f(REG)f(CG)} = \frac{1.49(0.98)^{1/6}}{\sqrt{32.2}(0.717)(12.2)(0.385)} = 0.078$$

Calculate Q_{i+1} using Manning's equation:

$$Q_{i+1} = \frac{1.49}{n}AR^{2/3}S^{1/2} = \frac{1.49}{0.078}(15.37)(0.951)^{2/3}(0.02)^{1/2} = 40.1 \quad \text{ft}^3/\text{s}$$

Step 5 (2nd (iteration). Since this estimate is within 2 percent of the design discharge, proceed to step 6 with the most recently calculated depth.

Step 6. Need to calculate the shear velocity and Reynolds number, Re, to determine Shields' parameter, F*, and SF. Calculate the shear velocity, V*, and the Reynolds number to determine the Shield's parameter and the safety factor, SF.

$$V^* = \sqrt{gdS} = \sqrt{(32.2)(1.96)(0.02)} = 1.12$$

Reynolds $\quad R_e = \frac{V^* D_{50}}{\upsilon} = \frac{1.12(0.83)}{1.127\text{x}10^{-5}} = 7.64\text{x}10^4$ number,

Since the Reynolds number is between $4\text{x}10^4$ and $2\text{x}10^5$, the F* value and the SF are interpolated, as described in FDM 13-30-25, Table 25.2, to get:

F* = 0.071

SF = 1.12

Since channel slope is less than 5 percent, use Equation 8 to calculate minimum stable D_{50}.

$$D_{50} \geq \frac{SFdS_{o}}{F^{*}(SG-1)} = \frac{(1.12)(1.96)(0.02)}{(0.071)(2.65-1)} = 0.4 \text{ ft}$$

Since the D_{50} calculated in step 6 is less than or equal to the trial riprap size of 0.83 ft, which is WisDOT light riprap, then the trial size is acceptable.

As was described by Equation 5 in FDM 13-30-10, the shear stress on the channel side is less than the maximum shear stress occurring on the channel bottom. However, since gravel and riprap linings are noncohesive, as the angle of the side slope approaches the angle of repose (Attachment 25.5) of the channel lining, the lining material becomes less stable. The stability of a side slope lining is a function of the channel side slope and the angle of repose of the riprap. This essentially results in a lower permissible shear stress on the side slope than on the channel bottom. These two counterbalancing effects lead to the design equation described in HEC-15 Section 6.3.2 for specifying a riprap size for the side slope given the riprap size required for a stable channel bottom.

Channels lined with gravel or riprap on side slopes steeper than 3:1 (H:V) may become unstable and should be avoided where feasible. If steeper side slopes are required, they should be assessed using both Equation 6.11 (HEC-15) and Equation 6.8 (HEC-15) in conjunction with Equation 6.15 (HEC-15). Use the larger of the two riprap sizes for the design.

Note that the increased shear stresses created by flow around a bend may produce scour that would not occur in straight channel reaches. To prevent bend scour, it may be necessary to increase the rock riprap size or use a different lining material in the bend. Refer to the section on bend stability for more guidance (FDM 13-30-10).

Riprap Channel (Mild Slope) WisDOT Spreadsheet Worksheet

Download a zipped working copy of the WisDOT Rock Channel Lining spreadsheet from the link at the top of FDM 13-30 Attachment 15.2.

1	**Lining Type: Riprap**								
2	**Project ID:**								
3	**Location:**								
4	**Designer/Checker:**								
5	**Date:**								
6									
7	**STA**	**10+00**	**11+00**	**12+00**	**13+00**	**14+00**	**15+00**	**16+00**	**17+00**
8	**Left, Center, or Right**	**R**	**R**	**R**	**R**	**R**	**R**	**R**	**R**
9	***Channel Geometry***								
10	Channel Slope, S_o (ft/ft)	0.02							
11	Channel Bottom Width, B (ft)	0							
12	Channel Side Slope, z_1	4							
13	Channel Side Slope, z_2	4							
14	Curvature Radius, R_c (ft)	50							
15	Depth of Flow, d (ft) Solve iteratively	2.00							
16	***Riprap Parameters***								
17	Median Riprap Size, D_{50} (ft)	1.04							
18	Riprap Specific Weight, γ_s (lb/ft³)	165							
19	Riprap Angle of Repose, ϕ , (degrees)	41.8							
20	Safety Factor, SF	1.20							
21	Safety Factor, SF (used in calculation)	1.20	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22	***Results Summary***								
23	Design Flow, Q (ft³/s)	40							
24	Calculated Flow, Q (cfs)	37.5	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
25	Difference Between Design & Calc. Flow (%)	-6.1%	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
26	**Bottom Lining Stable (Yes or No)**	Yes	No	No	No	No	No	No	No
27	**Side Lining Stable (Yes or No)**	Yes	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
28	**Bottom in Bend Stable (Yes or No)**	Yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A
29	**Side in Bend Stable (Yes or No)**	Yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A
30	**Downstream Length of Protection (ft)**	7	N/A	N/A	N/A	N/A	N/A	N/A	N/A
31	**Additional Freeboard Required, (ft)**	0.1	N/A	N/A	N/A	N/A	N/A	N/A	N/A
32	***Channel Parameters***								
33	Cross Sectional Area, A (ft²)	16.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
34	Top Width, T (ft)	16.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
35	Average Depth, d_a (ft)	1.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
36	Wetted Perimeter, P (ft)	16.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00
37	Hydraulic Radius, R (ft)	0.970	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
38	Depth to D_{50} Ratio, d_a/D_{50}	1.0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
39	Mannings n	0.088	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
40	Average Velocity, V (ft/s)	2.35	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
41	Calculated Flow, Q (ft³/s)	37.5	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
42	Difference Between Design & Calc. Flow (%)	-6%	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
43	Suggested Trial Depth, d_{i+1} (ft)	2.051	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
44	***Manning's n***								
45	Manning's n (Blodgett)	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
46	Manning's n (Bathurst)	0.088	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
47	Effective Roughness Concentration, b	0.320	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
48	Froude Number, Fr (design Q)	0.441	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
49	Froude Number function, f(Fr)	0.701	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
50	Roughness Element Geometry, f(REG)	10.3	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
51	Channel Geometry Function, f(CG	0.412	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
52	***Bottom Shear***								
53	Shear Velocity, V_* (ft/s)	1.13	0.00	0.00	0.00	0.00	0.00	0.00	0.00
54	Reynolds Number, R_e	9.7E+04	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
55	Shield's Parameter, F*	0.084	0.047	0.047	0.047	0.047	0.047	0.047	0.047
56	Safety Factor, SF	1.18	1.00	1.00	1.00	1.00	1.00	1.00	1.00
57	Maximum Shear Stress, τ_d (lb/ft²)	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00
58	Permissible Shear Stress, S_o<=10%, τ_p (lb/ft²)	9.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
59	Stability Number, η	0.26	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
60	Steepest Channel Side Slope, z	4	0	0	0	0	0	0	0
61	Channel Side Slope Angle, θ (radians)	0.24	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
62	Channel Bottom Slope Angle, α (radians)	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00
63	Riprap Angle of Repose, ϕ , (radians)	0.730	0.000	0.000	0.000	0.000	0.000	0.000	0.000
64	Weight Vector Angle, B (radians)	0.44	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
65	Channel Geometry and Riprap Size Func Δ	0.99	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
66	Permissible Shear Stress, S_o>=5%, τ_p (lb/ft²)	9.0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
67	Permissible Shear based on Slope, τ_p (lb/ft²)	9.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
68	Adjusted Permissible Shear, τ_p/SF (lb/ft²)	7.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
69	Bottom Lining Stable (Yes or No)	Yes	**No**	**No**	**No**	**No**	**No**	**No**	**No**
70	Stable D_{50} (ft)	0.35	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
71	***Side Shear***								
72	Channel Side to Bottom Shear Stress Ratio, K_1	0.93	0.77	0.77	0.77	0.77	0.77	0.77	0.77
73	Channel Side Shear Stress, τ_s (lb/ft²)	2.32	0.00	0.00	0.00	0.00	0.00	0.00	0.00
74	Side Slope Angle, θ (radians)	0.245	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
75	Side Slope Angle, θ (degrees)	14.0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
76	Tractive Force Ratio, K_2	0.93	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
77	Permissible Side Tractive Force, τ_{ps} (lb/ft²)	8.34	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
78	Side Lining Stable (Yes or No)	Yes	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
79	Stable D_{50} (ft)	0.35	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
80	***Bend Shear***								
81	Curvature Radius, R_c (ft)	50	0	0	0	0	0	0	0
82	Ratio of Radius of Curvature to Top Width, R_c/T	3.13	N/A	N/A	N/A	N/A	N/A	N/A	N/A
83	f(Channel Bend and Bottom Shear Stress), K_b	1.81	N/A	N/A	N/A	N/A	N/A	N/A	N/A
84	Shear Stress on the Channel Bottom, τ_b (lb/ft²)	4.51	N/A	N/A	N/A	N/A	N/A	N/A	N/A
85	Bottom in Bend Stable (Yes or No)	Yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A
86	Shear Stress on the Channel Side, τ_{bs} (lb/ft²)	4.19	N/A	N/A	N/A	N/A	N/A	N/A	N/A
87	Side in Bend Stable (Yes or No)	Yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A
88	Downstream Length of Protection, L_p (ft)	7	N/A	N/A	N/A	N/A	N/A	N/A	N/A
89	Addition Freeboard Required, Δd (ft)	0.1	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Instructions and Example Riprap Lined Design Example for Channel Slopes ≤ 20% Using the WisDOT Spreadsheet

WisDOT has prepared a spreadsheet that incorporates all the design guidelines and equations described in HEC-15 to design rock lined channels. The spreadsheet is divided into three sections:

1. a data entry section,
2. a results section, and
3. an intermediate calculations section.

Each of these sections is described within the line-by-line instructions below.

Riprap Spreadsheet Data Entry, Slopes ≤ 20%:

Enter data to analyze channel sections in the grey or blue cells of the spreadsheet, as described below.

Lines 2 - 5: Enter the project information as described.

Lines 7 - 8: For each channel segment, enter the highway station and whether the channel is on the left, center, or right (L, C or R) of the highway.

Line 10: Enter the channel bottom slope (ft/ft).

Line 11: Enter the channel bottom width, B (ft).

Lines 12-13: Enter the side slopes of the channel (z Horizontal: 1 Vertical)

1	**Lining Type: Riprap**		
2	**Project ID:**		
3	**Location:**		
4	**Designer/Checker:**		
5	**Date:**		
6			
7	**STA**	**10+00**	**10+00**
8	**Left, Center, or Right**	**R**	**R**
9	***Channel Geometry***		
10	Channel Slope, S_o (ft/ft)	0.02	0.02
11	Channel Bottom Width, B (ft)	0	0
12	Channel Side Slope, z_1	4	4
13	Channel Side Slope, z_2	4	4
14	Curvature Radius, R_c (ft)	50	50
15	Depth of Flow, d (ft) Solve iteratively	2.00	1.96
16	***Riprap Parameters***		
17	Median Riprap Size, D_{50} (ft)	1.04	0.83
18	Riprap Specific Weight, γ_s (lb/ft^3)	165	165
19	Riprap Angle of Repose, ϕ , (degrees)	41.8	41.8
20	Safety Factor, SF	1.20	1.20
21	Safety Factor, SF (used in calculation)	1.20	1.20

Figure 25.1 Riprap Lined Channel Design Spreadsheet Data Entry Section

Line 14: Enter the radius of curvature of the bend R_c, if there is one, to the channel centerline (ft).

Line 15: Your initial estimate of the water depth in the channel, d (ft).

Line 17: D_{50}, your initial estimate of the median size of the riprap (ft). Start with the smallest reasonable riprap size to prevent riprap over sizing. Use the drop-down menu to select the standard WisDOT riprap size you want to analyze.

Line 18: The specific weight of the rock, γ_s, (lbs/ft^3).

Line 19: The angle of repose for the rock (degrees). Find this value from Figure 8, Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone.

Line 20: SF, the safety factor you select to design the channel. The spreadsheet will also calculate a safety factor, and then use whichever value is higher (Line 21).

Riprap Spreadsheet Results Summary, Slopes ≤ 20%:

22	**Results Summary**		
23	Design Flow, Q (ft^3/s)	40	40
24	Calculated Flow, Q (cfs)	37.5	40.1
25	Difference Between Design & Calc. Flow (%)	-6.1%	0.3%
26	**Bottom Lining Stable (Yes or No)**	Yes	Yes
27	**Side Lining Stable (Yes or No)**	Yes	Yes
28	**Bottom in Bend Stable (Yes or No)**	Yes	Yes
29	**Side in Bend Stable (Yes or No)**	Yes	Yes
30	**Downstream Length of Protection (ft)**	7	7
31	**Additional Freeboard Required, (ft)**	0.1	0.1

Figure 25.2 Riprap Lined Channel Design Spreadsheet Data Entry Section

Line 23 Enter the design flow for the channel section (ft^3/s).

Line 24 The calculated flow for the channel, based upon the depth of flow you entered, the channel geometry, and the riprap parameters.

Line 25: The percent difference between the design flow and the calculated flow.

Line 26: Channel bottom lining stability status (Yes or No).

Line 27: Channel side lining stability status (Yes or No).

Line 28: Channel bottom lining stability status for channel bend sections (Yes, No or N/A if the section is a straight section).

Line 29: Channel side lining stability status for channel bend sections (Yes, No or N/A if the section is a straight section).

Line 30: The length of channel downstream of a bend that requires protection, in feet. (N/A if the section is a straight section)

Line 31: Additional freeboard required due to elevated flows in a channel bend, in feet. (N/A if the section is a straight section)

Analysis Process:

Once you have entered all the relevant information, you must change the channel depth, d, in Line 14 until the difference in the Design Q (line 22), which you must enter, and the Calculated Q (line 23), is less than 2%. The spreadsheet will indicate whether the riprap lined channel is stable or unstable for straight channels (bottom and sides) and channel bend bottom and sides. If it is unstable, you have the following options:

1. Flatten the longitudinal slope by either modifying the slope grade or putting in permanent ditch checks that are not in the clear zone. If you use properly spaced permanent ditch checks, assume a longitudinal slope of 1%. Ditch checks should be spaced such that the base of the upstream check is at the same elevation as the top of the downstream check
2. Flatten the side slopes and/or widen the ditch bottom to decrease the flow depth
3. Select a different riprap size.

Riprap Spreadsheet Intermediate Calculations, Slopes ≤ 20%:

To determine the stability of a riprap channel liner, the spreadsheet makes a number of calculations, based upon Chapter 6 of HEC-15, related to channel geometry, Manning's n, bottom shear, side shear and bend shear. Each of these types of calculations is described below. For a complete description, review Chapters 3 and 6 of HEC-15.

Channel Parameters:

32	**Channel Parameters**		
33	Cross Sectional Area, A (ft^2)	16.00	15.37
34	Top Width, T (ft)	16.0	15.7
35	Average Depth, d_a (ft)	1.000	0.980
36	Wetted Perimeter, P (ft)	16.49	16.16
37	Hydraulic Radius, R (ft)	0.970	0.951
38	Depth to D_{50} Ratio, d_a/D_{50}	1.0	1.2
39	Mannings n	0.088	0.078
40	Average Velocity, V (ft/s)	2.35	2.61
41	Calculated Flow, Q (ft^3/s)	37.5	40.1
42	Difference Between Design & Calc. Flow (%)	-6%	0%
43	Suggested Trial Depth, d_{i+1} (ft)	2.051	1.957

Figure 25.3 Riprap Lined Channel Design Spreadsheet Data Channel Parameter Section

Line 33: The cross section area, A, based upon the depth of flow (line 14) and channel geometry.

Line 34: The top width, T, based upon the depth of flow (line 15) and channel geometry.

Line 35: The average depth, d_a, which is the area (line 33) divided by the top width (line 34).

Line 36: The wetted perimeter, P, based upon the depth of flow (line 15) and channel geometry.

Line 37: The hydraulic radius, R, based upon the depth of flow (line 15) and channel geometry.

Line 38: The ratio of average depth to riprap D_{50}, used to determine which Manning's n equation is appropriate for the channel.

Line 39: The Manning's n value selected by the spreadsheet, which is determined in the Manning's n spreadsheet section below.

Line 40: The average velocity, which is calculated using Manning's equation.

Line 41: The calculated flow, which is the product of the velocity (line 40) and area (line 33).

Line 42: The percent difference in flow between the calculated flow and the design flow.

Line 43: The suggested trial depth, d_{i+1}, to aid in the iterative solution process. The user can enter this value into line 15, or just enter his or her best guesses of the depth until the calculated and design flows are within 2% of each other.

Manning's n:

44	**Manning's n**		
45	Manning's n (Blodgett)	0.000	0.000
46	Manning's n (Bathurst)	0.088	0.078
47	Effective Roughness Concentration, b	0.320	0.345
48	Froude Number, Fr (design Q)	0.441	0.463
49	Froude Number function, f(Fr)	0.701	0.717
50	Roughness Element Geometry, f(REG)	10.3	12.2
51	Channel Geometry Function, f(CG)	0.412	0.385

Figure 25.4 Riprap Lined Channel Design Spreadsheet Data Manning's n Section

Line 45: Manning's n calculated using the Blodgett equation.

Line 46: Manning's n calculated using the Bathurst equation.

Line 47: The effectiveness roughness concentration, which describes the relationship between effective roughness concentration and relative submergence of the roughness bed (HEC-15, pg. 6-2)

Line 48: The Froude number, which is channel velocity divided by the square root of the product of the gravity constant (32.2 ft/s^2) and the average channel depth.

Line 49 - 51: Functions of the Froude number, channel roughness and channel geometry used to calculate the Bathurst n value (line 46).

Bottom Shear:

52	***Bottom Shear***		
53	Shear Velocity, V_* (ft/s)	1.13	1.12
54	Reynolds Number, R_e	9.7E+04	7.7E+04
55	Shield's Parameter, F*	0.084	0.071
56	Safety Factor, SF	1.18	1.12
57	Maximum Shear Stress, τ_d (lb/ft^2)	2.50	2.45
58	Permissible Shear Stress, S_o<=10%, τ_p (lb/ft^2)	9.0	6.05
59	Stability Number, η	0.26	0.38
60	Steepest Channel Side Slope, z	4	4
61	Channel Side Slope Angle, θ (radians)	0.24	0.24
62	Channel Bottom Slope Angle, α (radians)	0.02	0.02
63	Riprap Angle of Repose, ϕ , (radians)	0.730	0.730
64	Weight Vector Angle, B (radians)	0.44	0.60
65	Channel Geometry and Riprap Size Func, Δ	0.99	1.05
66	Permissible Shear Stress, S_o>=5%, τ_p (lb/ft^2)	9.0	5.77
67	Permissible Shear based on Slope, τ_p (lb/ft^2)	9.0	6.0
68	Adjusted Permissible Shear, τ_p/SF (lb/ft^2)	7.5	5.0
69	Bottom Lining Stable (Yes or No)	Yes	Yes
70	Stable D_{50} (ft)	0.35	0.40

Figure 25.5 Riprap Lined Channel Design Spreadsheet Data Bottom Shear Section

Line 53 - 68: These are values and equations used to determine the actual shear stress on the channel bottom and the permissible shear stress for the channel bottom. The process is described in detail in HEC-15, pages 6-3 to 6-5. The spreadsheet compares the two values, and if the actual shear stress is less than the permissible shear stress, then the channel is considered stable (line 69).

Line 70: The riprap D_{50} that will provide a stable straight bottom channel section for the given design flow

Side Shear:

71	***Side Shear***		
72	Channel Side to Bottom Shear Stress Ratio, K_1	0.93	0.93
73	Channel Side Shear Stress, τ_s (lb/ft^2)	2.32	2.27
74	Side Slope Angle, θ (radians)	0.245	0.245
75	Side Slope Angle, θ (degrees)	14.0	14.0
76	Tractive Force Ratio, K_2	0.93	0.93
77	Permissible Side Tractive Force, τ_{ps} (lb/ft^2)	8.34	5.62
78	Side Lining Stable (Yes or No)	Yes	Yes
79	Stable D_{50} (ft)	0.35	0.40

Figure 25.6 Riprap Lined Channel Design Spreadsheet Side Shear Section

Line 72 - 77: These are values and equations used to determine the actual shear stress on the channel side and the permissible shear stress for the channel side. The process is described in detail in HEC-15, pages 6-10 to 6-11.The spreadsheet compares the two values, and if the actual shear stress is less than the permissible shear stress, then the channel is considered stable (line 78).

Line 79: The riprap D_{50} that will provide a stable straight side channel section for the given design flow

Bend Shear:

80	**Bend Shear**		
81	Curvature Radius, R_c (ft)	50	50
82	Ratio of Radius of Curvature to Top Width, R_c/T	3.13	3.19
83	f(Channel Bend and Bottom Shear Stress), K_b	1.81	1.80
84	Shear Stress on the Channel Bottom, τ_b (lb/ft^2)	4.51	4.40
85	Bottom in Bend Stable (Yes or No)	Yes	Yes
86	Shear Stress on the Channel Side, τ_{bs} (lb/ft^2)	4.19	4.09
87	Side in Bend Stable (Yes or No)	Yes	Yes
88	Downstream Length of Protection, L_p (ft)	7	7
89	Addition Freeboard Required, Δd (ft)	0.1	0.1

Figure 25.7 Riprap Lined Channel Design Spreadsheet Data Bend Shear Section

Line 81 - 87: These are values and equations used to determine the actual shear stress on the channel bottom and side, and indicate if the bottom of the bending channel is stable (line 85) and if the side of the bending channel is stable (line 87). The process is described in detail in HEC-15 section 3.4, pages 3-12 to 3-16.

Line 88: This equation determines the length downstream of the end of the channel bend that channel protection will need to be extended to resist bend stresses. See HEC-15, page 3-13.

Line 89: This equation determines the increase in the water surface elevation at the outside of the bend caused by the superelevation of the water surface. See HEC-15, page 3-13.

Example Problem:

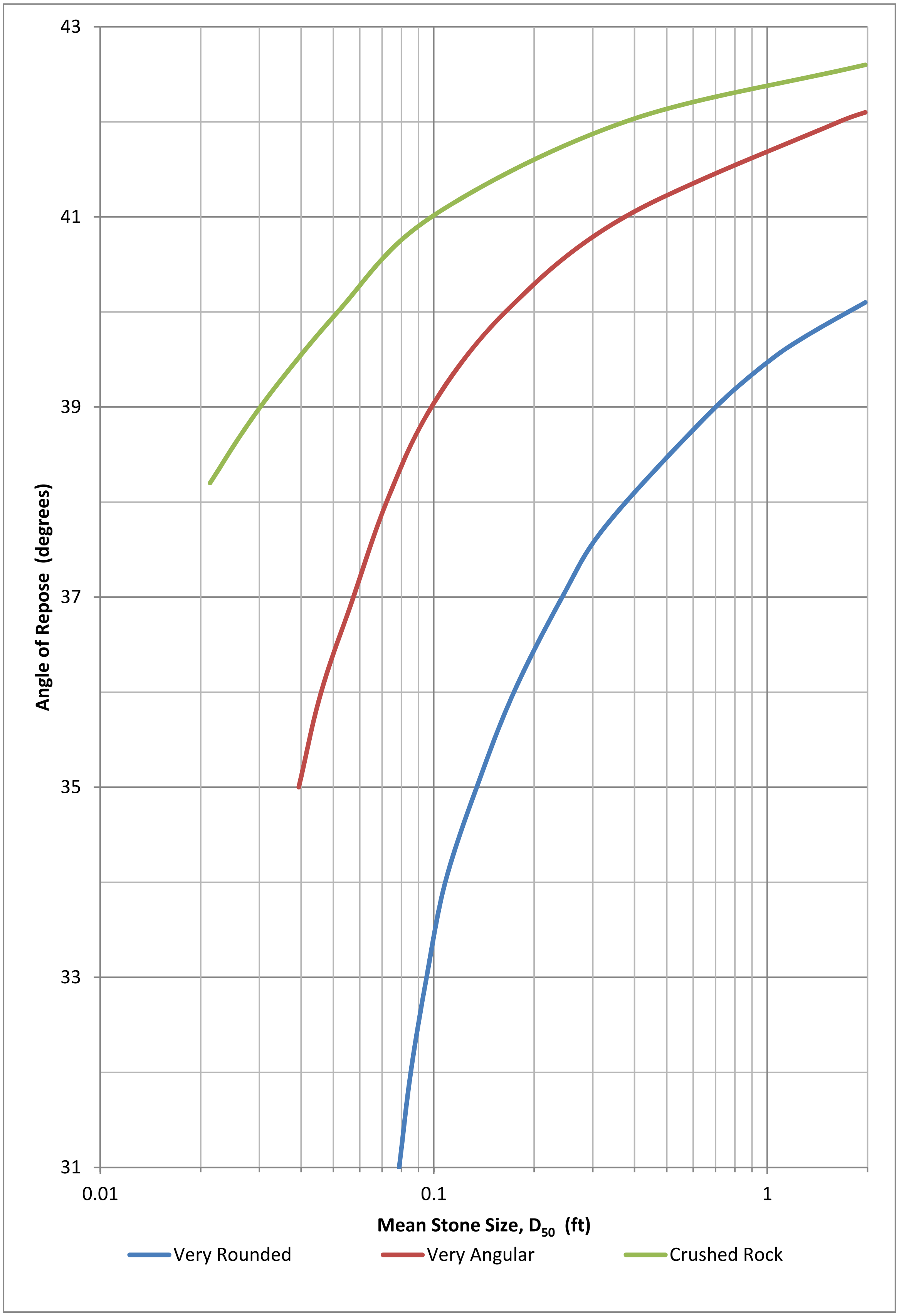
The following example illustrates the design procedure. The data for this example has been entered in the instructions listed above for this attachment. The second column of this example spreadsheet shown in the instructions describes the initial trial and the third column describes the final result of the design procedure. Note that the third column was added to demonstrate how this design spreadsheet works - designers do not need to duplicate each channel section in a set of columns to demonstrate the iterative process.

