Drainage Design and Erosion Control Manual

Nebraska Department of Transportation

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The information contained in the Introduction, dated August 2006, has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Manual chapters and other reference material citations occurring since the latest publication of this chapter.

Introduction

1. PURPOSE OF MANUAL

This manual has been developed to provide guidance and assistance to the roadway designer and other **Nebraska Department of Transportation (NDOT)** personnel in the practices and procedures for the detailed design of roadway drainage and erosion control. The manual has been developed to meet the following principal objectives:

- Document **NDOT** guidelines with regard to drainage design criteria and erosion control practices.
- Define design criteria to guide judgments and decisions made by **Roadway Design Division** personnel.
- Describe the most current and effective design techniques and practices and to present charts, tables and other information useful to designers.

2. HOW TO USE THE MANUAL

This manual has been written to provide information for both the new designer and for the more experienced designer. General guidelines and design practices are described. For more detailed explanation of the topics, references are provided for the reader.

Throughout the manual, the words "shall", "should", and "may" are used to describe the appropriate application of various design techniques. The following definitions describe the proper application of these terms:

"Shall" is a mandatory condition; the designer will make every practical effort to follow the criteria. If it is impractical to follow the "shall" criteria, the designer needs to obtain authorization for a design exception (See the Roadway Design Manual).

"Should" is an advisory condition; the designer is recommended, not mandated, to follow the criteria. For situations where it is impractical to follow the "should" criteria, the designer needs to obtain **Assistant Design Engineer** approval and document the decision made.

"May" is a permissive condition; it is recommended that the designer make reasonable efforts to follow the design criteria. For situations where it is impractical to follow the "may" criteria, the designer does not need authorization for design variances.

Several formatting conventions have been used in the manual to aid the designer in locating information. When Exhibits are discussed in the text, the titles are highlighted, e.g., EXHIBIT 2.1. Individuals, sections, divisions, and other organizations with which interaction may be required appear in bold lettering, e.g., **Roadway Design Division Engineer**. References to material in other chapters of this manual are shown as: Chapter Two: Erosion and Sedimentation Control, for example.

3. MANUAL UPDATES

This manual will be updated on a regular basis to reflect new Nebraska policies. Policy letters will be issued between manual updates. These letters should be inserted at the beginning of the appropriate chapter(s); the information from the policy letters will be incorporated into the manual when it is updated.

4. SOURCES OF INFORMATION

This manual is a principal source of information providing general guidance on design guidelines and practices. Other sources of information are listed in the REFERENCES section found at the back of each Chapter. Suppliers of construction materials also may be used as sources of information.

Where possible, Internet connections have been given for reference materials cited in this manual. This connection will follow the first citation of a document on a page and will also appear in the REFERENCES section found at the back of the chapter.

The information contained in Chapter One: Drainage, dated August 2006, has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

Chapter One Drainage

This chapter discusses hydrology, hydraulics, and culvert design for highway drainage systems. Designers who must prepare plans in metric units should design in English units, soft convert their findings to metric, and record them on the metric plan as metric units, (See Appendix A, "Metric Conversion Factors").

Drainage policies, procedures and guidelines are given subject to amendment as conditions warrant. They are not intended to be nor do they establish legal standards. Special situations may call for variations from these requirements, subject to approval from the designer's supervisor. The proper documentation of drainage decisions is vital for project records and archival purposes.

For additional design and engineering guidance, refer to the **AASHTO** Model Drainage Manual (Reference 1.1) and the following **Federal Highway Administration (FHWA)** publications:

- Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements (Ref. 1.2) (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec12.pdf)
- Hydraulic Engineering Circular No. 13: Hydraulic Design of Improved Inlets for Culverts (Ref. 1.17) (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec13.pdf)
- Hydraulic Engineering Circular No. 14: Hydraulic Design of Energy Dissipators for Culverts and Channels (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf)
- Hydraulic Engineering Circular No. 15: Design of Roadside Channels with Flexible Linings (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf)
- Hydraulic Engineering Circular No. 19: Hydrology (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec19.pdf)
- Hydraulic Engineering Circular No. 22: Urban Drainage Design Manual, (Ref. 1.26) (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf)
- Hydraulic Design Series 3: Design Charts for Open-Channel Flow (Ref. 1.13) (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds3.pdf)
- Hydraulic Design Series 4: Design of Roadside Drainage Channels (Ref.1.11) (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds4.pdf)
- Hydraulic Design Series 5: Hydraulic Design of Highway Culverts (Ref. 1.14) (https://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7&id=13)
- Federal-Aid Policy Guide. (Ref. 1.3) (http://www.fhwa.dot.gov/legsregs/directives/fapgtoc.htm)