For the example problem assume the following:

Design Q =	40 cfs
Channel Slope S_o	2 %
Channel Bottom Width =	0 ft
Channel Side Slopes =	4:1
Curvature Radius =	0 ft
Specific Weight of the riprap =	165 lb/ft^3
Riprap angle of Repose =	41.8
SF =	1.2
Curvature Radius =	50 ft.

Enter the data into the second column of the spreadsheet and select light riprap as a first guess. Select the depth of water cell in row 15, and enter your first guess at the water depth in the channel. Note the Calculated Q (ft^3/s) and percent difference between calculated and design flow in rows 24 and 25. Change the depth in row 15 until the percent difference is less than 2%. The cell in row 25 will become green.

If the spreadsheet indicates that, per rows 26 and 27, the bottom and side linings are stable, then you are done unless you want to try a smaller riprap size. In this example, the bottom and sides are not stable after the first trial depth. To correct this, the depth in the last column of this example was changed until the percent difference between the design flow and calculated flow was less than 2%. Rows 26 and 27 indicate that the channel is stable, so the appropriate riprap size for this section is Light Riprap, with a D_{50} of 0.83 ft.



Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone

Note: Use this map as a guide. Check with Soils Engineers or experienced Project Managers or Engineers to determine if angular riprap will be difficult to obtain for a project in a given area.

Rock Chute Design Spreadsheets

Download a zipped working copy of the spreadsheets from the link at the top of FDM 13-30 Attachment 15.2.

Rock Chute Design Data

(Version WI-April-2005, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)
Revised for WisDOT 9/2010

Project: Sample project **County:** Brown
Designer: jgv **Checked by:**
Date: January 13, 2009 **Date:**

Input Geometry:

→ Upstream Channel		→ Chute			→ Downstream Channel	
Bottom Width =	4.0 ft.	Bottom Width =	4.0 ft.		Bottom Width =	4.0 ft.
Side slopes =	3.0 (m:1)	Factor of safety =	1.20 (SF)	1.2 Min	Side slopes =	3.0 (m:1)
Mannings n value =	0.040	Side slopes =	3.0 (z:1)	→ 2.0:1 max.	Mannings n value =	0.040
Bed slope =	0.0100 ft./ft.	Bed slope =	0.2000 ft./ft.	→ 3.0:1 max.	Bed slope =	0.0100 ft./ft.
Note: Use procedures 13-30-15 or 13-30-25 for upstream and downstream Mannings n		Freeboard =	1.0 ft.			
		Outlet apron depth, d =	1.0 ft.		Base flow =	0.0 cfs

Flow and Elevation Data:

Apron elev. --- Inlet = 100.0 ft. ------ Outlet 91.0 ft. --- (H_{drop} = 8 ft.)

Degree of angularity = 1
1 --> 50% angular, 50% rounded
2 --> 100 % rounded

Q_{high} = Runoff from design storm
Q_5 = Runoff from a 5-year,24-hour storm

Q_{high} = 50.0 cfs High flow storm through chute → Tw (ft.) = Program
Q_{low} = 30.0 cfs Low flow storm through chute → Tw (ft.) = Program

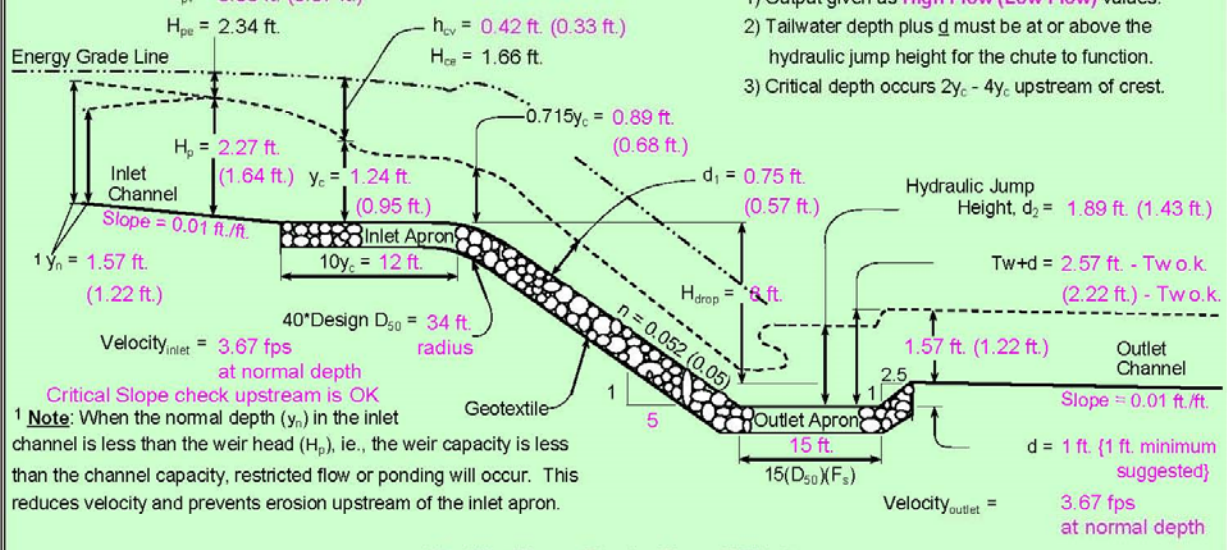
Note: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.

Input tailwater (Tw):

Profile and Cross Section (Output):

Starting Station = 3+00.0

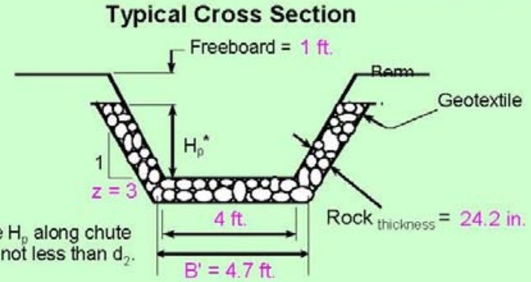
Notes:
1) Output given as **High Flow (Low Flow)** values.
2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.
3) Critical depth occurs $2y_c$ - $4y_c$ upstream of crest.



[1] **Note**: When the normal depth (y_n) in the inlet channel is less than the weir head (H_p), ie., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

Profile Along Centerline of Chute

Typical Cross Section



* Use H_p along chute but not less than d_2.

	7.87 cfs/ft.	Equivalent unit discharge
SF =	1.20	Factor of safety (multiplier)
d_1 =	0.75 ft.	Normal depth in chute
n-value =	0.052	Manning's roughness coefficient
D_{50}(SF) =	12.1 in.	Minimum Design D_{50}*
2(D_{50})(SF) =	24.2 in.	Rock chute thickness
Tw + d =	2.57 ft.	Tailwater above outlet apron
d_2 =	1.89 ft.	Hydraulic jump height
*** The outlet	will	function adequately

High Flow Storm Information

Rock Chute Design - Plan Sheet

(Version WI-April-2005, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)
Revised for WisDOT 9/2010

Project:	Sample project	**County:**	Brown
Designer:	jgv	**Checked by:**	
Date:	1/13/2009	**Date:**	

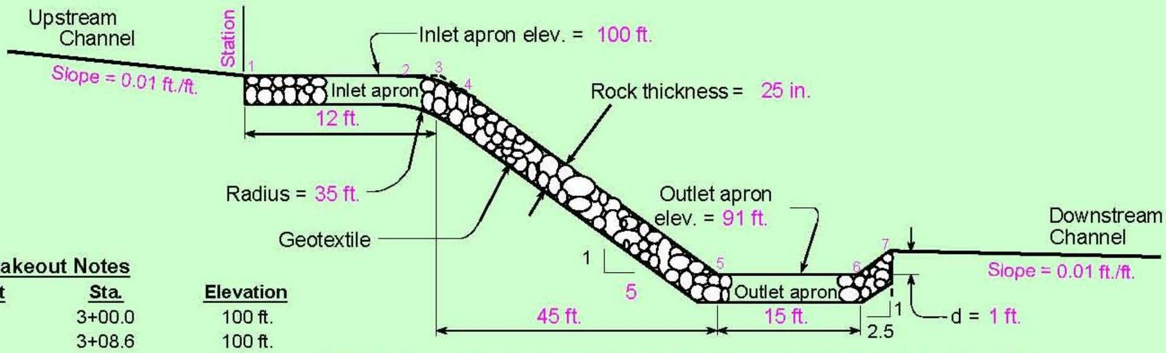
Design Values		Plan Values	
12.1 in.	D_{50} dia. =	12.5	in.
24 in.	Rock chute thickness =	25	in.
12 ft.	Inlet apron length =	12	ft.
15 ft.	Outlet apron length =	15	ft.
34 ft.	Radius =	35	ft.

Quantities[a]

Riprap = 148 yd^3

Geotextile = 271 yd^2

Notes: *[a] Rock and geotextile quantities are determined from the x-section below (neglect radius).*



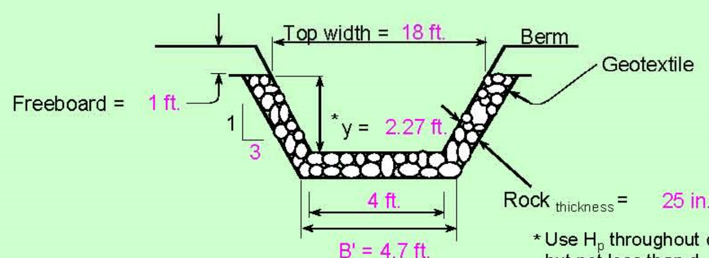
Profile Along Centerline of Rock Chute

Stakeout Notes

Point	Sta.	Elevation
(1)	3+00.0	100 ft.
(2)	3+08.6	100 ft.
(3)	3+12.0	99.8 ft.
(4)	3+15.4	99.3 ft.
(5)	3+57.0	91 ft.
(6)	3+72.0	91 ft.
(7)	3+74.5	92 ft.

Note: The outlet will function adequately

Rock gradation envelope can be met with DOT Medium Riprap Gradation



* Use H_p throughout chute but not less than d_2.

Rock Chute Cross Section

Profile, Cross Sections, and Quantities

	Sample project Brown County	Date Designed jgv Drawn Checked Approved	File Name Drawing Name Sheet __ of __

Rock_Chute.xls
for construction plan

Rock Chute Design Construction Detail

(Version WI-April-2005, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)
Revised for WisDOT 9/2010

Project: Sample project **County:** Brown
Designer: jgv **Checked by:**
Date: 1/13/2009 **Date:**

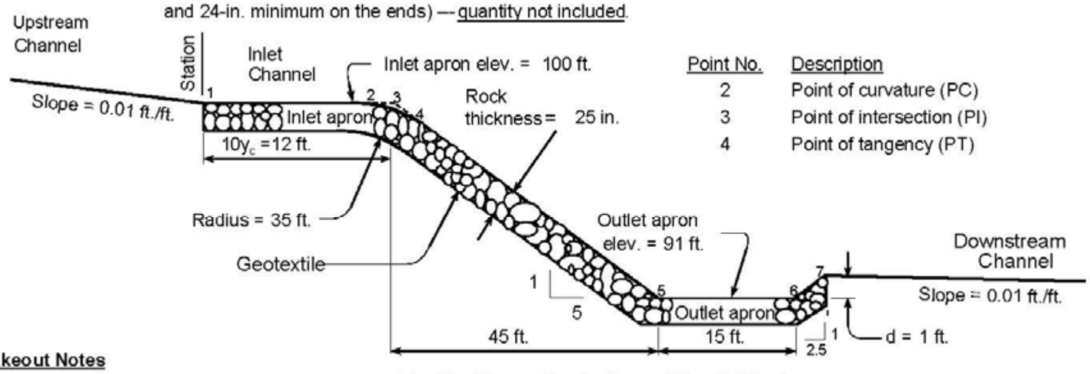
Design Values

D_{50} dia. = 12.5 in.
$Rock_{chute}$ thickness = 25.0 in.
Inlet apron length = 12 ft.
Outlet apron length = 15 ft.
Radius = 35 ft.

Quantities[a]

Rock = 148 yd^3
Geotextile (WCS-13)[b] = 271 yd^2

Notes: [a] Rock and geotextile quantities are determined from x-section below (neglect radius).
[b] Geotextile shall be overlapped (18-in. minimum) and anchored (18-in. minimum along sides and 24-in. minimum on the ends) — quantity not included.



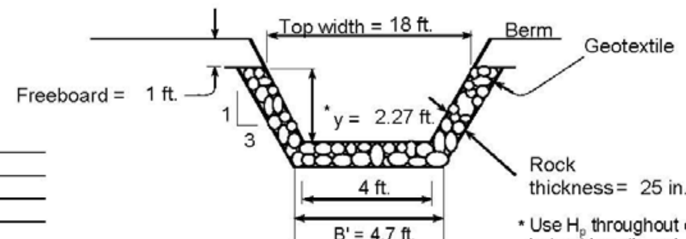
Point No.	Description
2	Point of curvature (PC)
3	Point of intersection (PI)
4	Point of tangency (PT)

Profile Along Centerline of Rock Chute

Stakeout Notes

Point	Sta.	Elevation
(1)	3+00.0	100 ft.
(2)	3+08.6	100 ft.
(3)	3+12.0	99.8 ft.
(4)	3+15.4	99.3 ft.
(5)	3+57.0	91 ft.
(6)	3+72.0	91 ft.
(7)	3+74.5	92 ft.

Notes:
Rock gradation envelope can be met with
DOT Medium Riprap Gradation



* Use H_p throughout chute but not less than d_2.

Rock Chute Cross Section

Profile, Cross Sections, and Quantities

		Date	
Sample project Brown County	Designed jgv		File Name
	Drawn		Drawing Name
	Checked		Sheet _ of _
	Approved		

FDM 13-35-1 Special Hydraulic Structures

August 8, 1997

1.1 Introduction

Various structures are used for special hydraulic purposes. These structures can retard a flow rate, reduce the velocity, clean the water, stop the flow of water, or improve the flow of water through a structure. Descriptions of some commonly used structures follow.

1.2 Flow Control Gates

Flap gates and sluice gates are normally used as flood protection devices at the outlets of drainage structures to prevent floodwater from backing into the structure. The flow of a culvert normally is unimpeded during low flow conditions in a river. However, the water in the river closes the flap gate by exerting pressure against it when the river is flowing at flood stage. This prevents the floodwaters from backing through the culvert and flooding low-lying property upstream from the inlet. The flow rate contributed to a structure from its drainage area is usually small when there is high water in the river because of the lag in time of concentration of the river compared to the small culvert. The need for a pumping station for tributary floods may have to be considered, but the provision for pumping must be fully justified.

1.3 Debris Control Structures

When debris accumulates at the inlet of a highway drainage structure, the structure normally ceases performing its design functions, resulting in damage to the structure, to the highway, to adjacent property, and to the traveling public. A presentation on debris control structures can be found in the FHWA publication HEC #9, entitled "Debris Control Structures." The need for such a structure is normally evident by a field review of the drainage area and its debris problems. HEC #9 contains a discussion of the types of debris control structures, the classification of debris, and the field survey data required to determine the need for a debris control structure.

One of the essential design conditions for a debris control structure is to allow for planned maintenance rather than emergency maintenance during flood period. If the proposed schedule of planned maintenance is not followed, emergency maintenance again will become the normal operation, not the exception. A further discussion on the maintenance of debris control structures is given in H.E.C. #9, Chapter 7.

1.4 Detention Basin

The detention basin is not to be confused with the retention basin. The detention basin is used only for the purpose of increasing the time of concentration of water flow to any point of discharge. A detention basin, unlike a retention basin, does not hold water for infiltration into the subsoil but is self-draining and after a period of time will be completely free of water.

A special control may exist within a drainage area that will affect the flow rate through a structure. The greatest effect is caused when a storage area is immediately upstream from the proposed drainage structure. When a detention basin is placed immediately above a drainage structure, it accomplishes the same effect as flow rate control. As a result of the field review of the drainage area, it may be determined that it is not feasible to increase the flow rate downstream from the proposed highway crossing. This will require consideration of constructing an artificial upstream control, such as a detention basin.

The detention basin can be used only when there is enough land available to assure that adjacent land will not be flooded or damaged by the backup water. Areas that can be used for detention purposes are the interchanges and wide medians of divided highways. The design of a detention basin can be accomplished by utilizing FDM 13-10-10, "Hydrograph Developing, and Routing."

1.5 Temporary Sediment Structures

A temporary sediment structure consists of a basin constructed at a suitable location to trap and store sediment. The Soil Conservation Service classifies temporary sediment structures into the following two major types:

1. Sediment Trap: A small storage area formed by excavation and/or an embankment, with further classification according to outlet or inlet type. It is limited to drainage areas of five acres or less.
2. Sediment Basin: A dam created to temporarily impound water with or without an excavated storage area. It is limited to drainage areas of 150 acres or less.

Temporary sediment structures trap and retain sediment from erodible areas in order to protect properties and stream channels below the installation from excessive siltation. These structures trap and store sediment that is generated in spite of the use of temporary erosion control measures. See Chapter 10 of the FDM for further information on sediment traps and basins.

Formal design information may be obtained from a publication entitled "Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas," U.S. Department of Agriculture, Soil Conservation Service, College Park, Maryland, July 1975. Also, additional general information may be obtained from "Model Drainage Manual 1991," AASHTO.

FDM 13-35-5 Energy Dissipaters

August 8, 1997

5.1 Introduction

Energy dissipating devices are used where it is desirable to reduce the discharge velocity by inducing high-energy losses at the inlet or discharge ends of the structure. They are generally warranted when discharge velocities exceed 14 feet per second.

Energy losses may be induced at the inlet by a drop inlet type of culvert or at the outlet by formation of a hydraulic jump. Drop inlets are used where headroom is limited, and outlet dissipators are used where headroom is not critical. In addition, energy losses can be induced by stilling basins, impact basins, riprap basins, riprap blankets, stilling wells, tumbling flow in culverts, increased culvert resistance, etc.

The energy dissipater(s) that will work for a specific drainage structure may be determined with the aid of Attachment 5.1, which lists several types of dissipaters with their hydraulic limitations. For schematic depictions and design procedures of some of the dissipaters listed in Attachment 5.1, see HEC #14, Hydraulic Design of Energy Dissipaters for Culverts and Channels, Wisconsin Department of Transportation, Federal Highway Administration, July 2006. The Bridge Design Manual also contains procedures and sample problems for a drop inlet, drop outlet, and chute spillway. Moreover, design procedures and sample problems are given in this procedure for a riprap blanket and outlet expansion.

5.2 Riprap Blanket

Attachment 5.2 gives the recommended configuration of a riprap blanket, subject to tail waters. The maximum tail water condition is when the tail water is equal to or greater than half the culvert diameter, whereas the minimum tail water condition is when the tail water is less than half the culvert diameter. The average size of stone and configuration of a horizontal blanket of riprap at a culvert outlet required to control localized downstream scour can be estimated by the following relationships:

$$d_{50}/D_o = 0.020\ (D_o/TW)\ (Q/D_o^{5/2})^{4/3}$$

$$L_{sp}/D_o = 1.7\ (Q/D_o^{5/2}) + 8$$

Where:

D_o = diameter of width of culvert (feet)

d_{50} = diameter of average size stone (feet)

L_{sp} = length of stone protection (feet)

Q = discharge (cfs)

TW = tail water depth above invert of culvert (feet)

The width of stone protection W_{sp} can be estimated by the proportions shown in Attachment 5.2.

Example Problem

Given: Culvert Diameter = 6 feet

Tail Water Depth = 3.5 feet

Discharge = 425 cfs

Find: The average size of stone and configuration of a horizontal blanket of riprap to control erosion.

Solution:

1. Stone size:

$$d_{50}/D_o = 0.020\ (D_o/TW)\ (Q/D_o^{5/2})^{4/3}$$

$d_{50}/6 = 0.020\ (6/3.5)\ (425/6^{5/2})^{4/3}$

$d_{50} = 1.67$ feet

2.Length of stone protection:

$L_{sp}/D_o = 1.7\ (Q/D_o^{5/2}) + 8$

$L_{sp}/D_o = 1.7\ (425/6^{5/2}) + 8$

$L_{sp} = 97$ feet

3.Width of stone protection:

W_{sp} (max. TW) $= 2\ (1.5\ D_o + 0.2\ L_{sp})$

$= 2\ 1.5\ (6) + 0.2\ (97)$

$= 57$ feet

5.3 Lined Channel Expansions

Channel expansions downstream of culvert outlets can also be lined with either sack revetment, cellular blocks, or rock riprap. Attachment 5.3 shows a practical configuration of the channel expansion geometry.

The required size of riprap, sack revetment, or cellular block can be determined by the following empirical formula:

d_{50}/D_o

or ù

T_s/D_o _ $= 0.016(D_o/TW)(Q/D_o^{5/2})^{4/3}$

or

T_B/D_o û

Where:

- T_s = thickness of sack revetment (feet)
- T_B = thickness of cellular block (feet)

The other terms are as defined for a riprap blanket.

The dimensions of the lined channel expansion can be determined by the proportions in Attachment 5.3.

Example Problem

Given: Culvert Diameter = 6 feet

Tail Water Depth = 3.5 feet

Discharge = 425 cfs

Find: The average size of stone and dimensions of a lined channel expansion.

Solution:

1.Stone size:

$d_{50}/D_o = 0.016\ (D_o/TW)\ (Q/D_o^{5/2})^{4/3}$

$d_{50}/6 = 0.016\ (6/3.5)\ 425/(6)^{5/2})^{4/3}$

$d_{50} = 1.34$ feet

2.Use Attachment 5.3 to determine the lined channel expansion dimensions. Refer to Attachment 5.4 for the final design dimensions of the lined channel expansion.

For further details on the design of riprap blankets, preformed scour holes, and lined channel expansions, the designer is referred to Miscellaneous Paper H-72-5, "Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets," published by the U. S. Army Engineer Waterways Experiment Station in May 1972.

5.4 Oulet Expansion

At many culvert installations the designer can specify a maximum outlet velocity to minimize the possibility of scour and erosion at the outlet. These velocities may range from six to 10 fps, which is low enough that the scour at the outlet may be controlled by riprap or sod. Attachment 5.5 shows a typical outlet expansion design. As shown by this figure, the tail water must be at or above the crown of the pipe at design flow. This condition is fairly common in flat land drainage problems or in multi-pipe installations where smaller diameters are used because of limited headroom.

The design procedure to be outlined here was constructed from an American Concrete Pipe Association article published in Concrete Pipe News entitled "Culvert Velocity Reduction with an Outlet Expansion."

Procedure

Refer to Attachment 5.5 for a schematic diagram of the outlet expansion.

1. With the pipe flowing full under conditions of outlet control, the required pipe size is determined from the Continuity Equation, as follows:

   $D_2 = (4\ Q/pV_2)^{1/2}$

   Where:

   D_2 = pipe diameter (feet)

   Q = design discharge (cfs)

   V_2 = maximum allowable pipe velocity (fps)

2. This type of design produces a net head gain or a lowering of the pipe headwater, as given by the following formula:

   $hg = V_2\ (V_1 - V_2)/g$

   Where:

   hg = net head gain

   V_1 = velocity in initial pipe segment (fps)

   V_2 = velocity in expanded pipe segment (fps)

3. Determine the required length of the expanded pipe segment according to Attachment 5.6, Detail A. For a value of D_1/D_2 different than those listed, use Detail "B" of Attachment 5.6 to obtain a pro-rated value for L/D_L where L is the required length of the expanded pipe.

Example Problem

Given: Culvert Diameter = 6 feet

Tail Water Depth = 7 feet

Discharge = 400 cfs

Find: The diameter and length of the pipe expansion required to lower the discharge velocity to 10 fps.

Solution:

1. Expansion pipe diameter:

   $D_2 = (4\ Q/pV_2)^{1/2}$

   $D_2 = (4 \times 400/p\times 10)^{1/2} = 7.1$ feet

   Use D_2 = 7 feet = 84 inches

2. Determine head gain:

   $hg = V_2\ (V_1 - V_2)/g$

   $V_1 = Q/pr_1^2$

   $= 400$ cfs/p $(3)^2$

   = 14.2 fps

   $V_2 = Q/pr_2^2 = 10.4$ fps

$hg = 10.4 \times (14.2 - 10.4)/32.2 = 1.23$ feet

3. Expansion pipe length:

$D_1/D_2 = 6/7 = 0.857$

Use Attachment 5.6, Detail B, to obtain the length for 100 percent complete expansion.

$L/D_1 = 2.1$

$L = 2.1\ D_1 = 2.1\ (6) = 12.6$ feet

Use L = 14 feet.

5.5 Literature on Energy Dissipaters

Design of Small Dams, U.S. Bureau of Reclamation, lst Edition, 1961, Chapter VIII, "Spillways," Washington, D.C.

Rouse, Hunter, Engineering Hydraulics, John Wiley & Sons, New York, 1949.

Blaisdell, Fred W., and Donnelly, Charles A., Hydraulic Design of the Box Inlet Drop Spillway, U.S. Department of Agriculture, Soil Conservation Service, SCS-TP-106, July 1951.

Blaisdell, Fred W., and Donnelly, Charles A., Straight Drop Spillway Stilling Basin, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, November 1954.

Handbook of Concrete Culvert Pipe Hydraulics, Portland Cement Association, 1964.

Technical Release No. 49, Criteria for the Hydraulic Design of Impact Basins Associated with Full Flow in Pipe Conduit, U.S. Department of Agriculture, Soil Conservation Service, March 1971.

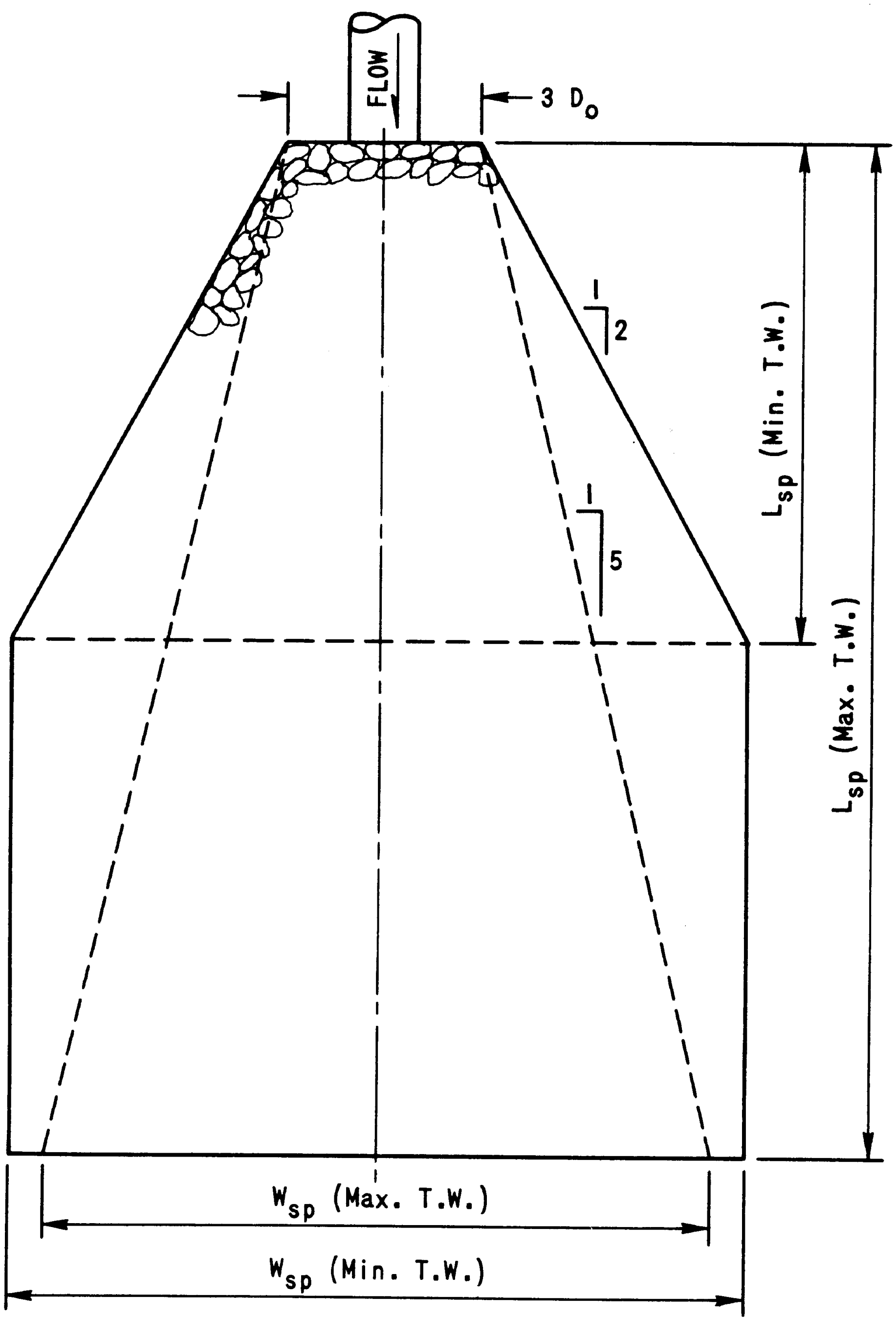
LIST OF ATTACHMENTS

DISSIPATOR LIMITATIONS

Dissipator Type	Froude Number Fr	Allowable Debris			Tailwater TW	Special Consideration
		Silt Sand	Boulders	Floating		
Free Hydraulic Jump	>1	H	H	H	Required	--
CSU Rigid Boundary	<3	M	L	M	--	--
Tumbling Flow	>1	M	L	L	--	$4 < S_o < 25$
Increased Resistance	--	M	L	L	--	Check Outlet Control HW
USBR Type II	4 to 14	M	L	M	Required	--
USBR Type III	4.5 to 17	M	L	M	Required	--
USBR Type IV	2.5 to 4.5	M	L	M	Required	--
SAF	1.7 to 17	M	L	M	Required	--
Contra Costa	<3	H	M	M	<0.5D	--
Hook	1.8 to 3	H	M	M	--	--
USBR Type VI	--	M	L	L	Desirable	Q <400 cfs V <50 fps
Forest Service	--	M	L	L	Desirable	D <36 inches
Drop Structure	<1	H	L	M	Required	Drop <15 ft.
Manifold	--	M	N	N	Desirable	--
Corps Stilling Well	--	M	L	N	Desirable	--
Riprap	<3	H	H	H	--	--

NOTE: N = None
L = Low
M = Moderate
H = Heavy

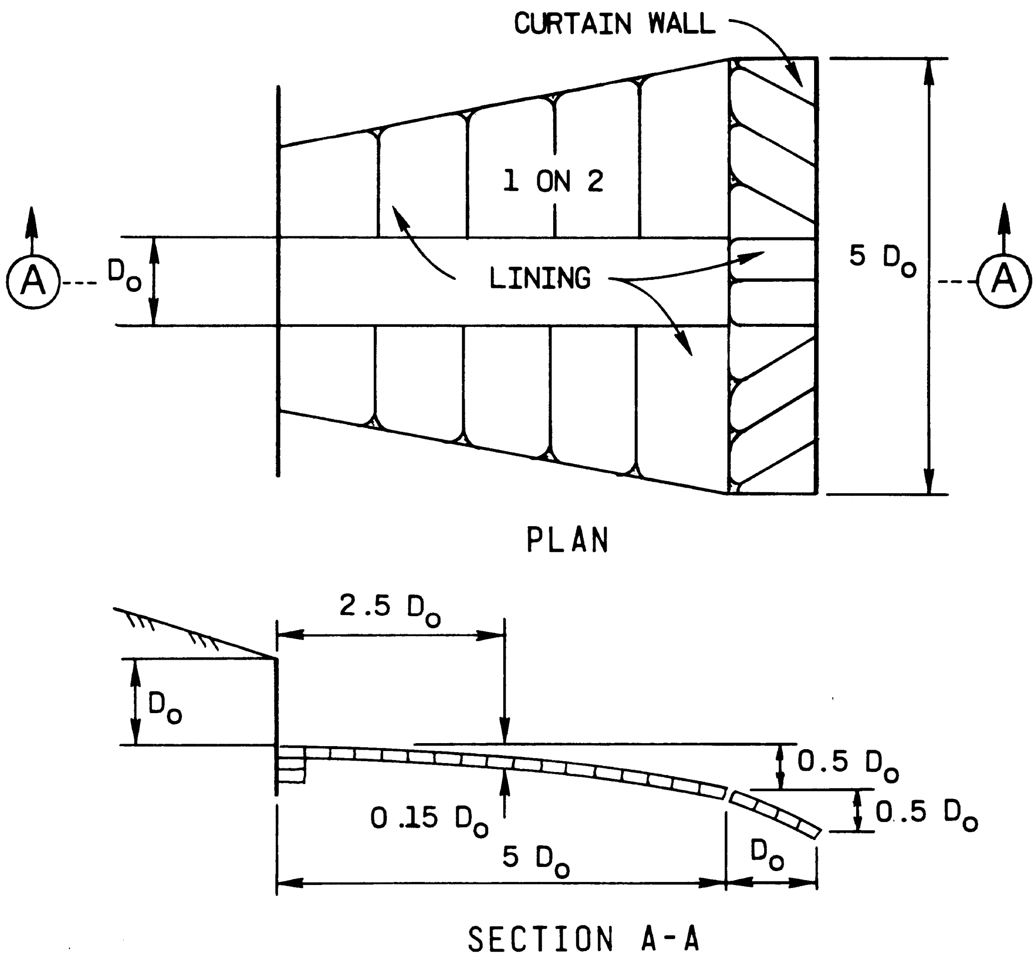
Source HEC No. 14, "Hydraulic Design of Energy Dissipators," FHWA, November, 1975.



RECOMMENDED CONFIGURATION OF RIPRAP BLANKET SUBJECT TO MAXIMUM AND MINIMUM TAILWATERS

Source: Miscellaneous paper H-72-5, "Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets", U.S. Army Engineer Waterways Experiment Station, May, 1972.

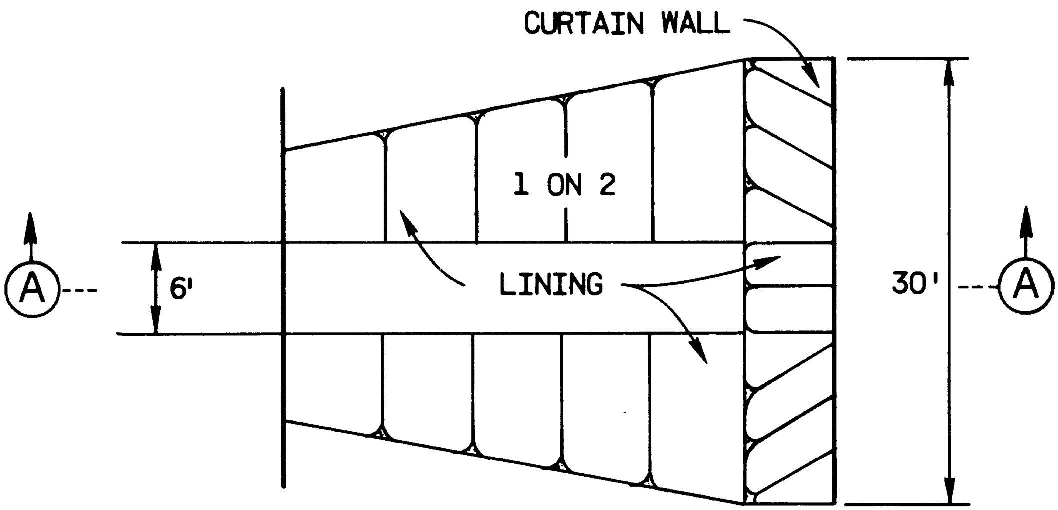
CULVERT OUTLET EROSION PROTECTION, LINED CHANNEL EXPANSIONS



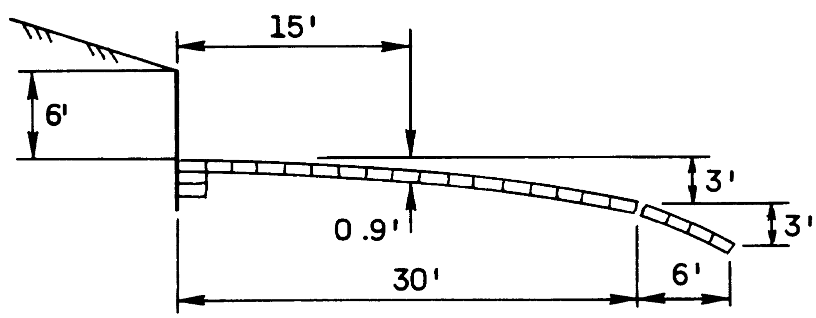
Source: Miscellaneous paper H-72-5, U.S. Army Engineer Waterways Experiment Station, May, 1972.

EXAMPLE PROBLEM

FINAL DESIGN DIMENSIONS FOR THE
LINED CHANNEL EXPANSION

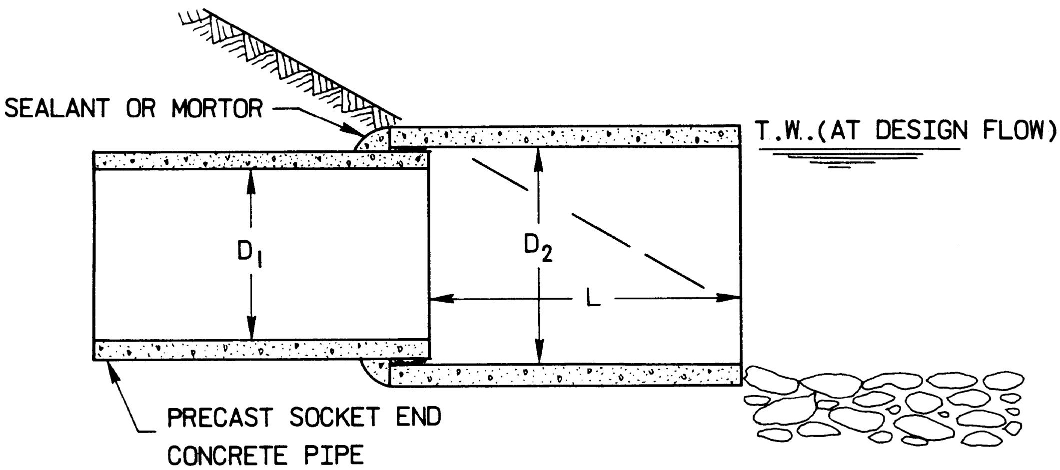


PLAN



SECTION A-A

OUTLET EXPANSION



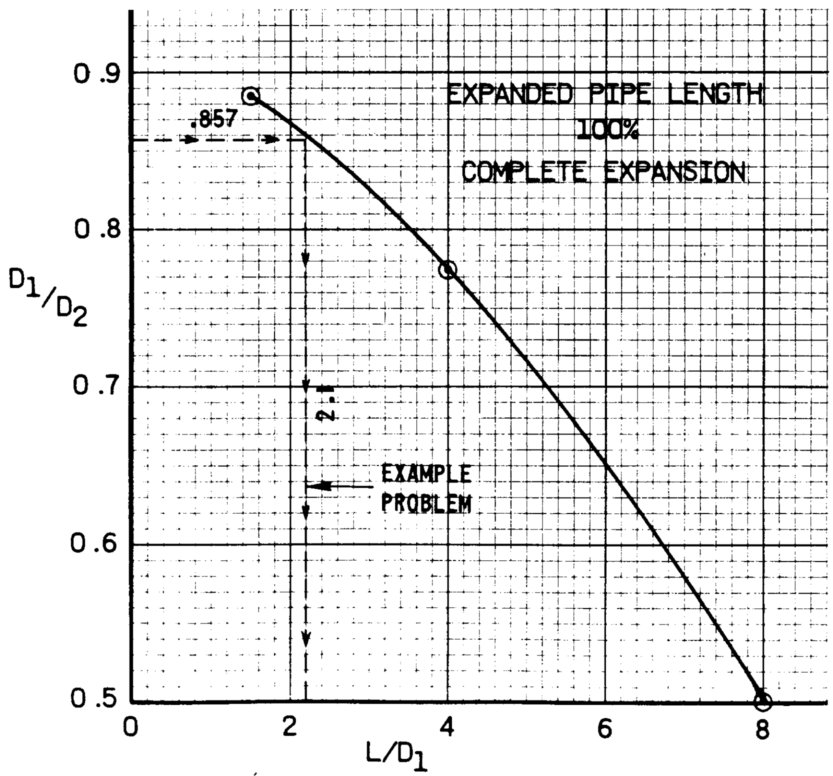
EXPANDED PIPE LENGTH

	100% Complete Expansion	90% Complete Expansion
D_1/D_2	L/D_1	L/D_1
.883	1.5	1.0
.775	4.0	--*
.500	8.0	--*

* NOT REPORTED.

Source: American Concrete Pipe Association, "Culvert Velocity Reduction With an Outlet Expansion," Concrete Pipe News.

DETAIL A



DETAIL B

FDM 13-40-1 Underdrains

June 19, 2013

1.1 Introduction

Underdrains are installed to control three specific types of groundwater:

1. Seepage in cuts or sidehill areas,
2. High-water tables, and
3. Subbase and/or subgrade areas where water enters from either the surface or below the surface. Often a subsurface drain performs multiple functions.

There are many variables and uncertainties about actual subsurface conditions. In general, the more obvious subsurface drainage problems can be anticipated in design; the less obvious are frequently uncovered during construction. Extensive exploration may be required to obtain the design variables with reasonable accuracy. For these reasons many designs are based on local experience and empirical rules that have given satisfactory results. Refer to FDM 10-10-33 for additional information about subsurface drains.

1.2 Descriptions

Underdrains are described below and shall be provided where required (refer to Attachment 1.1).

1.2.1 Sidehill Seepage

The interception of side hill seepage is accomplished by a perforated underdrain laid in a trench on the shoulder, in the ditch, or on the back slope. The flow line should be below the water bearing material but not more than six feet deep. The top six inches of backfill should be impervious.

1.2.2 High-Water Table

High water-tables can be lowered by providing entrenched perforated underdrain on each side of the pavement. The trench should be at least four feet deep to be effective but should not exceed six feet. The top six inches of backfill should be impervious.

1.2.3 Subgrade Drainage

To drain the subgrade or base (usually of water deposited by surface leakage through the pavement or shoulder), longitudinal perforated underdrains (edgedrains) are placed adjacent to the edge of the pavement to a depth equal to or slightly greater than the depth of the subbase. A minimum invert depth would be 20 inches below the top edge of pavement, placed in base aggregate open graded. The trench should be topped with about six inches of asphaltic material in rural sections. In curb and gutter sections, the edgedrain should be placed beneath the curb. Where a permeable base and subbase is extended out to the subgrade shoulder point, edgedrains are not usually required.

1.2.4 Outlets

Edgedrain outlets (drain outs) are placed at low points at a maximum spacing of 250 feet on long grades to provide outlet drainage. The outlets are connected to the edgedrain by elbows. The outlet underdrain and elbow should be unperforated. The outlet should be free from brush or dirt, above surrounding surface water, screened to keep out animals, and marked for location. The marker is to be 8" x 8", white, cold-painted and non-reflective, painted on the centerline of the apron endwall located 6" from the edge of pavement, See SDD "Edgedrain Outlet and Outfall Markers" for more detail. The outlet shall be elevated a minimum of one foot above the bottom of the ditch it is discharging into. Outlets should not be located near other drainage features such as culverts.

1.3 Design Criteria

1.3.1 Size and Length Requirements

The minimum inside pipe diameter for a standard pipe underdrain shall be six inches for lengths of 500 feet or less. As a general rule, this size is adequate as a collector or lateral in most soils. For lengths exceeding 500 feet, the minimum diameter shall be eight inches.

1.3.2 Separation of Drainage

Surface drainage shall not be permitted to discharge into an underdrain. The discharge from an underdrain into a roadway drainage system or a culvert is permissible if the outfall for the underdrain is not under pressure.

1.3.3 Cleanouts

A terminal cleanout is required at the upper end of the underdrain. This is made by bringing the pipe to ground level on a 45° angle. Intermediate inspection wells are required at maximum 500-foot intervals. They must consist of a vertical riser with a light cast-iron cover brought to ground level. The diameter of the riser shall be at least the diameter of the conduit.

1.3.4 Grade Requirements

In general, the grade should not be flatter than 0.5 percent. If this slope is unobtainable, grades of 0.20 percent for laterals and 0.25 percent for mains will be acceptable.

1.3.5 Depth and Spacing of Underdrains

The depth of the underdrain depends on the permeability of the soil, the elevation of the water table, and the amount of drawdown needed to ensure stability. When practical, an underdrain pipe should be set in between an impervious and pervious zone. The pipe should be set in the pervious zone just above the impervious layer. Attachment 1.2 gives suggested depths and spacing of underdrains according to soil types. It is only a guide and should not be considered a substitute for field observations or local experience.

1.4 Underdrain Conduit Installations

The types of underdrain installations relative to conduit characteristics and anticipated service are as follows:

1.4.1 Perforated Underdrains

Perforated underdrains should typically be used when the drainage layer (the pervious material the underdrain is installed in) is open graded aggregate, Installations in native soils may require a wrapped underdrain.

1.4.2 Unperforated Underdrains

Unperforated underdrains are typically used to connect longitudinal perforated pipe underdrains (edgedrains) to outlets.

1.4.3 Wrapped Underdrains

Wrapped underdrains can be plowed into place and are typically more cost effective than Pipe Underdrains placed in open graded material. Refer to FDM 14-5-5 for a discussion of base aggregate open graded. The geosynthetic material or “sock” can reduce their capacity so they should not be used under the roadway or where there is a significant drainage concerns. They should be used in native permeable soil outside the roadway where replacing backfill is not necessary or costs effective. In locations where native soil contains higher percentages of silt or clay their use should be determined on a case by case basis as the silt or clay can clog the "sock".

1.5 Material Considerations

Pipes for underdrains may be made of metal, corrugated polyethylene or other materials specified in WisDOT Standard Specifications.

Non-Metallic Pipes: Perforated pipes of corrugated polyethylene may be used in soils of low resistivity and in the presence of highly aggressive soil or water. Corrugated polyethylene is satisfactory in longitudinal drains where settlement is not anticipated.

Corrugated polyethylene should not be used in deep stabilization trenches where settlement is anticipated or in shallow installations subject to damage by construction traffic.

Metal Pipes: Perforated pipes of corrugated metal (either steel or aluminum) are satisfactory for use in the structural situations mentioned in subparagraph (a) above. However, their use is contingent upon providing the necessary protection against corrosion and abrasion where this is dictated by requirements of the location and limitations of the pipe material.

Steel pipes are appropriate for installations with a 50-year design service life and aluminum pipes for installations with a 25-year design service life.

If a material listed in the Standard Specification is not to be allowed than a note shall be made in the miscellaneous quantities.

1.6 Geotextile Fabric

Geotextile Fabric is used to separate the drainage layer from the surrounding soil. The need for the fabric is dependent on the thickness of the layer and location of the drain. See SDD "Edgedrains" for more guidelines on location of the fabric. If fabric is necessary, Geotextile Fabric Type DF Schedule A must be used unless otherwise justified and must be added as a separate bid item.

1.7 Selection of Type

In cases where more than one material meets the foregoing structural, corrosive, abrasive, and design service life expectancy requirements, alternatives should be specified on the basis of optional selection by the contractor. The selection of a single type of underdrain may be appropriate due to other related factors.

1.8 Construction

The filter material for backfill shall have the same or greater permeability than the surrounding soil but must be fine enough to prevent soil from washing into or through it. Research has shown that a graded material roughly equivalent to fine concrete aggregate is most suitable. Size requirements for this aggregate are found in the Standard Specifications.