1. DRAINAGE OBJECTIVES

Highway drainage facilities provide for carrying water across the right-of-way and for removal of stormwater from the roadway. These facilities include bridges, culverts, channels, ditches, curbs, gutters, various types of drains, inlets, manholes, storm sewers, or other hydraulic and erosion control devices. Erosion control is an integral part of drainage design, (See Chapter Two: Erosion and Sedimentation Control).

The objectives of highway drainage design are:

- To maintain the existing drainage patterns.
- To pass the design peak flow under the roadway.
- To prevent the accumulation of water on and adjacent to the highway.
- To minimize stormwater interference with vehicular traffic.
- To minimize damage to the surrounding environment.

The design of drainage systems should minimize the risk of traffic interruption by flooding, consistent with the roadway classification.

2. LEGAL, REGULATORY AND ENVIRONMENTAL ISSUES

2.A Drainage Law

Water, in all its forms, is the primary concern of drainage. Drainage, or the gradual draw off of water, is the subject of a sizable segment of legal code. There are a variety of types of water with very specific definitions dependent upon their origin, their location and their destination. See Sections 2.A.4 through 2.A.6, the Glossary, and Chapter 5 of **AASHTO's** Highway Drainage Guidelines (Reference 1.4) for additional information.

2.A.1 Drainage Easement

The methods of treatment, control, and disposal of the various types of water differ. Easements for drainage may give rights to impound, divert, discharge, concentrate, extend pipelines, deposit silt, erode, scour, or undertake other activities necessary to the consequent development of a highway.

By common law, an upstream owner has an easement over lands of a downstream owner for diffused surface waters (commonly called "surface waters") to flow or escape from his/her land by natural routes. The upstream owner may not, without liability, change the point of discharge of surface waters, nor concentrate them in ditches, nor divert in that direction waters which would have escaped in another direction, nor discharge them at higher velocity, nor add to their pollution. He/she may, however, increase the quantity by roofing or paving over previous soils, or by leveling his/her land so as to eliminate puddles and ponds. Likewise, he/she may decrease the quantity by retention for his/her own use, but in so doing risks loss of his/her easement. The downstream owner may not, without liability, obstruct natural flow of surface waters on to his/her land, either by excluding it or causing backwater on his/her neighbor. The highway owner may be either an upstream or downstream owner, with substantially the same rights and liabilities in law.

2.A.2 Disposal of Surface Water

The problem of disposal of surface waters is one of the most difficult tasks in highway design. The usual solution is to collect the surface waters in small concentrations and convey these concentrations to a natural watercourse. In developed areas, the waters may be discharged in a storm sewer. There may be no objection to discharge of small concentrations in rural areas. Small spreading areas and recharge wells may be used to sink the water into the ground. In suburban areas, future changes in land use must be anticipated.

The foregoing applies to state and county highways, but procedures are usually different in municipalities. If, by ordinance, a city establishes grade along the property line of a street, it can improve the street to that grade without liability to owners of abutting property lying above or below the established grade. Also, the pattern of a street system is more adaptable to disposal of collected surface waters than the pattern of rural highways.

2.A.3 Natural Watercourses

Where natural watercourses are unquestioned in fact and in permanence, there is little difficulty in applications of law. Highways cross channels on bridges or culverts. There is usually some constriction of the width of the channel, possibly causing backwater upstream and acceleration of the flow downstream. These changes must be so small as to not damage adjoining property, or those owners must be compensated.

Highways often discharge surface waters into the most convenient watercourse. This right is unquestioned if those waters are naturally tributary to the watercourse and unchallenged if the watercourse has adequate capacity. *However, if all or part of the surface waters have been diverted from another watershed to a small watercourse, any downstream owner may complain and recover for ensuing damage.* It is **NDOT** policy that we *do not* change natural watercourses.