The top layer shall always be impervious to avoid infiltration of silty surface water.

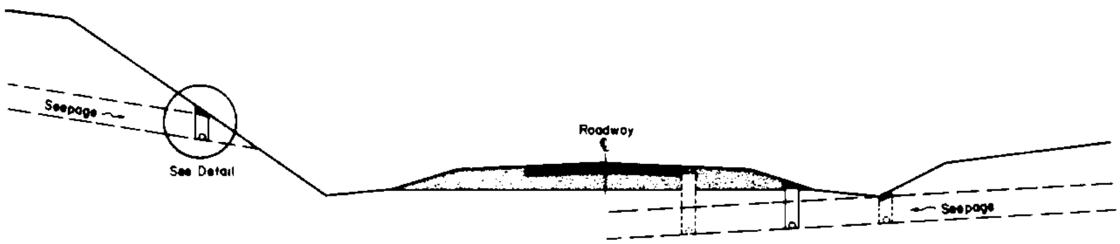
The pipe shall conform to the standard specification for underdrains and be laid with perforation down.

LIST OF ATTACHMENTS

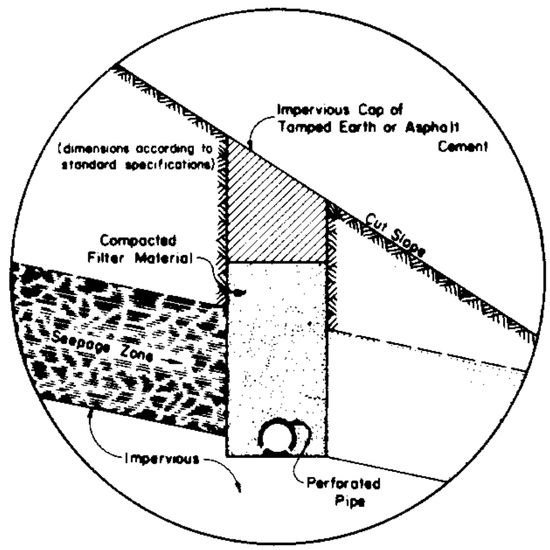
Attachment 1.1 Subdrains

Attachment 1.2 Suggested Depth and Spacing of Underdrains for Various Soil Types

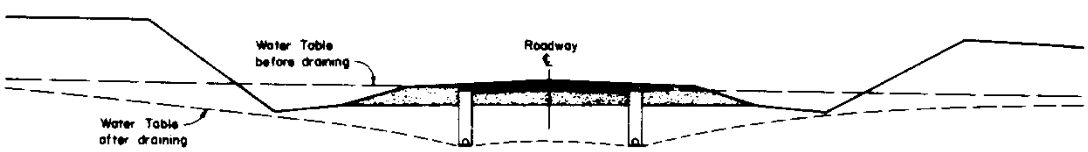
SUBDRAINS



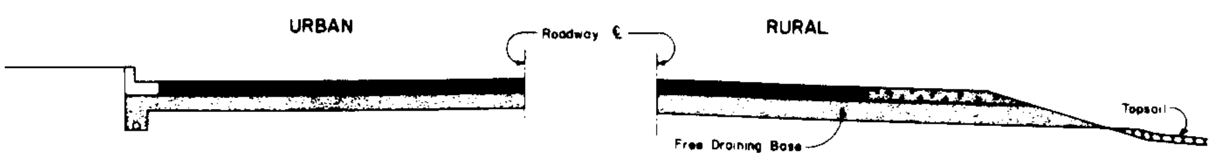
Subdrain to intercept sidehill seepage



Typical details of subdrains



Subdrains to lower water table



Subdrains to drain pavement structure

SUGGESTED DEPTH AND SPACING OF UNDERDRAINS FOR VARIOUS SOIL TYPES

Soil Classes	Soil Composition			Drain Spacing (feet)			
	Percent Sand	Percent Silt	Percent Clay	3 ft. Deep	4 ft. Deep	5 ft. Deep	6 ft. Deep
Clean Sand	80-100	0-20	0-20	110-150	150-200	--	--
Sandy Loam	50-80	0-50	0-20	50-100	100-150	--	--
Loam	30-50	30-50	0-20	30-60	40-80	50-100	60-120
Clay Loam	20-50	20-50	20-30	20-40	25-50	30-60	40-80
Sandy Clay	50-70	0-20	30-50	15-30	20-40	25-50	30-60
Silty Clay	0-20	50-70	30-50	10-25	15-30	20-40	25-50
Clay	0-50	0-50	30-100	15 (Max.)	20 (Max.)	25 (Max.)	40 (Max.)

Note: Depth is measured to invert of pipe.

Source: California Highway Design Manual (Drainage)

FDM 13-45-1 Background

November 30, 2018

1.1 Introduction

This procedure discusses methods for inspecting, repairing, rehabilitating and replacing culverts and storm sewer using various trenchless techniques. The primary focus of the chapter is culverts and specifically the criteria for the design and specification of culvert rehabilitation by sliplining. Sliplining is sliding a pipe or pipe-like liner into an existing culvert that is exhibiting deterioration along its length. This practice has been successfully utilized by WisDOT on LET and maintenance projects for several years. In addition to sliplining, several other methods for in-situ rehabilitation of culverts and trenchless replacement of culverts are discussed. In some cases, these practices can be applied to storm sewer as well.

Other than sliplining, WisDOT has few documented specifications and practices for culvert rehabilitation. With the increased emphasis by WisDOT on asset management improvement types it is important to discuss various options as well as to develop and provide additional design guidance. The practices discussed in this procedure not only offer potential cost savings over traditional open trench replacement but can minimize disruption to the traveling public and reduce project liability such as those resulting from unknown hazardous materials, or significant erosion events during construction. As WisDOT's implementation of these practices are evolving, designers should revisit this section as needed to review updates, revisions, and additions.

1.2 Design Responsibility and Coordination

FDM 13-1-1.4 describes the roles and responsibilities of The Bureau of Structures (BOS) and Bureau of Project Development (BPD) in relation to hydraulic design of drainage structures. Design guidance for the rehabilitation and replacement of culverts as described in this part can be the responsibility of BPD, BOS, and/or Bureau of Technical Services Materials Management Section (MMS). Roles and responsibilities will differ based on the size and type of structure and the proposed method of rehabilitation or repair. Contact one of the Statewide Drainage Engineers in the Bureau of Project Development Roadway Standards Unit (RDSU) with questions regarding the guidance in this section. The Statewide Drainage Engineer will consult with BOS and/or MMS as necessary to determine the design and materials requirements specific to the project. Also, notify the Statewide Drainage Engineers when plans include:

- Sliplining of pipes > 60 inches
- Sliplining of box culverts, structural plate culverts or arches, arch pipes, or horizontal elliptical pipes
- Sliplining of culverts in a floodplain
- Sliplining with machine wound liners
- Replacement of culverts or storm sewers by trenchless methods
- Rehabilitation of culverts or storm sewer by cast in place pipe (CIPP), centrifugally cast or spray liners, trenchless placement or other similar methods requiring project unique special details and/or special provisions.

1.3 Definitions

For the purposes of this procedure; replacement, rehabilitation, and repair are defined as follows:

Replacement - Replacement of culverts can be accomplished by traditional open trench excavation or trenchless construction methods. The existing culvert is either removed or abandoned in place.

Rehabilitation - Rehabilitation of culverts involves returning the culvert to its initial condition or better.

Repair - Repair of culverts is intended to keep the existing culvert in a safe condition and inhibit further deterioration.

Note that the National Cooperative Highway Research Program (NCHRP) Synthesis 303, Assessment and Rehabilitation of Existing Culverts report further defines rehabilitation and repair of culverts. Refer to FDM 13-45-99.1 (Resources) for more details.

The following acronyms and definitions are used to describe pipe materials.

ABS - Acrylonitrile butadiene styrene

CMP - Corrugated metal pipe

Flexible Pipe Culverts - Corrugated metal, polyvinyl chloride and thermoplastic culvert pipes

HDPE - High density polyethylene

HDPP - High density polypropylene

PVC - Polyvinyl chloride

Rigid Pipe Culverts - Concrete pipe culverts

RCP - Reinforced concrete pipe

SRPE - Steel reinforced polyethylene

Thermoplastic Pipe - In this document primarily refers to HDPE or HDPP pipe but can also include composite pipe (PVC and ABS) as well as other forms of plastic piping.

FDM 13-45-5 Design Considerations *November 30, 2018*

5.1 Introduction

The following part describes design considerations for repairing, rehabilitating, and replacing culverts. The primary focus is on repair and rehabilitation because replacement has traditionally been addressed in other project development sections of this manual. Design considerations for culvert replacement by trenchless methods is further addressed in FDM 13-45-20. With any of these methods, consideration should be given to service life of the repair, rehabilitation, or replacement option and whether it is appropriate to the pavement treatment service life of the associated roadway project.

5.2 Evaluation

Determining the level of deterioration, coupled with the consequences of failure, of a culvert or storm sewer is essential to determining when to repair, rehabilitate, or replace the structure. Routine monitoring of the deterioration will assist in determining when repair, rehabilitation or replacement are required. In 2018 WisDOT initiated new culvert inspection and rating procedures to assist in this effort. Additional resources for evaluating culverts are provided in FDM 13-45-99.1. Regional maintenance staff are an excellent resource as are national product organizations such as the American Concrete Pipe Association (ACPA) and National Corrugated Steel Pipe Association (NCSPA). Pipe product suppliers can be resources in assessing the level of damage as well. Depending on the level of damage, experienced hydraulic and/or structural engineers may ultimately be needed to evaluate the structure.

When assessing the level of damage, it is important to consider the differences between rigid and flexible pipes, as this will influence which observed deficiencies require additional investigation and testing. Rigid pipe is designed to support the circumferential soil and live loads with essentially no deflection and the pipe relies on the surrounding soil for only a small fraction of its overall strength. RCP is currently the only rigid pipe material used for culverts. Flexible pipe is designed using soil structure interaction, where the majority of the pipe’s strength is derived from the quality of the backfill soils and compaction. Typical flexible pipe culvert materials include CMP, PVC, HDPE and HDPP (Modified from MNDOT, 2014).

Table 5.1 and Table 5.2 are intended to assist in the evaluation of existing culverts and storm sewers when determining the need to repair, rehabilitate or replace them. Table 5.1 lists information that may be necessary to evaluate repair, rehabilitation and replacement options. Table 5.2 lists underlying issues that may have led to problems observed.

Table 5.1 Key Culvert Observations

Culvert Type	Observations
All Culverts	- Pipe size and type including material, coatings, wall thickness - Past repairs, invert paving, sediment depth - Age - Horizontal and vertical deflection of pipe - Size and location of voids visible through separated joints and holes in the culvert - Sounding the culvert interior with a hammer to listen for 'hollow' sounding areas indicating voids outside the culvert (also useful to check for voids in slipline grouting) - Width of separated of deflected joints, backfill loss at joints - Misalignment of pipe joints - Camber (bend) or settlement of pipe alignment (can be determined by stringing the invert for sagging) - Localized distortions
Rigid Pipe Culverts	- Crack size, location, length and extent of reinforcement corrosion. Corrosion typically occurs in crack widths exceeding 0.02", especially in the presence of chlorides - Depth of invert erosion. Amount of section loss for concrete, and reinforcement as applicable - Sound walls and inverts to locate areas of delaminating concrete
Flexible Pipe Culverts	- Deflection of pipes - Composition and compaction of pipe bedding materials - Corrosion/Pitting - Cracks or tears in pipe wall - Buckling of pipe wall (CMP only) - Failure of lock or welded seams (CMP only) - Corrugation Pattern – Helical (run in a spiral-more common to WisDOT) or annular (circumferential pattern – often riveted) - Seam construction – rolled/locked, riveted, bolted - Tearing at bolt or rivet holes as applicable

(Modified from MN DOT, 2014)

Table 5.2 Possible Causes of Culvert Deterioration

Observed Condition	Possible Cause
Loss/erosion of invert	- Erosion of culvert material due to stream bed loading (All pipe materials) - Corrosion or deterioration of culvert material due to pH and/or resistivity of water and soil, chemical attack, etc. (All pipe materials) - Corrosion of reinforcement and resulting expansive forces resulting in delaminations of concrete (RCP) - Freeze-thaw deterioration (RCP)
Joint separation and infiltration of soil	- Improper seating of joint during installation - Movement of pipe due to slope erosion, freeze-thaw or settlement - Movement of pipe due to excessive deflection or structural deterioration - Buoyancy of culvert with insufficient cover
Piping of soil on exterior of culvert	- Water flowing past holes in the culvert or separated joints causes migration of soil particles - High water head causing migration of soil particles around the outside of the culvert
Invert and crown cracking width in excess of 0.10” in RCP culverts	- Dead and live loading on culvert exceeding culvert design capacity - Increased loading on culvert due to increased soil or groundwater elevations - Excessive construction equipment loading with insufficient cover
Delamination or “Slabbing” (slabs of concrete "peeling" away from the sides of the pipe and a straightening of the reinforcement due to excessive deflection or shear cracks) in RCP culverts	- Dead and live loading on culvert exceeding culvert design capacity - Increased loading on culvert due to increased soil1. or groundwater elevations - Improper bedding of culverts
Deflections exceeding 7.5% of nominal pipe diameter in flexible culverts 2.	- Dead and live loading on culvert exceeding culvert design capacity - Increased loading on culvert due to increased soil1. or groundwater elevations - Improper installation or selection of backfill materials or insufficient compaction - Loss of soil through pipe wall or joints - Piping of materials on exterior of culvert - Excessive construction equipment loading with insufficient cover
Cracks, buckling or separated seams in flexible pipe culverts	- Pipes damaged during installation by equipment or rock in direct contact with pipe - Excessive loading on culvert - Environmental stress cracking in pipe material
Corrosion/Loss of section at crown of pipe	- Chloride’s from road salt infiltration at centerline joints and at edge of pavement (aluminum CMP)

(Modified from MN DOT, 2014)

Notes:

1. Evaluate changes in fill (dead load) over the culvert due to changes in roadway profile from the original design. Profile changes during reconstruction or reconditioning without culvert replacement may exceed of maximum fill height. Maximum fill height generally decreases with larger pipe diameters and elliptical or arch shapes. A search of past projects or the presence of culvert extensions may give clues as to if additional fill has been placed on the culvert.

2. Due to the nature of flexible culverts, and in particular thermoplastic pipe, some deflection is normal with flexible pipe materials. Excessive deflection or point deflection should be evaluated however. If excessive deflection is a concern, consider mandrel testing per CMM 5-50.9 as an option. Newer technologies such as laser profiling are also an option to measure deflection without entry. Profiling can be coupled with video inspection. Where dewatering a pipe is not possible, sonar or similar methodologies is becoming increasing viable. Make careful observations as to sidewall buckling.

3. Localized distortions and similar damage can often be safely removed or repaired to provide clearance for rehabilitation methods such as sliplining. The extent, size, location of the damage or distortion should be noted.

5.3 Hydraulics

Hydraulic evaluation of rehabilitated and repaired culverts is required as most methods will alter the hydraulic capacity of the structure. In addition, the original design conditions (land use, drainage area, precipitation data) under which the culvert was installed could have changed. For areas in floodplains, with a history of localized flooding, or in other high-risk areas, a full hydrology and hydraulic (H&H) analysis may be necessary. When in doubt contact one of the Statewide Drainage Engineers in the Roadway Design Standards Unit (RDSU) for input. Further discussion regarding the hydraulics of slipliners can be found in FDM 13-45-10.4.1.2.

5.4 Structural Condition

Determining the structural condition of a deteriorated culvert is not a straightforward process. As a result, on culvert rehabilitation projects, WisDOT practice is to assume the host pipe is fully deteriorated and the rehabilitation (slipliner, CIPP) carries the full loading condition. In most cases rehabilitation practices that depend on the strength of the host pipe are discouraged except where done as preventative maintenance or a spot repair. At times, however, issues such as site access or maximum depth of cover may prohibit more conventional rehabilitation methods, such as sliplining. In these cases, contact one of the Statewide Drainage Engineers in RDSU for input on the proposed rehabilitation method. This should be done early in the project to allow RDSU to involve BOS and the MMS engineers in evaluating the proposed design assumptions and engineering design methodology employed to analyze the rehabilitation.

Where the structural integrity of the pipe is not in question, localized deterioration, or other issues may be spot repaired without the need to perform a structural analysis. Typical spot repairs can include; resetting endwalls, joint separation or misalignment, small localized holes, localized concrete spalls, and early stage invert corrosion.

Resources are provided in FDM 13-45-99.1 that can assist in determining when a culvert is in need of repair versus the need of full structural rehabilitation. As stated previously with the number of unknowns, such as backfill compaction and condition, culvert material properties and condition, etc. this is not a straightforward process and a conservative analysis should be employed. As-built plans and related installation records, if available, can be used to gain a better understanding of the culvert and backfill materials involved, the original versus current geometric shape, culvert's age, and how a culvert was installed.

When considering a repair or structural rehabilitation of a culvert with areas of concern, consider if the proposed action will further degrade the condition of the culvert during construction. Will the operation cause the pipe to deflect, cause further separation of joints or cause collapse? Will inaction or delays in project initiation cause further degradation or collapse that could lead to failure prior to the repair operation? Such considerations need to be considered when determining if the repair or rehabilitation of a culvert can wait for the normal design and construction timeline of a standard LET project.

5.5 Cleaning and Verification of Clearance

WisDOT's standardized special provision (STSP) for culvert slipliners includes a provision for contractor verification of the interior clearance of the culvert as a well as a separate STSP item for *Cleaning Culvert Pipes for Liner Verification.* The cleaning item should be included on most culvert rehabilitation and repair jobs depending on the conditions observed in the field. Cleaning methods will vary by the size of the culvert and can be left to contractor to determine means and methods. Regardless of the method of cleaning, controls should be in place to capture and dispose of debris and sediment. In many cases, however, it is important to determine clearance prior to the design of a culvert rehabilitation project. Waiting for construction to find out a proposed repair won't work will lead to project delays and increased costs. Additional information regarding liner clearance can be found in FDM 13-45-10.4.1.1.

5.6 Environmental

Rehabilitation, repair, and replacement projects need to follow the same environmental processes as all WisDOT projects. Some specific considerations are covered in the following subsections.

5.6.1 Floodplains

Rehabilitation practices that reduce the culvert internal diameter, such as slipliners, require additional hydraulic modeling when in floodplain areas. The modeling must show that the upstream water surface elevation is not increased when compared to the host culvert. Bureau of Structure review and approval of the hydrology and hydraulics for the lined structure will be required in most cases regardless of whether the structure has a "B-" or "C-" number. Special liners, such as smooth lined CMP's, or improved inlets can assist in meeting pre-lining conditions. In these cases, special provisions specific to the project may be required to specify required Manning's roughness and internal diameter of the culvert. Construction details of improved inlets (bevels), if utilized, will also be required. Finally, often the liner will need to be set at a specific grade and the inlet and outlet to meet the modeled conditions of the floodplain. This will need to be detailed in the plan construction details and emphasized in the contract special provisions.

5.6.2 Aquatic Organism Passage

Rehabilitation practices can affect aquatic organism passage (AOP) in streams. Coordination with WDNR through the liaison process will be required to evaluate project impacts. Slipliners, for example, will often result in increased culvert velocities. In addition, the culvert inverts will be raised by at least the thickness of the liner. If a thick liner is specified this can create a "perched" condition of the culvert and further restrict aquatic organism passage. That said AOP concerns should not eliminate sliplining from consideration. Some slipliners may only raise the invert as much or slightly more than a cured in place or centrifugally cast liner and should not be eliminated from consideration as they provide a longer lasting structural solution for culvert rehabilitation.

5.6.3 Flow Diversion and Dewatering

Culvert rehabilitations and repairs may require flow diversion by bypass pumping, temporary impoundment or other means to keep water out of the work area. Dewatering may be required due to infiltration or high groundwater conditions. Coordination with WDNR through the liaison process will be required to evaluate the diversion or dewatering plans. Plan details may be required to detail this work and/or the project special provisions can require the contractor to develop and submit a dewatering or bypass plan. Special precautions are necessary when dewatering sites with contamination present. Contaminated water will require proper handling and disposal at a treatment plant or specialized facility. Contact the Bureau of Technical Services Environmental Services Section (BTS-ESS) for guidance.

5.6.4 Sediment and Debris

Debris and sediment removed from culverts should be captured and properly disposed of during cleaning operations. The environmental services section can provide guidance on proper disposal of sediments. Site erosion is also a concern. Most rehabilitation and repair methods will not involve trenching, but erosion and tracking is still a concern. For example, backhoes and other heavy equipment are used to install slipliners and bore pits are often excavated for trenchless culvert replacement. Cleaning sediment from a culvert may also involve removing accumulated sediment from the inlet and outfall ditchlines. A basic erosion control plan and material quantities should be included in project plans for account for these localized disturbances.

5.6.5 Additional Environmental Concerns

The materials used in culvert repair and rehabilitation can be of concern to the environment when not handled properly. Concrete and grout pumping and hauling equipment wash out needs to be controlled. Concrete washout and even slurry from sawing can contain metals and is caustic and corrosive, having a pH near 12. Caustic washwater can harm aquatic life. WisDOT will be developing additional guidance on concrete washout in response to the WDNR Construction General Permit first issued in 2018.

Joint repair and Cured in Place Pipe (CIPP) installations involve the use of chemicals that could be harmful to aquatic life. Studies have shown that styrene and other chemicals used in the CIPP resins are sometimes released in concentrations above toxicity thresholds. As a result, the installation of a CIPP liner requires careful planning and execution to reduce the potential for environmental impacts, especially to the downstream receiving waters. All process water used in the curing and post installation cleaning operations shall be captured. The captured waters should be transported to a local wastewater treatment facility capable of treating the impacted water. It is important to verify that the local wastewater facilities have the capabilities and capacity to handle the impacted water during the initial phase of design. If the local wastewater treatment plant can not take the water, contact BTS-ESS for disposal guidance. (modified from MNDOT 2014).

5.7 Safety

Safety is paramount on all WisDOT work sites. Standard Spec 107 requires the contractor to comply with all federal, state, and local laws governing safety, health, and sanitation, and to provide necessary safety devices, protective equipment, and safeguards. The contractor must also take all action reasonably needed to protect the life and health of employees on the job and the safety of the public. CMM 1-35 further describes construction safety.

Safety doesn't only apply to construction sites however. Evaluating culverts presents a number of safety concerns including:

- Hazardous atmosphere: This is of particular concern for blocked culverts. Air flow is restricted and dangerous gases can accumulate or be generated when sediment is disturbed.

- Culvert collapse: The culvert could collapse while workers are inside. Work activities could cause a marginally stable culvert to collapse.
- Water: High flow rates and deep water can create dangerous footing conditions. Falling into pools at the culvert ends is also a hazard.
- Animals in the culvert can be dangerous, especially if trapped.
- Entrapment: Deep mud can entrap personnel walking through it.
- Embankments: Steep slopes, loose cobbles, and wet or frozen ground can create dangerous footing conditions.

(Modified and supplemented from MNDOT, 2014)

Bureau of Structures has some additional guidelines for culvert inspection safety.

When inspecting storm sewers and appurtenances additional precautions are advised as it may meet the conditions of a “permit-required confined space”. Strongly consider remote operated inspection equipment when there is a need for inspecting storm sewer.

5.7.1 Safety Resources

Some additional safety resources are as follows:

- Part 1, Chapter 4 of the WisDOT Structure Inspection manual:

  https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/inspection-manual.aspx#p4

- WisDOT Safety Directive 75 (Confined Spaces – access available to WisDOT staff only).

5.8 Access

The current WisDOT Standardized Special Provisions (STSP) for culvert liners requires the contractor to "obtain easements if necessary for installing long sections of pipe". This is not intended to relieve the designer of the responsibility of determining and securing the easements necessary to carry out culvert lining or other rehabilitation or repair operations. Right of way for construction staging and operations should generally be secured in the form of Temporary Limited Easements before project letting. Reliance on Construction Permits should be avoided. The STSP is intended to require the contractor to secure additional easements for unanticipated means and methods employed. Easements will vary by the type of operation. For culvert liners, on at least one end of the culvert, provide a minimum work area 45 feet long emanating on alignment with the end of pipe and 30-40 feet wide, with adequate access and limited obstructions. It is preferable that this area is available at both ends of the pipe to allow push or pull operations. Alternately a smaller work area of at least 10 feet x 10 feet should be available on the opposing end of the culvert. Trenchless construction can require similarly sized easements for boring pits or equipment staging areas.

5.9 Traffic

Culvert repair, rehabilitation, and trenchless replacement projects generally have significantly less impacts to traffic than open trench replacement. That said there are sometimes where short-term lane or shoulder closures may be necessary. During culvert lining material delivery and grout pumping equipment may need to operate from the shoulder due to limited access or steep embankments. Cured in place pipe lining (CIPP) requires support equipment such as CCTV inspection trucks, cleaning equipment, boiler or curing equipment, compressors, etc.

FDM 13-45-10 Culvert Rehabilitation by Sliplining — *November 30, 2018*

10.1 Introduction

Culvert lining, or sliplining, is sliding a pipe or pipe-like liner into an existing culvert, then grouting the void between the host pipe and the liner. Liners are inserted into the host pipe by either pushing or pulling the liner into place. Sliplining is frequently used on pipes that show deterioration along the whole length of the pipe. Metal culverts with holes along the inverts are the most common candidates for sliplining. Slipline repairs may be a cost-effective alternative to trenching in a new pipe but the use of sliplining is limited to culverts that will have adequate hydraulic capacity after the size reduction.

10.2 Types of Sliplining

Segmental sliplining consists of lining the deteriorated culvert with sections shorter than that of the existing culvert. Bell or spigot joint is commonly used to join culvert segments. Segments of the liner are assembled at entry points and forced into the host culvert. As each segment is added, the liner is forced further into the existing culvert until lining has been completed (FHWA, 2005). Segmental sliplining is the practice primarily utilized by WisDOT. WisDOT has several approved segmental slipliners on the approved products list.

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

Continuous sliplining involves the lining of a deteriorated culvert with a continuous liner. Liners are generally made from polyethylene or high-density polyethylene pipe segments that are butt-fused together. The continuous liner is pulled, pushed, or simultaneously pushed and pulled into the host culvert. Once installed, the annular space is generally grouted and service connections are reopened (FHWA, 2005).

10.3 Sliplining Materials

Many types of pipe materials can be used for sliplining however WisDOT's list of approved liners consist of the following:

- Dual Wall Corrugated PVC (ASTM F949)
- Steel Reinforced Polyethylene - SRPE (AASHTO MP-20)
- Closed Profile HDPE - Solid Wall HDPE (ASTM D-3350 /ASTM F714/ASTM F894)

Manning's roughness (n) values for approved liners are laboratory tested specifically to the product and published with the approved list.

WisDOT employs other materials such as smooth lined corrugated metal pipes (CMP's) and smooth-lined polycoated CMP's for specialty applications. Refer to FDM 13-45-10.4.9 for more details.

***Figure 10.1* PVC Culvert Slipliner**

10.4 Slipliner Design Considerations

10.4.1 Liner Sizing

A slipliner needs to be sized for the hydraulic conditions of the site as well as sized to physically fit in the host culvert.

10.4.1.1 Liner Dimensions

As discussed in FDM 13-45-5.5 WisDOT's standardized special provision (STSP) for culvert slipliners includes provisions for contractor verification of the interior clearance of the culvert as a well as a separate STSP item for Cleaning Culvert Pipes for Liner Verification. The liner STSP also requires the contractor to verify the internal clearance of the culvert prior to ordering a liner to identify obstructions, deformations, or deflections that may require repair or special consideration. This does not absolve the designer from confirming host pipe dimensions and obstructions during the design process. Where hydraulics is critical, and the maximum sized liner is required, it may be necessary to confirm the host pipe can fit a liner by pulling a mandrel, laser profiling, or taking detailed direct measurements. In addition to variations in the pipe, the designer must consider the thickness of the liner wall, additional external liner thickness at joints (bells or flanges), while still allowing the minimum recommended clearance between the liner and host pipe. In absence of a recommended clearance, the designer should assume at least 2 inches between the host pipe and outside of the liner to ensure effective grouting.

If needed, a separate special provision can be used to address areas in need of spot repair damage that would interfere with the intended lining. Examples of spot repairs include jacking or bracing an area of the host culvert to remove a bulge or repairing joints with specialty grouts or bands to control groundwater infiltration prior to sliplining.

WisDOT's approved list of slipliners is found here:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

Internal and outside diameter dimensions can be found at the associated manufacturer websites.

10.4.1.2 Liner Hydraulics

In the past WisDOT has only hydraulically analyzed culvert liners by performing a Manning's calculation comparison for the roughened host pipe (often CMP) and the smooth liner. Often the reduction in Manning's roughness of the smoother pipe can offset reducing the pipe diameter with a liner. This is only the case for culverts operating under outlet control (flow controlled by culvert barrel). For culverts in inlet control (flow controlled by the upstream opening, not the barrel), reducing the diameter will increase the headwater condition. Improving the inlet configuration may help offset the headwater increase by allowing flow to enter the culvert more efficiently (refer to FDM 13-45-10.4.7). In addition, the culvert may still perform adequately even with the reduction in opening size, but an analysis is required to confirm this.

At a minimum, a hydraulic analysis comparing the existing culvert to the lined condition should be performed. FWHA's HY-8 free culvert hydraulics software can be used to analyze the headwater and velocity conditions of the existing and lined culvert. Assumptions regarding roughness of the existing pipe should be documented from a recognized resource such as the FHWA HDS-5 User Manual, an existing flood study, or other recognized publication. Attachment 3.1 provides an example of the minimum hydraulics analysis recommended for a liner project. As described in FDM 13-45-5.3 for areas in floodplains, with a history of localized flooding, or in other high-risk areas, a more detailed hydrology and hydraulic (H&H) analysis may be necessary. When in doubt contact one of the Statewide Drainage Engineers in the RDSU for input.

10.4.2 Invert Height Change

Lining a culvert will result in a rise of the culvert invert. At a minimum, this will be the thickness of the liner. Additional increase can result from the use of guides or rails used to install the liner, protruding bolts in a steel plate culvert or from increased external liner diameter (thickness) at joints. The rise in invert will typically be around 0.2 ft to 0.3 ft. This should be accounted for in the hydraulic analysis of the liner, especially when floodplain impacts are being considered. The designer should not only evaluate the change in headwater relative to the culvert. The raise in invert must be accounted for.

For example:

A 36-inch diameter host culvert has an invert of EL. = 800 ft and according to hydraulic analysis a 25-year design event depth of flow of 3.9 feet (EL.=803.9 ft). A 27-inch I.D. diameter slipliner is considered and results in a headwater of 3.75 feet. By all appearances the headwater has decreased.

In fact, when considering the change in elevation of the liner (in this case assume 0.25 ft) and the vertical datum the headwater has increased by 0.10 ft (Existing Invert EL. = 800 ft +0.25 ft + proposed Hw EL = 804.0 ft).

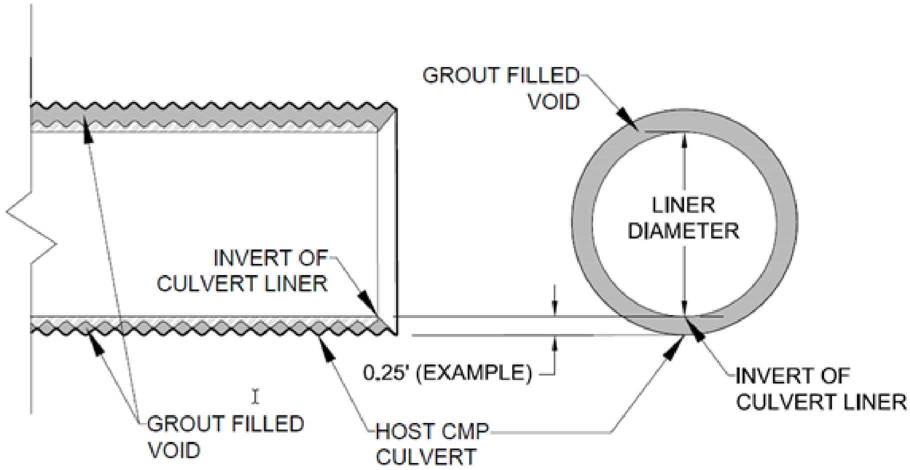


***Figure 10.2** Invert Change in Sliplined Culvert*

For a situation where the culvert is in a floodplain (see FDM 13-45-5.6.1) the change in headwater would not be allowed. For this situation, if we were to assume the 36-Inch culvert is not in a regulated floodplain, the headwater to depth (Hw/D) ratio of 1.5 for the host pipe still meets the requirements of FDM 13-15-5. Design Freeboard and Headwater-to-Depth Ratio and as long as no significant impacts resulted from the increased headwater, the 27-inch liner is assumed to meet the hydraulic needs of the site.

For situations with limited clearance the designer can evaluate the use of liners with specialty joints that have no increase in external diameter at the joint. A few WisDOT approved liners meet this condition. A special provision article can be used to modify the existing liner STSP to require a low-profile joint.

10.4.3 Installation Loads

The manufacturers of the culvert liners listed on WisDOT’s Approved Products List should provide recommendations for the maximum loads that can be placed on their products for push or pull installation and grouting operations. Contractors are to follow this guidance during installation. For special applications as described in FDM 13-45-10.4.9 it is advised that the designer evaluate the loading information for the intended product and consider including references to relevant information, special installation requirements, or precautions in the contract special provisions.

10.4.4 Pipe Joints In Liners

Segmental culvert liners have some form of watertight (gasketed) bell and spigot joints between pipe segments. These joints need to withstand pushing and pulling loads as well as grout pressure. If the contractor follows manufacturer guidance most gasketed joints can withstand normal grouting pressures. For nonstandard liner materials like CMP or structural plate culverts, additional restraint and/or joint sealing may be required to prevent leakage of grout.

As described previously the thickness of the liner at the joint needs to be accommodated as well. For example, a liner wall may be 0.2 ft thick resulting in a 0.4 ft difference between the inner and outer wall diameters, however at the joint the outer diameter increases another 0.2 ft due to the protrusion at the joint. Therefore, the true clearance required for the pipe is 0.6 ft larger than the nominal internal diameter. In addition, when assuming the new invert of the lined pipe this thickness at the joint should be considered, not the thickness of the outer wall of the pipe. As described above, in limited clearance situations liners are available without “bells” at the joint.

10.4.5 Liner Grouting

10.4.5.1 Liner Grouting

The annular space between the host culvert and liner should be completely grouted. Just bulkheading the ends of the pipe is not appropriate. Completely grouting the void forms the soil-structure interface important in developing the strength in flexible pipe systems. Annular space grouting has the additional benefits of reducing future movement of roadbed material through misaligned joints, distribution of vehicle load and dead loads on the liner, potential stabilization of voids surrounding the host culvert, and reduces the likelihood of damage or deflection from point loads should the host pipe someday collapse. The contractor’s grouting plan should include locations for both grout injection and witness ports to confirm complete filling of the void space.

10.4.5.2 Grouting Materials

Materials for annular space grouting ideally are easily placed/pumped over large distances, flowable and self-leveling, and require minimal pumping pressures to mobilize. Two types of grout are typically used, flowable cementitious grouts or cellular grouts. Cementitious grouts are more commonly used due to cost reasons. Cementitious grouts for sliplining often are a mix of fly ash, cement, and water. Fly ash is added to reduce cured permeability and retard set time. Aggregates, when used, are small. Compressive strength can be low. The primary purpose of the grout is to form the structure-soil interface for the liner. An added strength of the liner due to the grout is secondary. Some research has suggested higher strength grouts can be a disadvantage as the grout cracks into larger pieces which can point load the liner. Cementitious grouts should be readily available and do not require specialized equipment. They also may be better suited for displacing groundwater infiltration. To date this has been the most common material used to grout culvert liners on WisDOT projects and may be the best alternative for small or remote projects.

Cellular grout is a mixture of cement and water which also employs foaming agents and/or low-density aggregate to reduce fluid loads during grouting. In general specialty contractors and equipment are required for this material. These grouts have low compressive strengths but high flowability. They will also tend to cause fewer issues related to pumping pressure and liner buoyancy and may be best suited for long culvert liner applications or situations with minimum annular space between the host pipe and liner.

WisDOT's standard STSP for culvert lining allows the contractor to select either cementitious or cellular grout. The designer may wish to consider a special provision article requiring one or the other mixes based on site conditions. Some considerations include:

- groundwater infiltration
- annular space size
- soil conditions
- liner length
- pressure capabilities of the liner and host pipe.
- weather/temperatures (PVC liners are more brittle in the cold)
- structure depth and access

As discussed in FDM 13-45-5.6.5, concrete and grout pumping and hauling equipment wash out needs to be controlled. Concrete washout and even slurry from sawing can contain metals and is caustic and corrosive, having a pH near 12. Caustic washwater can harm aquatic life. WisDOT will be developing additional guidance on concrete washout in response to the WDNR Construction General Permit first issued in 2018.

10.4.5.3 Grouting Pressures

Excessive pumping pressures can be an issue during sliplining. Pumping pressures need to be monitored during the grouting process especially for long or steep installations as discussed in FDM 13-45-10.4.6. For most liners recommended grouting pressure will be 5 psi or below. Excessive grouting pressure can cause joint leakage, joint deflection, liner floatation, and/or liner deformation.

***Figure 10.3* Bulge at Liner Joint Likely from Excessive Pumping Pressure**

10.4.5.4 Liner Floatation During Grouting

The slipliner must be kept from floating or deflecting during grouting. Grout will tend to be denser than the slipliner and can cause the liner to become buoyant and float or deflect and misalign. If there are obstructions or damage left in place in the host pipe these can damage the liner if it floats. Figure 10.4 shows an example of obstructions left in place that punched through a PVC liner that floated during grouting.

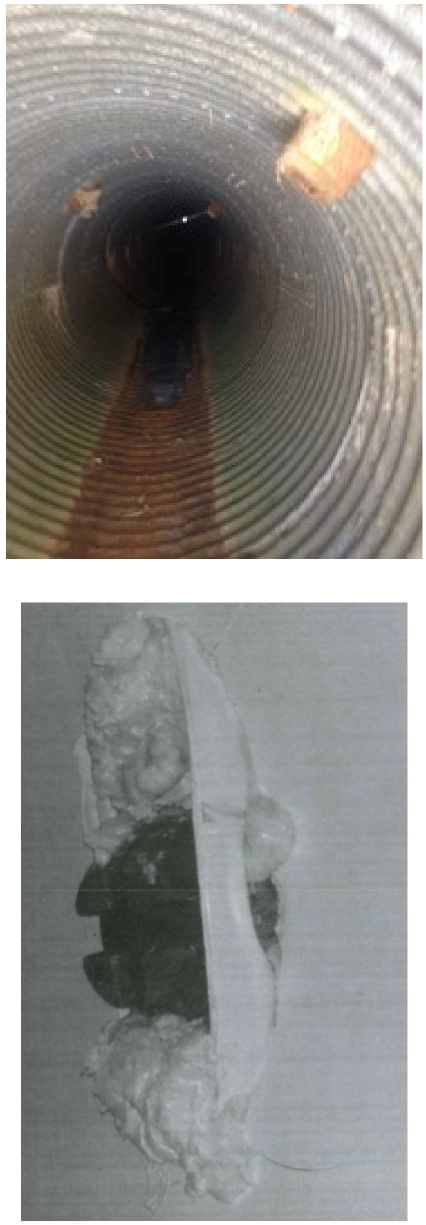


Figure 10.4 ***Obstruction Pierced Liner During Grouting***

Liner floatation may also raise an invert higher than the designer intended and cause headwater concerns. Excessive grouting pressures will increase the likelihood of floatation.

The contractors grouting plan should describe the intended methods to prevent floatation. Grout staging is one of the best methods to prevent floatation and control loading on the liner from grouting. Some additional measures include bulkheading and filling the liner with water, blocking between the host pipe and liner, and installing jacks through holes in the liner, or through grout ports. The manufacturer should approve the methods of preventing floatation especially when blocking liners (point load concerns) or cutting holes for jacks or other bracing.

***Figure 10.5* Floatation Bracing Examples with Blocking or Jacks (Photos courtesy of Contech)**

10.4.6 Sliplining Long or Steep Culverts

Both long and/or steep culverts warrant additional consideration, especially during grouting operations. Long culverts can place additional stress on liner pipes during push or pull sliplining. Buoyancy forces during grouting, which normally may be analyzed by the supplier two dimensionally for a typical culvert, require additional consideration on a steep slope to accommodate additional uplift loads and non-uniform uplift loads being exerted along the liner. In addition to liner floatation, excessive grout pressures can be an issue and need to be considered. For long and/or steep pipes grouting should be performed in multiple lifts and pressures need to be monitored closely to avoid liner bulging, deflecting or misshaping joints, or otherwise damaging the liner. On steep pipes additional shoring of downstream bulkheads may be required and grouting should be performed in multiple stages to prevent excessive loading on the pipe during curing. Some manufacturers may recommend cellular grouts for especially long or steep culverts.

10.4.7 Improved Inlets for Culvert Liners

Culvert Liners shall not be allowed to extend beyond the host culvert unless the designs call for constructing a new headwall. This can result in inefficient inlet capacity of the culvert as it may behave like a “culvert projecting from fill”. Plan details or notes should be included when the inlet configuration of the slipliner is critical to meet the intended hydraulic conditions. In some cases, a bevel-edged inlet can be formed between the host culvert and the slipliner (Figure 10.6).

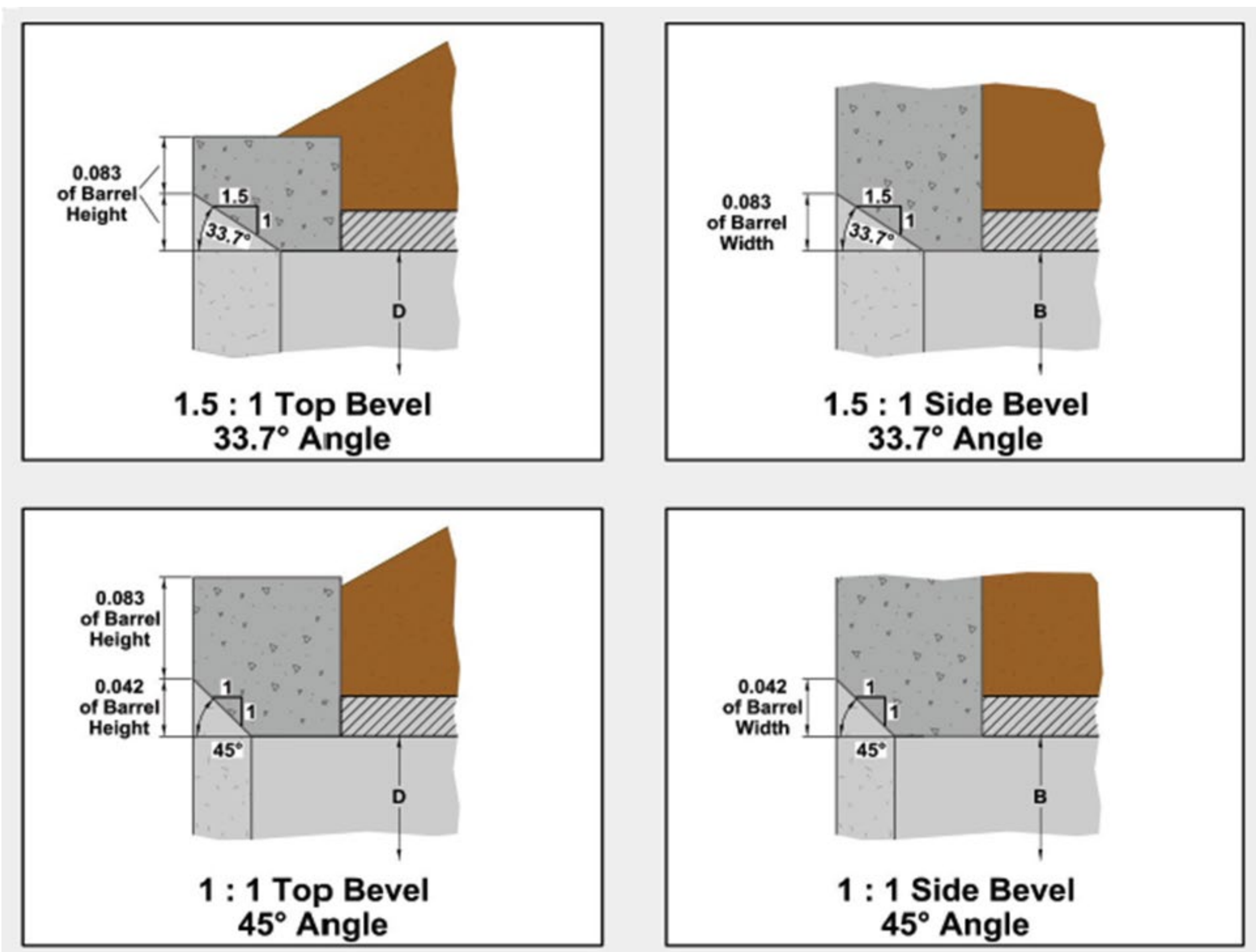


***Figure 10.6* Beveled Inlet Configurations (Source: FHWA HDS 5)**

FDM 13-15-5.7 describes improved inlet types. Regarding bevel-edged inlets it states, *“The bevel-edged inlet is the most economical method of improving the capacity of a conventional culvert. The addition of bevels to a conventional culvert with a square-edged inlet increases culvert capacity by five to 20 percent. Note: Bevels should be used on all cast-in-place culvert entrance headwalls, both conventional and improved inlet types.”*

In some cases, especially for larger diameter culverts, it may make sense to remove a deteriorated metal endwall and construct a concrete masonry endwall with a bevel-edged inlet. The designer should be mindful of roadside safety concerns for the new masonry endwalls.

10.4.8 Discharge Velocity

As described in FDM 13-45-10.4.1.2, when in outlet control, liner hydraulic capacity often depends on offsetting the reduction in culvert diameter by decreasing the Manning roughness of the pipe. As a result, the velocity of the flow in the pipe will increase. Additional downstream scour and erosion control measures may be necessary to offset the increased outlet velocity. For high outlet velocity conditions (typically >14 ft/s) riprap may not provide sufficient energy dissipation or right of way limits may limit placement of riprap. In these cases, energy dissipators may be necessary. FHWA HEC 14 *Design of Energy Dissipators for Culverts and Channels* can be used to evaluate energy dissipation at culverts. Fortunately, FWHA's HY8 software has incorporated the HEC 14 process allowing for increased efficiency in design and evaluation of energy dissipation options.

10.4.9 Special Sliplining Applications

10.4.9.1 Lining Pipe Arches

Steel plate and CMP pipe arches can be sliplined in some instances. In these cases, a project specific special provision will be required. The existing WisDOT Culvert Pipe Liner STSP cannot be used as it applies to round culverts. The designer may, however, use this STSP as a basis for a special provision for items of work such as verification, cleaning, grouting, or handling and installation.

It is important to perform detailed measurements of arch culverts when considering a rehabilitation project, especially for steel plate structures. Arches and especially plate structures may differ in radius in the haunches (corners) significantly between sizes even when compared to tables in historical industry or design manuals. Some plate structures may also have been specified with a slight vertical ellipse, especially in high fill situations. Good measurements, and where possible, review of as-built documentation, are important to the design process.

In general, round liners do not meet the hydraulic conditions required for sliplining pipe arches but they should be analyzed as a lower cost option for a site. For larger or multi-culvert installations, it may be worth evaluating the cost of using round slipliners in the host arch pipes and installing a "relief" culvert by trenchless methods to meet the hydraulics of the site. Although some manufacturers promote the practice, WisDOT does not allow the installation of thermoplastic liners that have been braced or otherwise deflected into an arch or horizontal elliptical (HE) shape for sliplining.

Figure 10.7 "Smooth-Lined" Poly-Coated CMP Arch used to Line Existing Steel Arch Culvert

For sliplining steel plate and CMP pipe arches WisDOT advocates the use of smooth lined CMP pipe arches (Figure 10.7). In these cases, the manufacturer fabricates an additional steel liner that covers the interior corrugations of the pipe. The Manning's roughness of the liner is reduced closer to that of a concrete pipe. The reduction in Manning's roughness compared to the host pipe in some cases will offset the reduction in pipe diameter. For additional longevity and protection from corrosion and abrasion, the liners can be polymer coated (Figure 10.8). In some cases, these smooth lined CMP pipe arch slipliners can also be manufactured in nonstandard sizes to meet the needs of specific applications. In special circumstances, arch pipes can also be lined with specialized field welded steel plate arch shaped liners.

As previously stated, the standard STSP for culvert liners cannot be used for “non-circular” pipe and a project specific special provision is necessary. Contact one of the Statewide Drainage Engineers in RDSU for guidance for analyzing and specifying slipliners for arch or HE culverts.