2.A.4 Flood Waters

Flood waters are stream waters that have escaped from the natural watercourse. Surface waters do not become floodwaters, no matter how fast, deep, or where they flow, unless en route they have entered a natural watercourse and have escaped. They have not escaped if they run in an overflow channel or in an outer slough of a threaded channel.

In common law, flood waters are a "common enemy" of the people, lands, and property attacked or threatened by them. Everyone, including owners of highways, can act in any reasonable way to protect themselves and their property from the common enemy.

By definition, floodwaters are not necessarily infrequent occurrences; where frequent, the highway may be designed to obstruct, divert, or pass the flow to abutting or other property owners, without liability.

2.A.5 Ground Waters

Interference by a highway with flow or stage of ground waters may be cause for complaint with claim for relief. Excavations could possibly drain away ground waters that have been used for irrigation supply. Embankments may compress underlying water-bearing soils and restrict the percolation of ground water from one side to the other.

Federal, state, and local laws restrict operations that may pollute or contaminate ground waters. Plans for disposal of surface waters by spreading ponds or recharge wells must comply with such restrictions.

2.A.6 Waste, Artificial and Unnatural Waters

As a general rule, waters other than flood waters, which have been forced out of natural pools or paths by the works of man, become a responsibility of some person or agency, which must divert, store, use and waste such waters without damage to others. Highways may affect and be affected by irrigation and drainage canals, and by the use, conveyance and ponding of water for power, mining, navigation, flood control, industry, sanitation and recreation.

2.B Regulatory and Environmental Issues

Regulatory and environmental issues must be addressed at the local (city and county), state and federal levels. Refer to the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 4, (http://www.roads.nebraska.gov/business-center/design-consultant/rd-manuals/), (Reference 1.24) for additional information.

3. COORDINATION WITH OTHER ENTITIES AND PUBLIC INVOLVEMENT

Coordination with federal, state, and local (city and county) governmental agencies is often necessary because of legal implications or special local drainage ordinances. The **Army Corps of Engineers (COE), Federal Emergency Management Administration (FEMA), Environmental Protection Agency (EPA), Department of Environmental Quality (DEQ),** and the **Natural Resource Districts (NRD)** have provisions that must be followed. The **Department of Natural Resources (DNR)** deals with urban floodplain issues, designers may consult with **DNR** regarding drainage computations for urban areas. **DEQ** also has authority and primary responsibility in erosion control. See the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 4, (Reference 1.24) for further discussions.

Where highway fills are to be used as dams to permanently impound water more than 50-acre ft. in volume or 25 ft. deep, the hydrologic, hydraulic, and structural design of the fill and appurtenant spillways shall have the approval of the state or federal agency responsible for the safety of dams or like structures within the state, prior to authorization by **FHWA** to advertise for bids for construction.

4. DESIGN CONSIDERATIONS

4.A Economic Considerations

Economic factors are important drainage design considerations. Benefits provided by the drainage facility versus cost to construct and maintain the drainage structure should be evaluated. The following economic factors are considered:

- Environment impacts.
- Cost of construction and right-of-way.
- Delays, interruptions and inconvenience to traffic due to floods or roadway failures.
- Injuries and hazards to persons and vehicles caused by floods or roadway failures.
- Damage and resultant repair costs to state highway drainage facilities.
- Damage to private and public property and resultant liability.

- Pavement or subgrade failure and resultant repair costs.
- Damage to public utilities and resultant liability.
- Contamination of public water supplies and sewage systems and attendant consequences to public health.

4.B Safety Considerations

Safety is a major concern throughout roadway design, including drainage design. Water on the pavement may lead to loss of steering control, loss of visibility due to splashing and/or spray, and may disguise a serious pavement fault. Drainage structures may be located in the lateral obstacle clearance zone, with proper safety treatments.