***Figure 10.8* “Smooth” Polymer Coated CMP**

10.4.9.3 Lining Box Culverts

Lining box culverts is possible in some cases but requires coordination with the Statewide Drainage Engineers in the RDSU and the BOS for guidance. Box culvert sliplining can be accomplished with specialized field welded steel plate box culvert liners, structural plate boxes or arches, or in some instances round culverts or steel plate culverts.

When considering lining a box culvert contact one of the Statewide Drainage Engineers in RDSU who will in turn provide guidance and engage BOS. A thorough hydrology and hydraulic as well as structural analysis will be required in most cases, especially when the culvert is located in a floodplain (see FDM 13-45-5.6.1)

10.4.9.4 Tunnel Plate Liners

Round, arched and box shaped culverts in some cases can be lined with tunnel liner plate. Tunnel liner plate, like steel plate pipe, bolts together in sections that form the culvert. Tunnel liner plate assembles from the inside allowing for it to serve as shoring during construction. The tunnel liner plate can also provide shoring to facilitate the complete removal of portions of a failing or severely deflected culvert while providing a safe working environment. Tunnel liner plates may have internal rings as part of the structure that should be considered as part of the hydraulic analysis of the liner.

Figure 10.9 ***Liner Plate (Photo courtesy of Contech)***

10.4.9.5 Machine Wound Liners

Machine wound liners, often referred to spiral wound liners, consist of strips of interlocking or continuously welded-seam thermoplastic pipe material (often PVC or steel reinforced HDPE) that are field fabricated by on-site machinery. The liners can be used to address corrosion or groundwater infiltration and are best suited for limited access installations where traditional sliplining is constrained. Often the machinery required can fit within a small-bore pit, manhole, or even the pipe itself. Some specialized equipment can travel down the pipe and wind the liner tight to the wall of the host pipe. Other methods allow the insertion of the liner at a fixed diameter where it is then expanded against the wall of the host pipe. Most often the expanding type of liner is done in smaller diameter pipes. For larger pipes or otherwise when the liner is not expended tight to the host pipe wall grout should be used to fill the annular space. Machine wound liner joints can be made watertight with machine applied sealants or welds during the installation process.

Figure 10.10 ***LSpiral Wound Liner (Photo courtesy of Caltrans, 2011)***

For installations with higher design loads steel reinforced thermoplastic strips are available. The steel reinforcement is added to the exterior ribbing of the field fabricated pipe so the resulting interior is smooth. The resulting annular space is grouted.

WisDOT does not have a standard item or STSP for machine wound liners. Coordinate with the Statewide Drainage Engineers when this type of liner is under evaluation for a project. The drainage engineer will involve BOS and the MMS as appropriate. General guidance for spiral wound and similar liners is as follows:

- Pipes 48” and less can be lined based on manufacturer-recommended empirical analysis for structural capacity, stamped by a professional engineer registered in the state of Wisconsin and submitted to the project for review 14 days prior to delivery of the material.

- Pipes larger than 48” require a site-specific numerical (finite-element) structural analysis that incorporates soil boring data from the site and any additional anticipated loadings from dead, live, or adjacent foundation sources, stamped by a professional engineer registered in the state of Wisconsin and submitted to the project for review 30 days prior to the delivery of the material. It is recommended that a geotechnical subsurface investigation be performed during the design process and an initial liner analysis be performed by the design engineer to determine the feasibility of lining pipes greater than 48 inches in diameter using a spiral wound liner. The geotechnical subsurface investigation should provide the necessary level of detail to allow the accurate computational analyses of pipe lining design. The actual geology and site conditions will determine how many, and what spacing of, borings are required.
- The liner shall meet the hydraulic conditions of the site (see FDM 13-45-10.4.1.2).

10.4.10 Additional Sliplining Considerations and Restrictions

The following is a list of additional considerations and restrictions for sliplining culverts. Some of this material is repeated from previous sections for emphasis.

- Remember the exterior dimensions of a liner when considering a liner product. For example, steel plate liners not only need clearance for the exterior corrugations but an additional 1 inch or more of clearance may be lost to bolts.
- WisDOT’s STSP is only intended to be used for circular culverts. The STSP should not be used on culverts that are:
  - Located in a mapped floodplain unless culvert with liner is modeled to show upstream water surface elevation is not increased when compared to the existing culvert. (FDM 13-45-5.6.1)
  - Located in drainage districts unless the drainage board approves the installation of the liner. (FDM 5-15-1)
  - Located on a stream where aquatic organism passage is a concern unless WDNR agrees with the use of a liner. (FDM 13-45-5.6.2)
  - Crushed, collapsed, or have excessive deflection that may make liner installation impossible.
  - Horizontal elliptical or arch pipes. Project specific special provisions are required. (FDM 13-45-10.4.9.1)
- Do not line previously lined culverts. The liner must meet the hydraulic condition of the original host pipe, not that of a previously installed liner.
- Project plans must include characteristics of the existing culvert including material of construction, diameter, pipe slope, depth of fill and any additional loading requirements. Because of variation in liner size, do not show proposed liner diameters on the plans unless it is required to meet special hydraulic conditions such as maintaining a critical water surface elevation.
- Field verify culvert material of construction, size, shape and condition during design. Include a sufficient quantity under the Cleaning Culvert Pipes for Liner Verification bid item for the construction staff to confirm required liner dimensions before ordering material. Typically, the bid item is not needed for every liner on a multi-liner project but only for those likely to be under water or otherwise obscured by sediment or debris.
- When specifying liners for concrete pipe, designers should verify the hydraulic requirements of this special provision can be met.
- Perform a complete culvert hydrology and hydraulic analysis on culverts to be lined that have hydraulically sensitive structures and/or property upstream and in low cover areas where over topping may be a concern.
- The liner STSP states "Obtain easements if necessary for installing long sections of pipe". This IS NOT intended to relieve the designer of the responsibility of determining and securing the easements necessary to carry out culvert lining operations (FDM 13-45-5.8).

LIST OF ATTACHMENTS

Attachment 10.1	Culvert Liner Hydraulic Analysis Example

FDM 13-45-15 Other Culvert Repair and Rehabilitation Practices

November 30, 2018

15.1 Introduction

The following is a brief description of some additional repair and rehabilitation practices for culverts. These practices are not widely utilized by WisDOT and there are no standard items of work or STSP's covering these methods. When one of these methods, or similar non-standard repair or rehabilitation is proposed for a project, notify one of the Statewide Drainage Engineers in the Bureau of Project Development, Roadway Design Standards Unit (RDSU). The Statewide Drainage Engineer will consult with the BOS and/or MMS as necessary to determine the design and materials requirements specific to the project and the appropriateness of the proposed method to the location in question.

15.2 Invert Paving

Invert paving involves placing reinforced concrete on the invert of an existing culvert. For culverts with partial deterioration, and where the structural capacity of the culvert has not been compromised, invert paving may be possible without a comprehensive structural review.

When a culvert is fully deteriorated, the invert paving section, reinforcement, and connections between the paving and culvert should be structurally analyzed and designed to restore the culvert's capacity. Consideration should be given to performing a paved invert analysis and design. The flexural, diagonal tension and radial tension capacity of a fully deteriorated culvert can be evaluated using the software PIPECAR, available from the American Concrete Association or CANDE, which is available from FHWA. The culvert should be substantially unloaded or shored, before the structural repairs are completed, otherwise the repair will not participate in carrying load until additional load is imposed on the culvert. (Modified from MNDOT 2014). Under most conditions a metal pipe requiring invert repair should be considered fully deteriorated.

***Figure 4.1* Extending and Invert Paving a Structural Plate Pipe Arch (Photo courtesy of Contech)**

15.3 Cured in Place Pipe Liner (CIPP)

Cured in place pipe lining involves inserting a resin impregnated fabric (often synthetic felt, commonly known as needle-punched geosynthetic) tube into a culvert or storm sewer. One of two methods is generally employed to install the liner, pulled-in-place or inversion. Depending on the method, the liner is placed and inflated, with air or hot water, and then is cured with hot water, steam, or more recently ultraviolet light. The liner will conform to the wall of the host pipe so any deflection or damage within the pipe will reflect through the liner, and if protruding, may damage the liner. This may require that areas of the culvert or storm sewer be repaired or replaced prior to insertion of the CIPP liner. Voids along the pipe will also need to be filled prior to CIPP lining.

***Figure 15.2** CIPP Culvert Lining*

15.3.1 Additional Environmental Considerations for Cured in Place Pipe Liner (CIPP)

Depending on the curing method, CIPP lining has the potential to create conditions harmful to aquatic life. Studies have shown that styrene and other chemicals can be released in concentrations above toxicity thresholds. Careful planning and execution of CIPP lining is critical to reducing the potential for environmental impacts, especially to the downstream receiving waters. Excessive temperatures from hot water or steam can also be a concern to aquatic life. Due to potential impacts to aquatic life, downstream receiving waters shall be protected from discharges from CIPP operations. Project specifications shall include provisions to capture process waters from the CIPP process. Further, the specifications shall require that the captured waters be transported to a local wastewater treatment facility capable of treating the impacted water. It is recommended that the designer may wish to verify with the local plant that it has the capability to handle the impacted waters during the design process. If the local wastewater treatment plant can not take the water, contact BTS-ESS for disposal guidance. Alternately, ultraviolet light curing is becoming more common and will significantly reduce the potential for construction discharges from a site. Unfortunetly, ultraviolet curing is limited to smaller diameter pipe. Regardless of the method, a liner that is not properly cured can release increased amounts of styrene and other harmful chemicals to the receiving waters, even if the process water is properly collected and disposed of.

15.4 Centrifugally Cast and Spray-on Liners

Lining systems are available where cementitious mortar or other material are sprayed or centrifugally cast to the interior of a pipe. Without additional reinforcement, spray-on lining may add little or no structural integrity to the existing culvert. Some cementitious mortar systems use fibers in the mix to enhance the flexural strength. Non-cementitious systems should only be considered where improved watertightness and corrosion resistance is desired. Multiple passes may be necessary to reach the design thickness of the liner. Infiltration into the pipe will also need to be stopped prior to application.

WisDOT's current position is that unless additional reinforcement is provided, in most cases this practice shall be limited to applications where the pipe diameter is small (<48-inch diameter) and the strength of the host pipe is not in question. With centrifugally cast products keep in mind that the culvert invert needs to be in sufficient condition for the applicator to travel across it.

Figure 4.3 ***Spray-on Mortar Invert Repair and Cast Mortar Liner (Source FHWA and MNDOT)***

15.5 Pre and Post Installation Inspection of Cured in Place Pipe Liners (CIPP), Cast, and Spray-on Liners

Inspection of pipes both during the design phase and construction should be part of all culvert and storm sewer projects. However, with storm sewer projects or small diameter culverts access is limited and possibly dangerous. In those cases, consider requiring pre- and post-installation televising of the culvert or storm sewer as part of the contract special provisions. For storm sewer, this may help locate previously unidentified direct connections sewer pipe in spans between manholes. These connections may otherwise be inadvertently covered by the liner.

15.6 Design Requirements for Cured in Place Pipe Liners (CIPP), Cast, and Spray-on Liners

Prior to specifying cured in place pipe liners (CIPP), cast liners, spray-on liners, or other non-standard culvert repair or rehabilitation, the designer shall carefully verify that a conventional slipliner will not meet the needs of the site. Document the reasoning conventional sliplining will not work in the project DSR. As stated previously, if a non-standard repair or rehabilitation is ultimately proposed for a project, notify one of the Statewide Drainage Engineers in RDSU. The Statewide Drainage Engineer will consult with the BOS and/or MMS as necessary to determine the design and materials requirements specific to the project and the appropriateness of the proposed method to the location in question.

15.6.1 Structural Design Requirements and Submittals for Cured in Place Pipe Liners (CIPP), Cast, and Spray-on Liners

Claims that CIPP, spray liners or similar non-traditional methods of lining culverts "creates a structural pipe within a pipe" does not absolve the designer from verifying, or causing to be verified, the structural integrity of the repair. As with any culvert, minimum anticipated loading conditions need to be verified. The analysis should assume a fully deteriorated pipe and that the liner is carrying the full loading conditions of the site.

At this time, the following minimum design conditions shall be considered if a culvert or storm sewer is lined by methods other than sliplining such as CIPP, Cast or Spray-on Liners:

- **Pipes 48-Inch Equivalent Diameter and less** can be verified by empirical analysis for structural capacity, stamped by a professional engineer registered in the state of Wisconsin and submitted to the project for review 14 days prior to delivery of the material. The analysis should assume a fully deteriorated pipe. For CIPP systems the structural analysis for pipes under 48-inches shall be performed using Appendix XI from ASTM F1216-16 Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube. For other non-traditional methods of culvert rehabilitation, such as cast or spray-on liners, a manufactured-recommended empirical analysis for structural capacity can be utilized.
- **Pipes larger than 48-Inch Equivalent Diameter** cannot be lined with spray, cast or similar liner systems. CIPP installations larger than 48-inch equivalent diameter require a site-specific numerical (finite-element) structural analysis that incorporates soil boring data from the site and any additional anticipated loadings from dead, live, or adjacent foundation sources, stamped by a professional engineer registered in the state of Wisconsin and submitted to the project for review 30 days prior to the delivery of the material. MMS can assist with reviews if needed. It is recommended that a geotechnical subsurface investigation be performed during the design process and an initial liner analysis be performed by the design engineer to determine the feasibility of lining pipes greater than 48 inches in diameter using CIPP methods. The geotechnical subsurface investigation should provide the necessary level of detail to allow the accurate computational analyses of pipe lining design. The actual geology and site conditions will determine how many, and what spacing of, borings are required.

15.6.2 Hydraulic Design Requirements for Cured in Place Pipe Liners (CIPP), Cast, and Spray-on Liners

The hydraulics impacts of CIPP, cast, and spray liners needs to be evaluated by the designer. While these liners reduce the interior diameter less than a slipliner in most cases, the Manning's roughness may not decrease as significantly. For example, when lining a corrugated metal pipe with CIPP, the CIPP may reflect the corrugations creating a Manning's roughness higher than the 0.009 to 0.011 expected for a smooth lined pipe. For CIPP installation in concrete pipe a Manning's roughness of 0.01 is to be used. For CIPP lining of corrugated metal pipe a Manning's roughness of 0.018 is to be used. See FDM 13-45-10.4.1.2 for additional guidance on liner hydraulics.

***Figure 15.4** Roughness within CIPP Lined Culvert*

15.7 Cost Considerations for Cured in Place Pipe Liners (CIPP), Cast, and Spray-on Liners

On average CIPP, cast, and spray-on liners will cost more to install than a conventional culvert slipliner. The designer shall carefully verify the hydraulics of a conventionally sliplined culvert will not meet the conditions of the site. A slight raise in headwater may be acceptable for culverts outside regulated floodplain areas so long as the impacts of the headwater increase are considered. CIPP and other non-traditional liners may become more cost effective where multiple culverts or a storm sewer system are lined where the cost of mobilizing specialized equipment is spread out. That said, the designer shall consider the life cycle and risk of these repairs versus conventional sliplining where, in essence, a "new" pipe meeting intended design loads is placed.

FDM 13-45-20 Trenchless Installation of New or Replacement Culvert Pipe and Storm Sewer *November 30, 2018*

20.1 Introduction

To this point this section has focused on repair or rehabilitation of culverts and storm sewers using trenchless methods. There are, however, trenchless methods of construction available to replace or install new culverts and storm sewer where traditional open-cut excavations are impossible or undesirable. The following is intended as an introduction to these technologies and to provide guidelines for WisDOT's minimum project requirements for these installations. While one or more of these methods may meet the needs of a particular installation, each of these methods has its own advantages and limitations. As stated previously, when one of these methods, or similar non-standard repair or rehabilitation is proposed for a project, notify one of the Statewide Drainage Engineers in RDSU. The Statewide Drainage Engineer will consult with the BOS and/or MMS as necessary to determine the design and materials requirements specific to the project and the appropriateness of the proposed method to the location in question.

With the installation of new storm sewer or culverts through trenchless methods the primary design concern will be if the installed pipe can handle the installation loads. This is not just a matter of specifying a higher-class pipe or a "jacking pipe". The pipe should be analyzed by a professional engineer registered in the state of Wisconsin to verify it can handle the intended construction and in place loading. For RCP used in jacked installations, consider requiring documentation and observation of quality control testing of the pipe that will be installed. It also must be determined whether a casing pipe should be installed. The need for a casing pipe should be determined based on the specific installation considering such factors as; the method of trenchless installation, risk of damaging a pipe driven without a casing, installation tolerance, risk of joint separation, the effects of abandoning and reboring a new pipeline for installations stuck or severely off alignment, or other unanticipated events that may result in disruption to the overlying roadway. In most cases the trenchless installation of a culvert should include a casing and the annular space between the casing and carrier pipe should be grouted.

Another consideration is construction access. Trenchless construction often may still require excavation for bore and receiving pits and space is needed to stage materials and to set and brace trenchless equipment.

Some additional considerations for trenchless construction include:

- Depth to groundwater
- Required dewatering
- Required geotechnical information
- Site constraints, space considerations, need for easements
- Potential obstructions
- Utility conflicts
- Monitoring of settlement and heave
- Monitoring of adjacent structures
- Monitoring of vibration, especially with pipe ramming
- Appropriateness of various trenchless methods

20.2 Environmental Considerations

FDM 13-45-5.6 describes various environmental considerations when planning a trenchless rehabilitation, repair or replacement project. For trenchless replacement, some additional considerations include preventing the discharge of spoils and lubricants (usually bentonite or polymer mixtures). "Frac-out, or inadvertent return of drilling lubricant, is a potential concern when the horizontal direction drilling (HDD) is used under sensitive habitats, waterways, and areas of concern for cultural resources. The HDD procedure uses bentonite slurry, a fine clay material as a drilling lubricant. The bentonite is non-toxic and commonly used in farming practices, but benthic invertebrates, aquatic plants, and fish and their eggs can be smothered by the fine particles if bentonite were discharged to waterways." (AASHTO TC3). Contract documents should include provisions for the contractor to maintain a spill response plan which includes provisions for handling and storage of materials to address potential environmental concerns. WisDOT is in the process of developing such spill plan language for all projects.

20.3 Geotechnical Considerations

No proposal for the placement of a trenchless pipe installation should be made without performing a geotechnical subsurface investigation. The geotechnical subsurface investigation should provide the necessary level of detail to allow the accurate computational analyses of pipe lining design. The actual geology and site conditions will determine how many, and what spacing of, borings are required. More complex projects may require a complete Geotechnical Baseline Report (GBR), dependent on project needs. It is advisable to secure recommendations within the geotechnical investigation regarding the feasibility of the concepts under consideration. Some additional considerations related to soils include: “Is the proposed [trenchless construction] equipment compatible with the anticipated soil conditions? Where is the water table? Can the equipment function in unstable ground conditions? Or, will the soil conditions need to be stabilized prior to the trenchless process being employed? If so, how? For example, will the soil need to be dewatered? Is dewatering reasonable at the specified project site? Are contaminated soils or groundwater anticipated? What is the likelihood of ground heaving or settlement? Need to establish allowable limits for ground movement and need to determine how ground movement will be measured” (Caltrans 2014).

20.4 Trenchless Construction Methods

20.4.1 Pipe Jacking

Pipe jacking is a method of tunnel construction where hydraulic jacks in a pit or drive shaft are used to push pipes through the ground, it is one of the first and most common trenchless methods of installing drainage facilities. On WisDOT projects pipe jacking is usually accompanied by a boring operation to excavate soils at the face of the operation and to remove the spoils. Space to configure jacking and receiving pits need to be of sufficient size to set and secure the jack and to stage materials. Jack and boring reinforced concrete pipe without a casing can be done considering the risks described in FDM 13-45-20.1. When jack and boring reinforced concrete pipe without a casing the driving ends of the pipe should be properly protected against spalling and other damage, and intermediate joints should be similarly protected. “The axial or thrust jacking loads are transmitted from one pipe section to another through the joint surfaces. It is essential that the pipe ends are parallel so that there will be a relatively uniform distribution of forces around the periphery of the pipe. Specifying a higher class of pipe provides little or no gain in axial crushing resistance” (Caltrans 2014).

As described in FDM 13-45-20.1, the voids between casing and carrier pipe should be grouted after completion of a jacking operation. Since the principal risk associated with trenchless methods is settlement of the overlying fill, additional consideration should be given to providing grout ports within the casing itself to allow the filling of any voids created by collapse while pushing the pipeline. “If there is a possibility of the excavation face collapsing, various soil stabilization techniques, including dewatering and grouting, may be required.” (AASHTO TC3).

***Figure 20.1** Jack and Bore Operations (Source AASHTO TC3)*

20.4.2 Microtunneling

For long runs, where a high level of accuracy is required, and/or in soils with a higher risk of collapse at the excavation face, jacking operations can be coupled with microtunneling. Microtunneling is a type of pipe jacking where the steering and excavation equipment are operated remotely. This process provides continuous support to the excavation face. Spoils are generally removed by auger or mixed and pumped as a slurry.

***Figure 20.2** Microtunneling Machine (Source AASHTO TC3)*

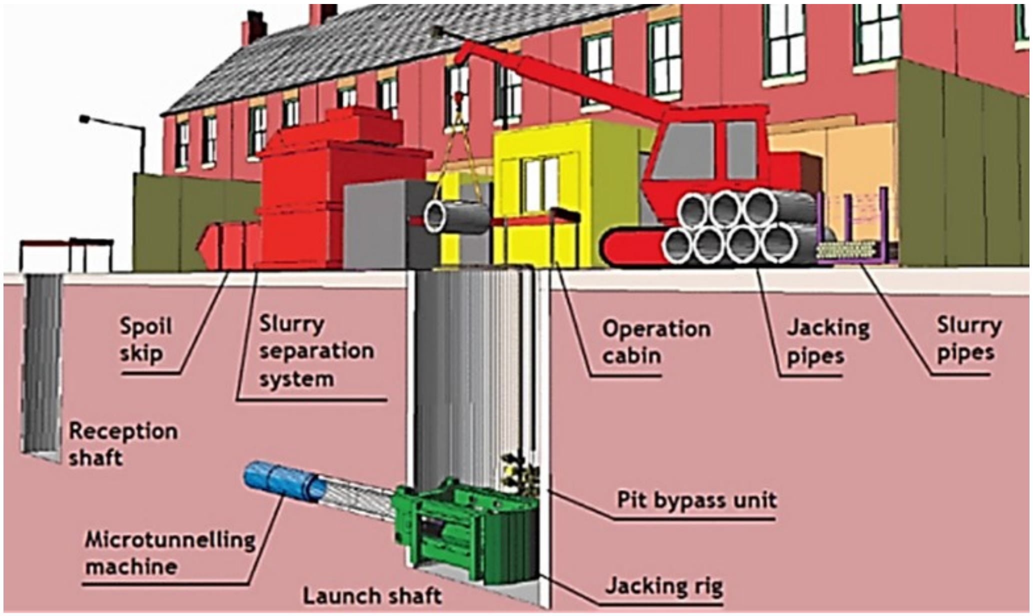


***Figure 20.3* Microtunneling (Source AASHTO TC3)**

20.4.3 Pipe Ramming

With pipe ramming a pneumatic hammer pushes an open-ended steel casing that is cleaned out during and after completion of pipe installation. The ramming hammer is attached to the casing with tensioning straps and pneumatic percussive blows drive the casing. Depending on the situation, pipe ramming can be faster than jack and bore installations. Ramming is generally unguided and is not as accurate as other methods. In some special applications pipe ramming can be guided for greater accuracy. Pipe ramming does not have the same space requirements for installation equipment and can be used in difficult site conditions. When considering pipe ramming, the impact of the noise should be considered.

This method should only be used to install casing. Ramming of reinforced concrete pipe on WisDOT projects is not allowed.

***Figure 20.4* Pipe Ramming (Source AASHTO TC3)**

20.4.4 Pipe Bursting

With pipe bursting, an expansion tool is guided through the old pipe and pushes it out of the way while a new pipe is guided through. This method works on pipes that will fracture such as cast iron, clay, and unreinforced concrete. CMP cannot be easily removed and replaced by pipe bursting. With CMP pipe, other methods should be considered.

20.4.5 Pipe Swallowing/Pipe Crushing

Pipe swallowing and pipe crushing both involve installing a casing around the existing pipe. This may be advantageous where environmental or hydraulic considerations make it desirable to install the new pipe in exactly the same horizontal location as the existing. With pipe swallowing, an oversized casing is rammed over the existing culvert. After installation is complete, the old culvert and spoils are removed, and the new culvert can be inserted. Pipe swallowing may be advantageous in correction of an existing culvert that is perched or where a lowering of the invert of a culvert is desired for hydraulics, fish passage or other environmental considerations. Pipe crushing is primarily used with CMP culverts. Blades in the casing crush the existing pipe as the casing is driven. The old culvert and spoils can then be extracted from within the casing similar to pipe swallowing.

20.4.6 Horizontal Directional Drilling

Horizontal directional drilling, also known as directional boring, uses a steerable drilling rig to install primarily high-density polyethylene or a similar flexible conduit or pipeline. Some systems can install metal pipelines as well. The bore path can be monitored and adjusted according to the location of the proposed utility or obstacles that are encountered. The pipeline can be welded as it is installed, virtually eliminating joints within a run.

On WisDOT projects horizontal direction drilling is primarily used by private utility companies for associated utility relocates or replacements. It may have applications for installing non-drainage WisDOT systems such as conduit for communications or power cabling. HDPE pipe can be drilled up to 48 inches in diameter so there may be a site-specific drainage consideration when other trenchless methods do not meet the needs of the project.

FDM 13-45-99 Resources and References *November 30, 2018*

99.1 Resources

The following is a brief list of useful resources for learning more about evaluating culverts, culvert liners and culvert repair in general.

Assessment

FHWA. (2010). Culvert Assessment and Decision-Making Procedures Manual. Lakewood, CO. http://www.ctiponline.org/publications/view_publication.aspx?id=125

FHWA (2014). Hydraulic Toolbox Version 4.2. [Offers hydraulic tools including a culvert assessment tool based on the 2010 Culvert Assessment and Decision-Making Procedures Manual.]

NCHRP. (2002). NCHRP Synthesis 303 Assessment and Rehabilitation of Existing Culverts. Washington, D.C.: Transportation Research Board (TRB). http://www.trb.org/Publications/Blurbs/161494.aspx

Design

Federal Highway Administration. Culvert hydraulic analysis program and supporting documentation, HY-8, Version 7.5. 2016.

Federal Highway Administration. Hydraulic Design of Highway Culverts Hydraulic Design Series Number 5 (HDS 5) Third Edition, FHWA-HIF-12-026. 2012.

FHWA (2006). Hydraulic Engineering Circular No. 14, Third Edition Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC-14), Third Edition, FHWA-NHI-06-086. 2006.

FHWA (2014). Hydraulic Toolbox Version 4.2. [Offers hydraulic tools including a culvert assessment tool based on the 2010 Culvert Assessment and Decision-Making Procedures Manual.]

Wisconsin Department of Transportation. Approved Products List, Culvert Pipe Liners – Prequalification to Manning's Coefficient, https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

Repair and Rehabilitation

Caltrans. (Updated 2014). Caltrans Supplement to FHWA Culvert Repair Practices Manual, Design Information Bulletin No. 83-04. http://www.dot.ca.gov/hq/oppd/dib/dib83-02.pdf

Donaldson, Bridget M, Andrew J. Baker (2008). Understanding the Environmental Implications of Cured-in-Place Pipe Rehabilitation Technology.

FHWA. (1995). Culvert Repair Practices Manual, Pub. No. FHWA-RD-94-096. (Archived – Out of Print).

FHWA. (2005). Culvert Pipe Liner Guide and Specifications, FHWA-CFL/TD-05-003. https://flh.fhwa.dot.gov/other/documents/culvert-pipe-liner-guide-and-specs.pdf

NASSCO CIPP Committee (2008). Guidelines for the Use and Handling of Styrenated Resins in Cured-In-Place-Pipe.

Trenchless Construction

AASHTO TC3 Transportation Curriculum Coordination Council. Trenchless Technology (Online Course - 5.5 PDHs). This course provides an introduction to trenchless technology, including its purpose and history, and explains the applications, permitting considerations, construction practices, and inspection guidelines. The development of this course was provided by Iowa DOT in partnership with TC3. https://training.transportation.org/item_details.aspx?ID=3707

99.2 References

AASHTO TC3 Transportation Curriculum Coordination Council. Trenchless Technology Online Course. https://training.transportation.org/item_details.aspx?ID=3707

Caltrans, Caltrans Supplement to FHWA Culvert Repair Practices Manual, Design Information Bulletin No. 83-04., Updated 2014. http://www.dot.ca.gov/hq/oppd/dib/dib83-02.pdf

Minnesota Department of Transportation, Culvert Repair Best Practices, Specifications, and Special Provisions – Best Practices Guidelines, Updated, January 2014. http://www.dot.state.mn.us/research/TS/2014/201401.pdf

Culvert Liner Hydraulic Check Example

The following examples are meant to assist in the analysis of the hydraulic impacts of a culvert lining project. For project designers, this procedure can be used to determine the feasibility of sliplining a culvert. This procedure can also be used by a contractor or supplier to verify that an intended liner will meet WisDOT hydraulic performance requirements. In the case of a contractor supplied liner, the Manning's roughness value approved by WisDOT must be used for the specific liner to be placed. WisDOT has several approved segmental slipliners on the approved products list.

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx.

For project designers, because the specific culvert liner product may not be known, start by assuming a worst-case Manning's value of 0.012. This is the high end of the Manning's values on WisDOT's approved list of liners. If the desired results are not achieved, consider improving the inlet with a beveled headwall and/or set a maximum allowable Manning's roughness and minimum liner diameter in the project contract documents. Ideally multiple liners on the approved products list meet the modified design conditions. See FDM 19-1-5 regarding proprietary products.

Example 1 – Sliplining a Culvert with Known Hydrological Conditions

Given:

WisDOT is proposing to slipline a 48-inch diameter corrugated metal pipe (CMP). The pipe is 80 feet long, is not located in a floodplain or drainage district, and has not been previously lined. The culvert is normally dry so WDNR has not identified aquatic organism passage concerns associated with the site. The downstream channel has the approximate shape of a trapezoidal ditch with a 2-foot wide bottom and 3:1 sideslopes.

Additional project conditions are as follows:

Q25 (Design Storm) = 75 cfs

Q100 (Check Storm) = 100 cfs

Allowable Headwater = Existing at Design Storm or Hw/D < 1.5

Roadway Centerline Elevation = 912 ft

Downstream Channel Slope = 2%

	Existing Culvert	Proposed Liner
Manning's Roughness	0.024	0.010
Internal Diameter	48 Inch	36 Inch
Culvert Inlet Invert Elevation	905 ft.	905.3 ft. (See Proposed Liner Conditions)
Culvert Outlet Invert Elevation	904.2 ft.	904.5 ft. (See Proposed Liner Conditions)

Existing Conditions:

The first step in the process is to analyze the hydraulics of the existing culvert. In this example, Federal Highway's HY-8 software (available for free) will be utilized. Figure A10.1 shows the inputs and results of the analysis.

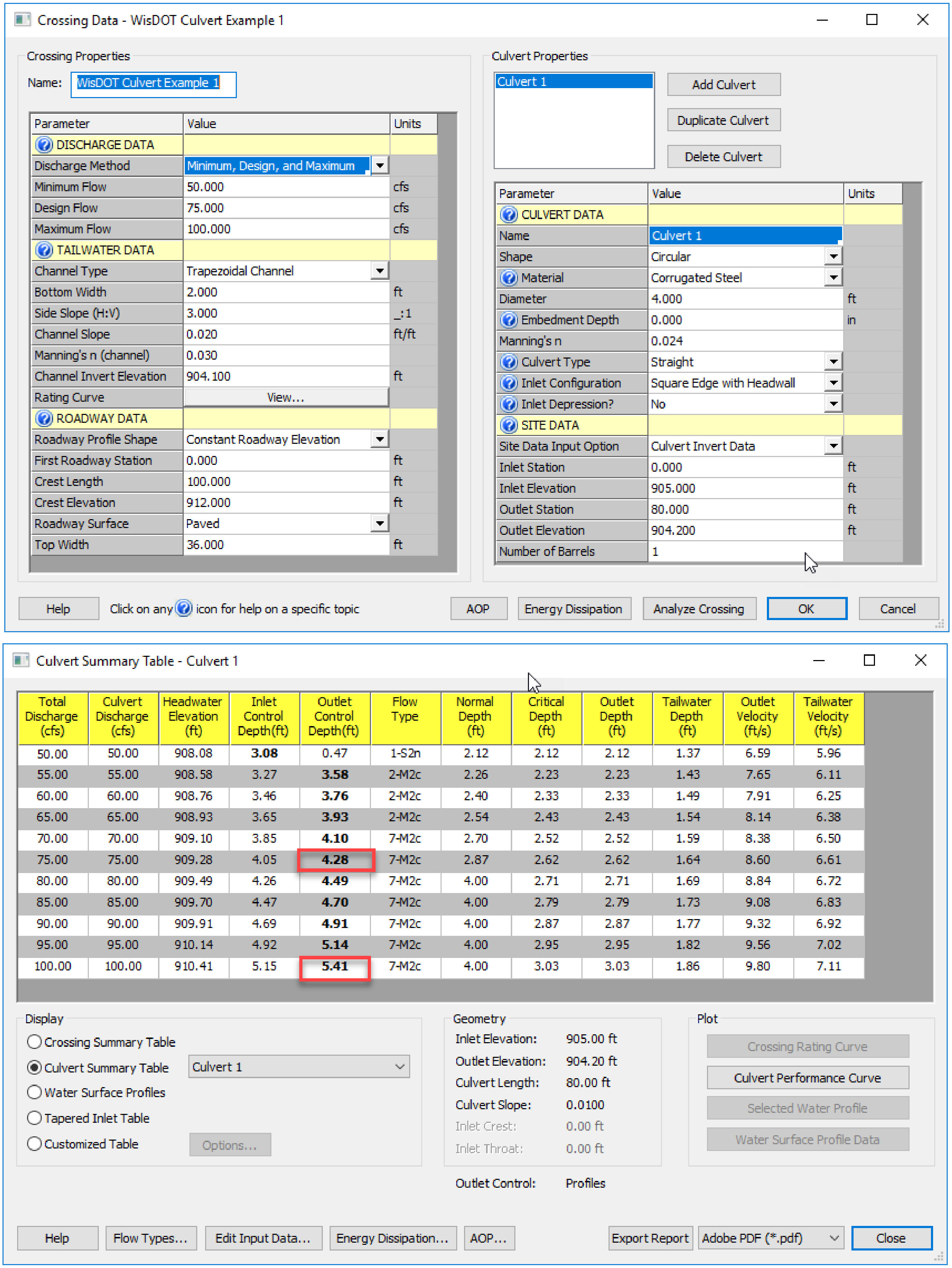
Based on the results of the analysis, the headwater at Q25 is 909.28 ft (4.28 ft deep) and 910.41 ft (5.41 ft deep) at Q100. The roadway does not overtop and Hw/D is <1.5 at the Q25 design storm.

Post Liner Conditions:

A 36-inch culvert liner is proposed with a Manning's roughness of 0.010. The outer diameter of the liner is 41.2 inches. The liner wall is therefore 2.6 inches thick or 0.22 ft. Allowing for construction tolerances and joints, it is assumed that the liner will raise the culvert invert at least 0.3 ft to 905.3 ft. Using the same site conditions, the proposed liner is analyzed in HY-8 by adjusting the culvert inverts, diameter, and Manning's roughness. Figure A10.2 shows the inputs and results of the hydraulic analysis of the liner.

Based on the results of the analysis as shown in Figure A10.2, the headwater at the Q25 design storm is 911.74 ft (6.44 ft deep + liner thickness) and 912.17 ft (6.87 ft deep + liner thickness) at Q100. The roadway is overtopping at Q100, headwater has increased by 2.46 ft for the design storm, and Hw/D is >1.5. The liner is in inlet control and does not meet the needs of the site. Ideally this would be caught in design and a specific liner could be specified, an inlet bevel could be analyzed, or the culvert may need to be replaced by conventional or trenchless methods.

This example demonstrates the importance of checking the hydraulic capacity of a liner, and not just performing a Manning's roughness based comparison. Had WisDOT's past standard of comparing Manning's values been used, the liner would have been accepted because it would be shown to provide 112% of the Manning's full flow capacity.



Crossing Data - WisDOT Culvert Example 1

Crossing Properties

Name: WisDOT Culvert Example 1

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	50.000	cfs
Design Flow	75.000	cfs
Maximum Flow	100.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	2.000	ft
Side Slope (H:V)	3.000	_:1
Channel Slope	0.020	ft/ft
Manning's n (channel)	0.030	
Channel Invert Elevation	904.100	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	912.000	ft
Roadway Surface	Paved	
Top Width	36.000	ft

Culvert Properties

Culvert 1

Add Culvert | Duplicate Culvert | Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 1	
Shape	Circular	
Material	Corrugated Steel	
Diameter	4.000	ft
Embedment Depth	0.000	in
Manning's n	0.024	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	905.000	ft
Outlet Station	80.000	ft
Outlet Elevation	904.200	ft
Number of Barrels	1	

Help | Click on any icon for help on a specific topic | AOP | Energy Dissipation | Analyze Crossing | OK | Cancel

Culvert Summary Table - Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	50.00	908.08	**3.08**	0.47	1-S2n	2.12	2.12	2.12	1.37	6.59	5.96
55.00	55.00	908.58	3.27	**3.58**	2-M2c	2.26	2.23	2.23	1.43	7.65	6.11
60.00	60.00	908.76	3.46	**3.76**	2-M2c	2.40	2.33	2.33	1.49	7.91	6.25
65.00	65.00	908.93	3.65	**3.93**	2-M2c	2.54	2.43	2.43	1.54	8.14	6.38
70.00	70.00	909.10	3.85	**4.10**	7-M2c	2.70	2.52	2.52	1.59	8.38	6.50
75.00	75.00	909.28	4.05	**4.28**	7-M2c	2.87	2.62	2.62	1.64	8.60	6.61
80.00	80.00	909.49	4.26	**4.49**	7-M2c	4.00	2.71	2.71	1.69	8.84	6.72
85.00	85.00	909.70	4.47	**4.70**	7-M2c	4.00	2.79	2.79	1.73	9.08	6.83
90.00	90.00	909.91	4.69	**4.91**	7-M2c	4.00	2.87	2.87	1.77	9.32	6.92
95.00	95.00	910.14	4.92	**5.14**	7-M2c	4.00	2.95	2.95	1.82	9.56	7.02
100.00	100.00	910.41	5.15	**5.41**	7-M2c	4.00	3.03	3.03	1.86	9.80	7.11

Display: Crossing Summary Table | Culvert Summary Table (Culvert 1) | Water Surface Profiles | Tapered Inlet Table | Customized Table | Options...

Geometry

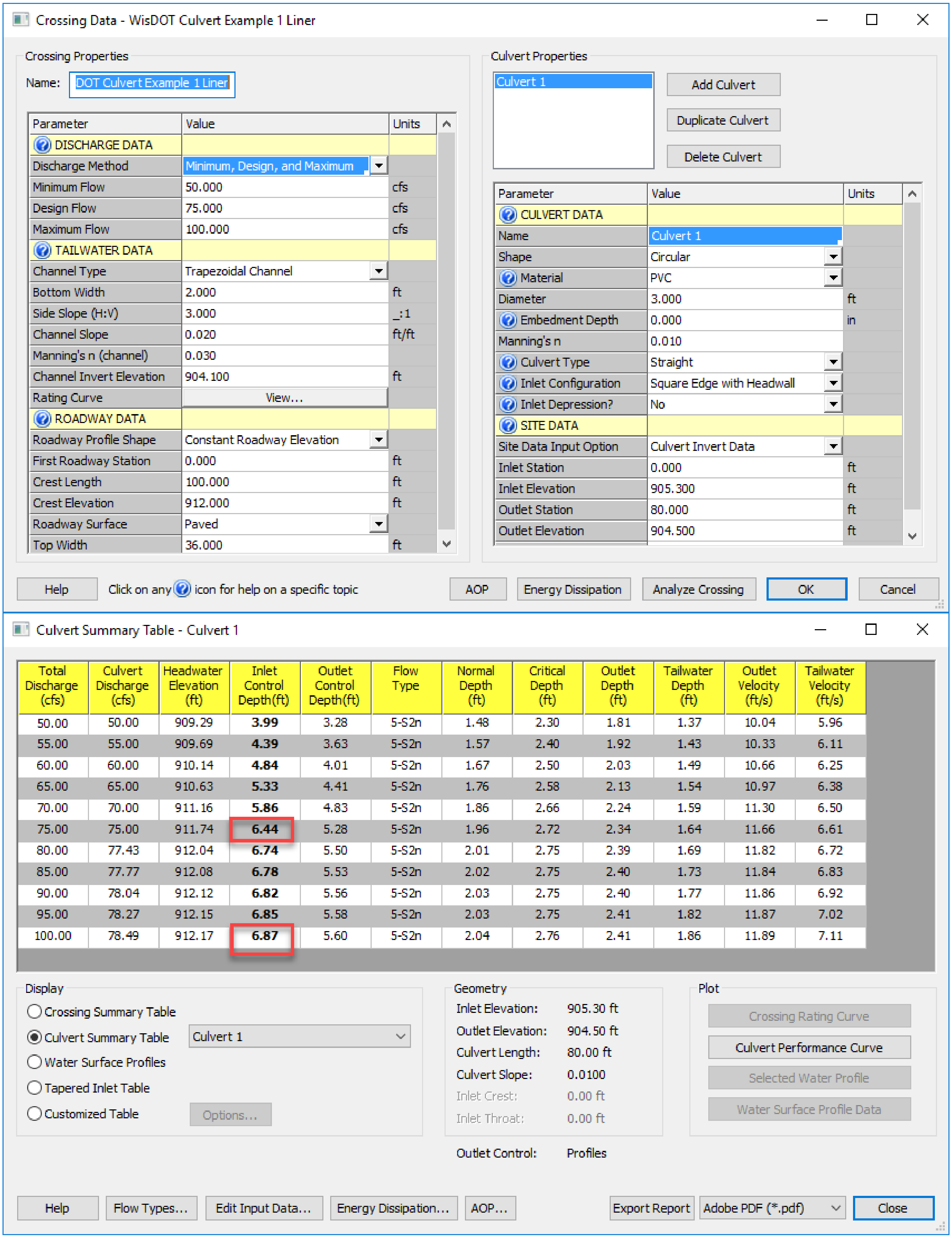
Inlet Elevation: 905.00 ft
Outlet Elevation: 904.20 ft
Culvert Length: 80.00 ft
Culvert Slope: 0.0100
Inlet Crest: 0.00 ft
Inlet Throat: 0.00 ft

Outlet Control: Profiles

Plot: Crossing Rating Curve | Culvert Performance Curve | Selected Water Profile | Water Surface Profile Data

Help | Flow Types... | Edit Input Data... | Energy Dissipation... | AOP... | Export Report | Adobe PDF (*.pdf) | Close

Figure A10.1 HY-8 Inputs and Results for Existing Condition - Example 1



Crossing Data - WisDOT Culvert Example 1 Liner

Crossing Properties

Name: DOT Culvert Example 1 Liner

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	50.000	cfs
Design Flow	75.000	cfs
Maximum Flow	100.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	2.000	ft
Side Slope (H:V)	3.000	_:1
Channel Slope	0.020	ft/ft
Manning's n (channel)	0.030	
Channel Invert Elevation	904.100	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	912.000	ft
Roadway Surface	Paved	
Top Width	36.000	ft

Culvert Properties

Culvert 1

Add Culvert
Duplicate Culvert
Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 1	
Shape	Circular	
Material	PVC	
Diameter	3.000	ft
Embedment Depth	0.000	in
Manning's n	0.010	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	905.300	ft
Outlet Station	80.000	ft
Outlet Elevation	904.500	ft

Help | Click on any icon for help on a specific topic | AOP | Energy Dissipation | Analyze Crossing | OK | Cancel

Culvert Summary Table - Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	50.00	909.29	**3.99**	3.28	5-S2n	1.48	2.30	1.81	1.37	10.04	5.96
55.00	55.00	909.69	**4.39**	3.63	5-S2n	1.57	2.40	1.92	1.43	10.33	6.11
60.00	60.00	910.14	**4.84**	4.01	5-S2n	1.67	2.50	2.03	1.49	10.66	6.25
65.00	65.00	910.63	**5.33**	4.41	5-S2n	1.76	2.58	2.13	1.54	10.97	6.38
70.00	70.00	911.16	**5.86**	4.83	5-S2n	1.86	2.66	2.24	1.59	11.30	6.50
75.00	75.00	911.74	**6.44**	5.28	5-S2n	1.96	2.72	2.34	1.64	11.66	6.61
80.00	77.43	912.04	**6.74**	5.50	5-S2n	2.01	2.75	2.39	1.69	11.82	6.72
85.00	77.77	912.08	**6.78**	5.53	5-S2n	2.02	2.75	2.40	1.73	11.84	6.83
90.00	78.04	912.12	**6.82**	5.56	5-S2n	2.03	2.75	2.40	1.77	11.86	6.92
95.00	78.27	912.15	**6.85**	5.58	5-S2n	2.03	2.75	2.41	1.82	11.87	7.02
100.00	78.49	912.17	**6.87**	5.60	5-S2n	2.04	2.76	2.41	1.86	11.89	7.11

Display
- Crossing Summary Table
- Culvert Summary Table — Culvert 1
- Water Surface Profiles
- Tapered Inlet Table
- Customized Table — Options...

Geometry

Inlet Elevation:	905.30 ft
Outlet Elevation:	904.50 ft
Culvert Length:	80.00 ft
Culvert Slope:	0.0100
Inlet Crest:	0.00 ft
Inlet Throat:	0.00 ft
Outlet Control:	Profiles

Plot
- Crossing Rating Curve
- Culvert Performance Curve
- Selected Water Profile
- Water Surface Profile Data

Help | Flow Types... | Edit Input Data... | Energy Dissipation... | AOP... | Export Report | Adobe PDF (*.pdf) | Close

Figure A10.2 HY-8 Inputs and Results for Proposed Liner - Example 1

Example 2 – Sliplining a Culvert with Unknown Hydrological Conditions

Given:

WisDOT is proposing to slipline a 36-inch diameter corrugated metal pipe (CMP). The project is in the design phase so the initial condition is that no specific liner is proposed. The worst case of the allowable liner materials will be assumed as a starting point.

The pipe is 80 feet long, is not located in a floodplain or drainage district, and has not been previously lined. The pipe is 80 feet long, is not located in a floodplain or drainage district, and has not been previously lined. The culvert is normally dry so WDNR has not identified aquatic organism passage concerns associated with the site. The downstream channel has the approximate shape of a trapezoidal ditch with a 2-foot-wide bottom and 3:1 sideslopes.

Additional project conditions are as follows:

Q25 (Design Storm) = Undetermined – Assumed as Hw/D =1.5

Q100 (Check Storm) = Undetermined – Assumed as point of roadway overtopping

Allowable Headwater = Existing at Design Storm or Hw/D < 1.5

Roadway Centerline Elevation = 912 ft

Downstream Channel Slope = 0.5%

	Existing Culvert	Proposed Liner
Manning's Roughness	0.024	0.012 (First Attempt)
Internal Diameter	36 Inch	TBA
Culvert Inlet Invert Elevation	905 ft.	TBA (See Proposed Liner Conditions)
Culvert Outlet Invert Elevation	904.6 ft.	TBA (See Proposed Liner Conditions)

Existing Conditions:

The first step in the process is to analyze the hydraulics of the existing culvert. In this example, Federal Highway's free HY-8 software will again be utilized. Since design flows are undetermined they can be assumed by iteratively determining the flow and elevation at three check points; top of existing culvert, culvert at Hw/D = 1.5, and roadway overtopping. For this example, it is found that overtopping occurs at 66 cfs at elevation 912 ft, top of existing culvert is 30 cfs at elevation 908 ft, and Hw/D = 1.5 flow is 46.5 cfs at elevation 909.5 ft. These flows and elevations will be compared to the proposed lined conditions to determine the suitability of lining the culvert.

Post Liner Conditions:

A nominal 24-inch culvert liner is tried first with a maximum Manning's roughness of 0.012. The outer diameter of the liner is assumed at a worst case of 28.2 inches. The liner wall is therefore 2.1 inches thick or 0.175 ft. Allowing for construction tolerances and the thickness at joints it is assumed the liner will raise the culvert invert at least 0.25 ft to 905.25 ft. Using the same site conditions, the liner is analyzed in HY-8 by adjusting the culvert inverts, diameter, and Manning's roughness.

Where the design storms are not determined results will be analyzed based on the flows at the three suggested check points. Results for this example were as follows.

Existing Top of Culvert Flow = 30 cfs	Liner Flow Elevation = 910.20 ft
Existing Culvert Hw/D=1.5 = 46.5 cfs	Liner Flow Elevation = 912.10 ft (OVERTOPPING)
Roadway Overtopping = 66 cfs	Liner Flow Elevation = 912.21 ft (OVERTOPPING)

A 24-inch liner reduces the capacity of the culvert by nearly half. Assuming the design storm was at Hw/D=1.5, the assumed design storm of 46.5 cfs overtops the roadway. Trying different inlet configurations and Manning's roughness still does not meet the desired design conditions. The analysis needs to be rerun with a 27 inch or 30 inch I.D. liner.

Similarly, a 27-inch ID liner at n=0.012 with a standard inlet configuration operates in inlet control and does not meet the project conditions. Adding an inlet bevel and using a liner with a Manning's roughness no higher than 0.009 still results in roadway overtopping at the assumed 46.5 cfs design storm. Had just a Manning's comparison been made, the 27-inch ID liner would have shown to provide 124% of the existing culvert capacity when in fact it likely worsens the conveyance of water at the site.

A 30-inch liner will be required. Assuming a worst-case invert rise at 0.3 ft and starting with n=0.012 the results are as follows:

Existing Top of Culvert Flow = 30 cfs	Liner Flow Elevation = 908.51 ft
Existing Culvert Hw/D=1.5 = 46.5 cfs	Liner Flow Elevation = 910.51 ft (1-foot change assumed at design flow)
Roadway Overtopping = 66 cfs	Liner Flow Elevation = 912.11 ft (OVERTOPPING)

This still does not meet the desired performance but it is much closer. Adjustments to the Manning's roughness and inlet configuration can be analyzed. Using a Manning's roughness of 0.01 and a 1.5:1 beveled inlet results in the following:

Existing Top of Culvert Flow = 30 cfs	Liner Flow Elevation = 908.18 ft
Existing Culvert Hw/D=1.5 = 46.5 cfs	Liner Flow Elevation = 909.61 ft
Roadway Overtopping = 66 cfs	Liner Flow Elevation = 911.99 ft

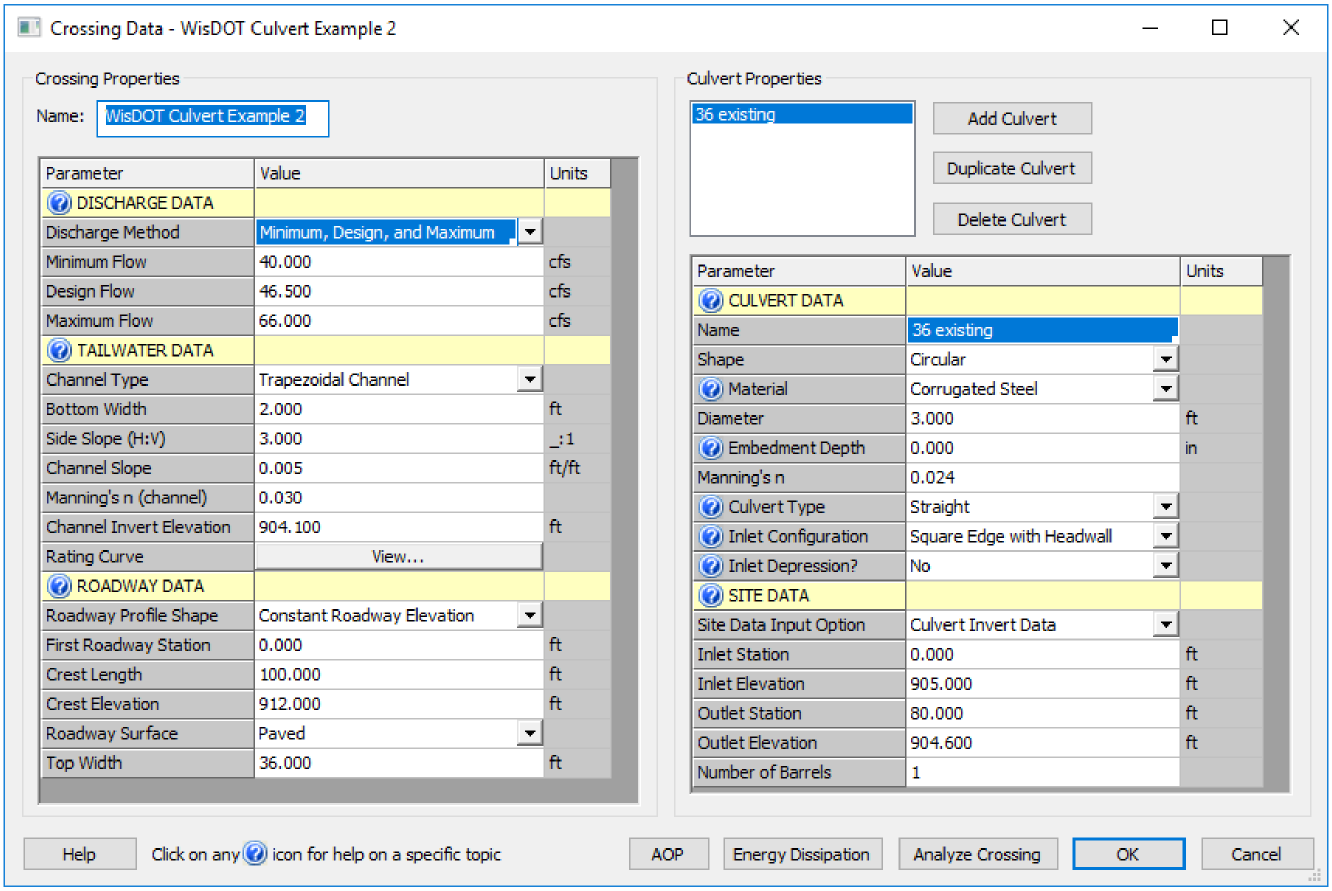
Depending on the situation, and when not in a regulated floodplain, this small increase in headwater may be fine. If necessary the design could be refined and a Manning's roughness of 0.009 could be specified. A few WisDOT standard liners meet this condition. For this example, we will assume this slight rise in headwater elevation meets the project conditions and a Manning's roughness of 0.01 or less is adequate.

Checking Liner Performance:

Assuming the project specifications require the liner to meet or exceed 95% of the original culvert capacity, two additional checks are performed. Determining the flow at Hw/D=1.5 of the original liner and flow at culvert full. This can be done by iteratively entering flow values into the minimum flow, design flow and maximum flow sections of the HY-8 input screen.

From this it is found that at the existing culvert Hw/D =1.5 (Elevation = 909.5 ft) the flow is 45.5 cfs and the culvert flowing full value is 33 cfs (See Figure 10.4). At Hw/D =1.5 a flow of 45.5 cfs is 98% of the original flow. At the culvert flowing full conditions the flow is essentially identical (Existing = 30 cfs at 908, Proposed = 33 cfs at the liner invert of 908.4 or 110% of the original flow). Both conditions are exceeding 95% of the design flow so any liner on the approved list with a Manning's roughness $\leq$ 0.01, 30-Inch I.D. and O.D. less than 34 inches should be sufficient so long as the inlet configuration is improved. A smaller O.D. liner may be required for constructability but the contractor can make that determination during liner verification.

To improve the inlet, a headwall can be formed at the inlet end to create the 1.5:1 bevel. This will need to be detailed in the project plans and special provisions and roadside safety should be considered. Alternately one manufacturer offers an improved inlet bell fitting that serves the same function as the bevel. In that case, the Materials Management Section will need to be involved to get approval for a one-time project exception for a proprietary item, or it may be possibly to develop an SPV with the special fitting as an alternative option to a cast in place headwall. See FDM 19-1-5 and contact the MMS for guidance on proprietary items.



Crossing Data - WisDOT Culvert Example 2

Crossing Properties

Name: WisDOT Culvert Example 2

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	40.000	cfs
Design Flow	46.500	cfs
Maximum Flow	66.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	2.000	ft
Side Slope (H:V)	3.000	_:1
Channel Slope	0.005	ft/ft
Manning's n (channel)	0.030	
Channel Invert Elevation	904.100	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	912.000	ft
Roadway Surface	Paved	
Top Width	36.000	ft

Culvert Properties

36 existing

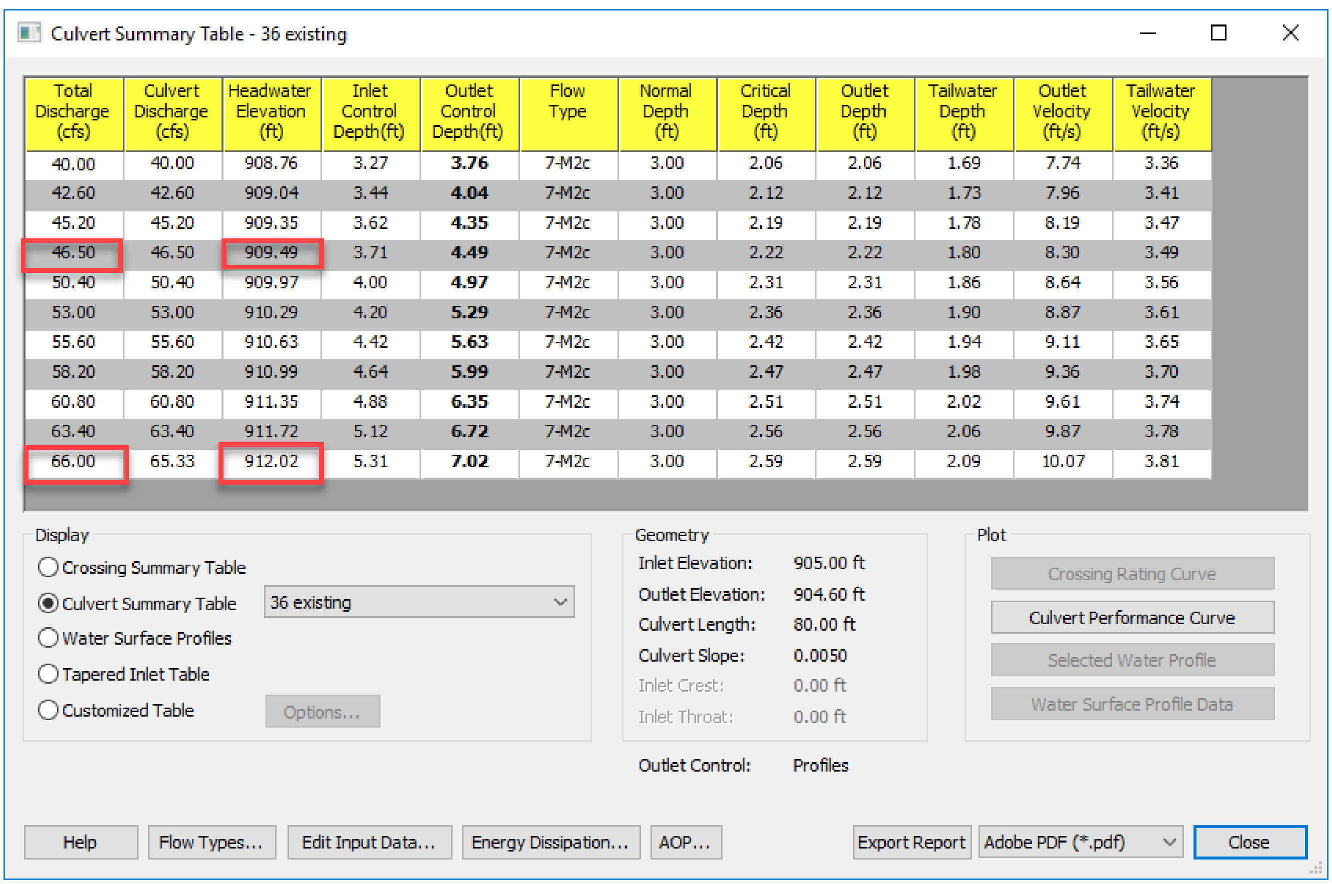
Add Culvert

Duplicate Culvert

Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	36 existing	
Shape	Circular	
Material	Corrugated Steel	
Diameter	3.000	ft
Embedment Depth	0.000	in
Manning's n	0.024	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	905.000	ft
Outlet Station	80.000	ft
Outlet Elevation	904.600	ft
Number of Barrels	1	

Help | Click on any icon for help on a specific topic | AOP | Energy Dissipation | Analyze Crossing | OK | Cancel



Culvert Summary Table - 36 existing

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
40.00	40.00	908.76	3.27	**3.76**	7-M2c	3.00	2.06	2.06	1.69	7.74	3.36
42.60	42.60	909.04	3.44	**4.04**	7-M2c	3.00	2.12	2.12	1.73	7.96	3.41
45.20	45.20	909.35	3.62	**4.35**	7-M2c	3.00	2.19	2.19	1.78	8.19	3.47
46.50	46.50	909.49	3.71	**4.49**	7-M2c	3.00	2.22	2.22	1.80	8.30	3.49
50.40	50.40	909.97	4.00	**4.97**	7-M2c	3.00	2.31	2.31	1.86	8.64	3.56
53.00	53.00	910.29	4.20	**5.29**	7-M2c	3.00	2.36	2.36	1.90	8.87	3.61
55.60	55.60	910.63	4.42	**5.63**	7-M2c	3.00	2.42	2.42	1.94	9.11	3.65
58.20	58.20	910.99	4.64	**5.99**	7-M2c	3.00	2.47	2.47	1.98	9.36	3.70
60.80	60.80	911.35	4.88	**6.35**	7-M2c	3.00	2.51	2.51	2.02	9.61	3.74
63.40	63.40	911.72	5.12	**6.72**	7-M2c	3.00	2.56	2.56	2.06	9.87	3.78
66.00	65.33	912.02	5.31	**7.02**	7-M2c	3.00	2.59	2.59	2.09	10.07	3.81

Display

- Crossing Summary Table
- Culvert Summary Table — 36 existing
- Water Surface Profiles
- Tapered Inlet Table
- Customized Table — Options...

Geometry

Inlet Elevation:	905.00 ft
Outlet Elevation:	904.60 ft
Culvert Length:	80.00 ft
Culvert Slope:	0.0050
Inlet Crest:	0.00 ft
Inlet Throat:	0.00 ft
Outlet Control:	Profiles

Plot

Crossing Rating Curve

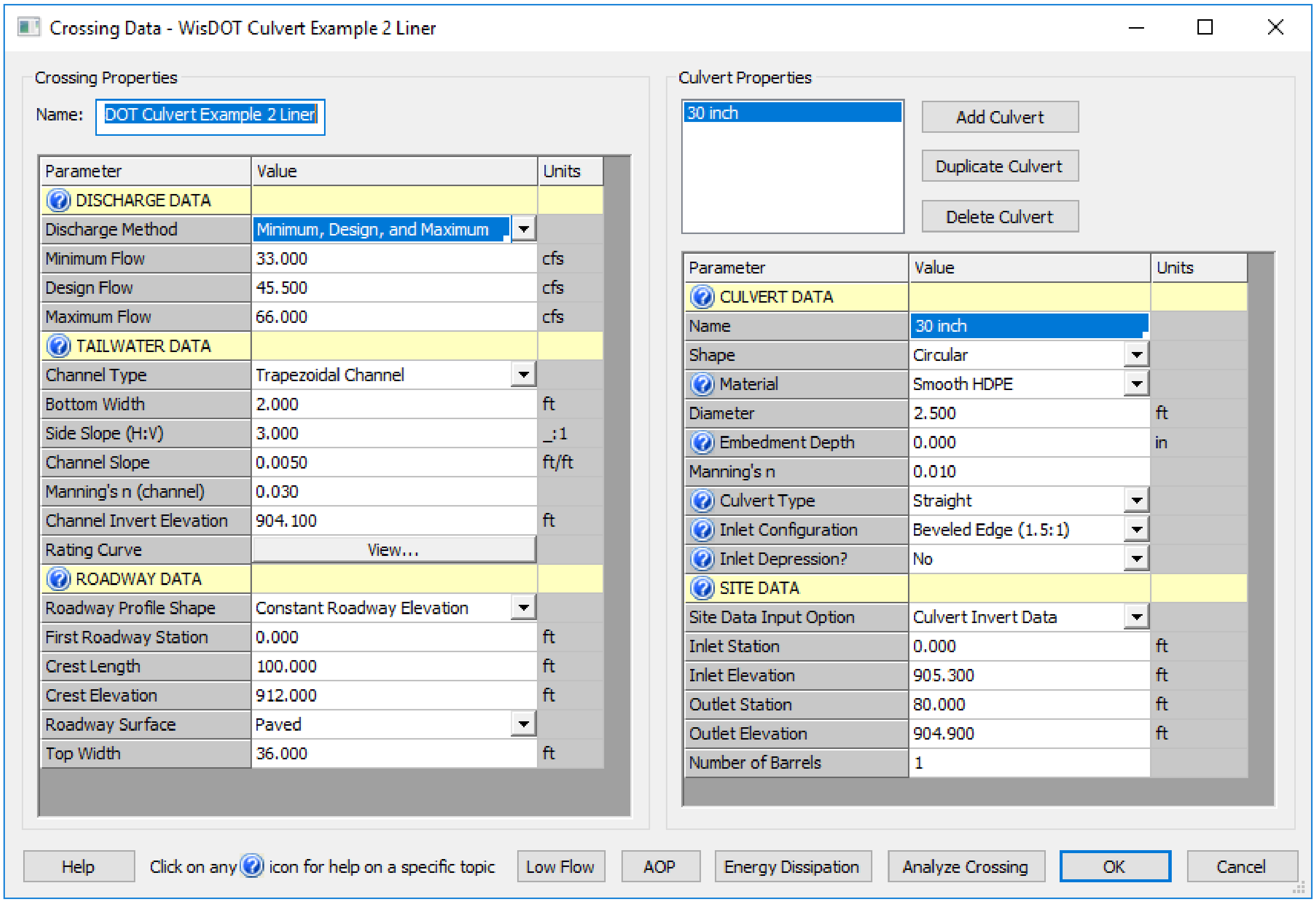
Culvert Performance Curve

Selected Water Profile

Water Surface Profile Data

Help | Flow Types... | Edit Input Data... | Energy Dissipation... | AOP... | Export Report | Adobe PDF (*.pdf) | Close

Figure A10.3 HY-8 Inputs and Results for Existing Condition - Example 2



Crossing Data - WisDOT Culvert Example 2 Liner

Crossing Properties

Name: DOT Culvert Example 2 Liner

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	33.000	cfs
Design Flow	45.500	cfs
Maximum Flow	66.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	2.000	ft
Side Slope (H:V)	3.000	_:1
Channel Slope	0.0050	ft/ft
Manning's n (channel)	0.030	
Channel Invert Elevation	904.100	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	912.000	ft
Roadway Surface	Paved	
Top Width	36.000	ft

Culvert Properties

30 inch

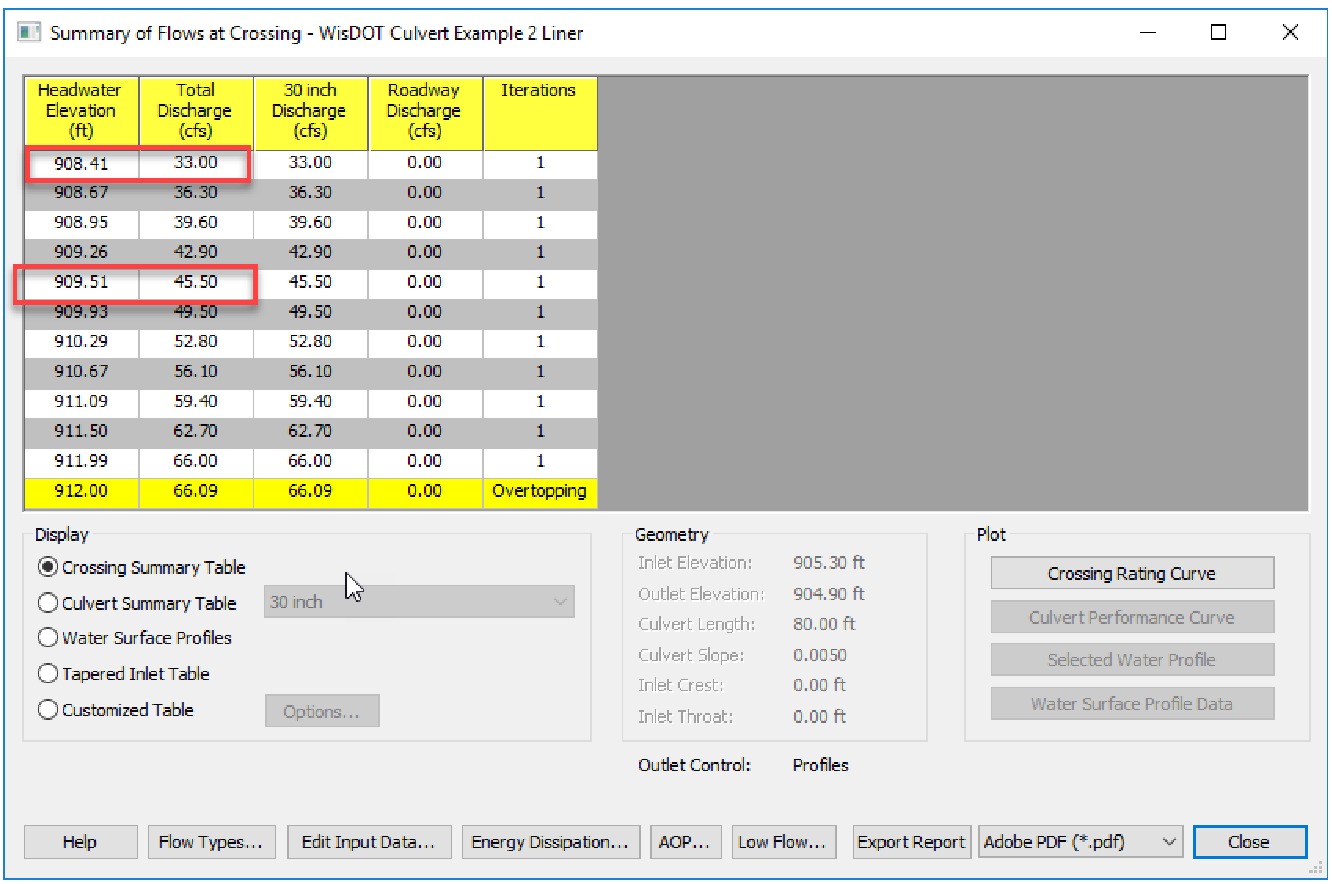
Add Culvert

Duplicate Culvert

Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	30 inch	
Shape	Circular	
Material	Smooth HDPE	
Diameter	2.500	ft
Embedment Depth	0.000	in
Manning's n	0.010	
Culvert Type	Straight	
Inlet Configuration	Beveled Edge (1.5:1)	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	905.300	ft
Outlet Station	80.000	ft
Outlet Elevation	904.900	ft
Number of Barrels	1	

Help | Click on any icon for help on a specific topic | Low Flow | AOP | Energy Dissipation | Analyze Crossing | OK | Cancel



Summary of Flows at Crossing - WisDOT Culvert Example 2 Liner

Headwater Elevation (ft)	Total Discharge (cfs)	30 inch Discharge (cfs)	Roadway Discharge (cfs)	Iterations
908.41	33.00	33.00	0.00	1
908.67	36.30	36.30	0.00	1
908.95	39.60	39.60	0.00	1
909.26	42.90	42.90	0.00	1
909.51	45.50	45.50	0.00	1
909.93	49.50	49.50	0.00	1
910.29	52.80	52.80	0.00	1
910.67	56.10	56.10	0.00	1
911.09	59.40	59.40	0.00	1
911.50	62.70	62.70	0.00	1
911.99	66.00	66.00	0.00	1
912.00	66.09	66.09	0.00	Overtopping

Display

- Crossing Summary Table
- Culvert Summary Table (30 inch)
- Water Surface Profiles
- Tapered Inlet Table
- Customized Table (Options...)

Geometry

Inlet Elevation:	905.30 ft
Outlet Elevation:	904.90 ft
Culvert Length:	80.00 ft
Culvert Slope:	0.0050
Inlet Crest:	0.00 ft
Inlet Throat:	0.00 ft

Outlet Control: Profiles

Plot

Crossing Rating Curve

Culvert Performance Curve

Selected Water Profile

Water Surface Profile Data

Help | Flow Types... | Edit Input Data... | Energy Dissipation... | AOP... | Low Flow... | Export Report | Adobe PDF (*.pdf) | Close

Figure A10.4 HY-8 Inputs and Results for Proposed Liner – Example 2