The following guidelines apply to the location and treatment of drainage structures:

- All portions of the roadway surface, especially those at intersections, entrance and exit ramps, sharp curves, bridge approaches and sag vertical curves, should be designed so that ponding does not occur and that sheet or concentrated flow over the pavement is minimized.
- Median drainage inlets should be flush with the median grade.
- Dikes should be placed with 10:1 or flatter slopes within the lateral obstacle clear distance in the direction of oncoming traffic.
- Headwalls shall not be located within the lateral obstacle clearance zone unless protected by guardrail.
- Where possible, cross drainage culverts greater than 36 in. diameter shall be extended to the lateral obstacle clearance distance.
- Protection devices, (e.g., traversable end sections for flared end sections), shall be provided at culvert termini within the lateral obstacle clearance zone for pipe larger than 36 in. in diameter. These devices may also be used on smaller culverts presenting unusual hazard.
- Guardrail treatment shall be provided for box culverts and for culvert pipes greater than 36 in. in diameter that terminate within the lateral obstacle clearance zone without protection devices, unless the guardrail installation itself has been analyzed to be a greater hazard than the unprotected structure.
- When culverts are extended, slopes also should be extended to provide cover for the culvert end. Earthwork computations should reflect the slope change.
- Grates or modified debris barriers should be provided at culvert openings where access by children or animals could create hazardous situations (primarily in urban areas). Application of such protective devices should be consistent with the policy of the urban area in which the device is located.

5. PRELIMINARY DRAINAGE DESIGN

The proper design of a storm drainage system involves the accumulation of certain basic data:

- Familiarity with the project site.
- A basic understanding of hydrologic and hydraulic principles.
- The drainage policy associated with the project under design.

This section outlines and discusses various information and activities that need to be assembled and completed during the preliminary design phase of the project.

5.A Background Information

As-built project plans and construction books should be reviewed for applicable information and data (e.g., culvert location, size and type; Drainage Area; Design Q; Headwater). County or city offices may have records, which could yield valuable information of past flooding events or other drainage problems. Drainage information may also be obtained from contacts with the general public, interviews with local residents concerning past flooding events may be helpful. The **District Engineer** should be consulted for information about previous flooding experiences.

5.B Mapping

The designer should obtain the existing topographic mapping of the project site, including maps from the **U.S. Geologic Survey (USGS)**, the **Natural Resources Conservation Service (NRCS)**, and other topographic maps available from state and local agencies. Street maps and land use maps should also be obtained and reviewed, as necessary.

5.C Floodplain Information

The designer should obtain information identifying existing flood boundaries and floodways at the project site. **FEMA** maps should be obtained. Records of historical flood data and elevations should be gathered.

EXHIBIT 1.1 diagrams the design process to use when a project is in a floodplain. If the project is in a **FEMA** regulated floodplain, certification must be obtained and sent to the **Environmental Unit** in the **Project Development Division**. This will usually be accomplished by either the **Roadway Design Hydraulic Engineer** or the **Bridge Hydraulic Engineer**.

The Nebraska Administrative Code Title 455, Chapter 1, (Reference 1.16), (http://www.sos.ne.gov/rules-and-regs/regsearch/Rules/Natural_Resources_Dept_of/Title-455/Chapter-1.pdf), provides rules and regulations related to construction in floodplains. When adequate flood elevation and other pertinent information are available, determine if the proposed obstruction is in the flood fringe or in a floodway (See EXHIBIT 1.2). Normally, the floodway shall be determined using the method requiring equal loss of conveyance on opposite sides of the stream. Other methods may be used if they are more appropriate for specific situations.

No new construction, substantial improvements or other obstruction (including fill) shall be permitted in the floodplain of a base (100-year) flood, unless it is demonstrated that the cumulative effect of the proposed new construction, when combined with all other existing and anticipated new construction or substantial improvements, will not increase the water surface elevation of the base flood more than one foot at any location.

A watercourse or drainway in the floodplain shall not be altered or relocated in any way which in the event of a base flood or more frequent flood will alter the flood carrying characteristics of the watercourse or drainway to the detriment of upstream, downstream or adjacent locations.

Furthermore, no new construction, substantial improvements or other obstruction (including fill) shall be permitted within the floodway unless it has been demonstrated through hydrologic and hydraulic analyses that the proposed new construction would not result in any increase in water surface elevations along the floodway profile during the occurrence of the base flood. For additional information see the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 4, (Reference 1.24).

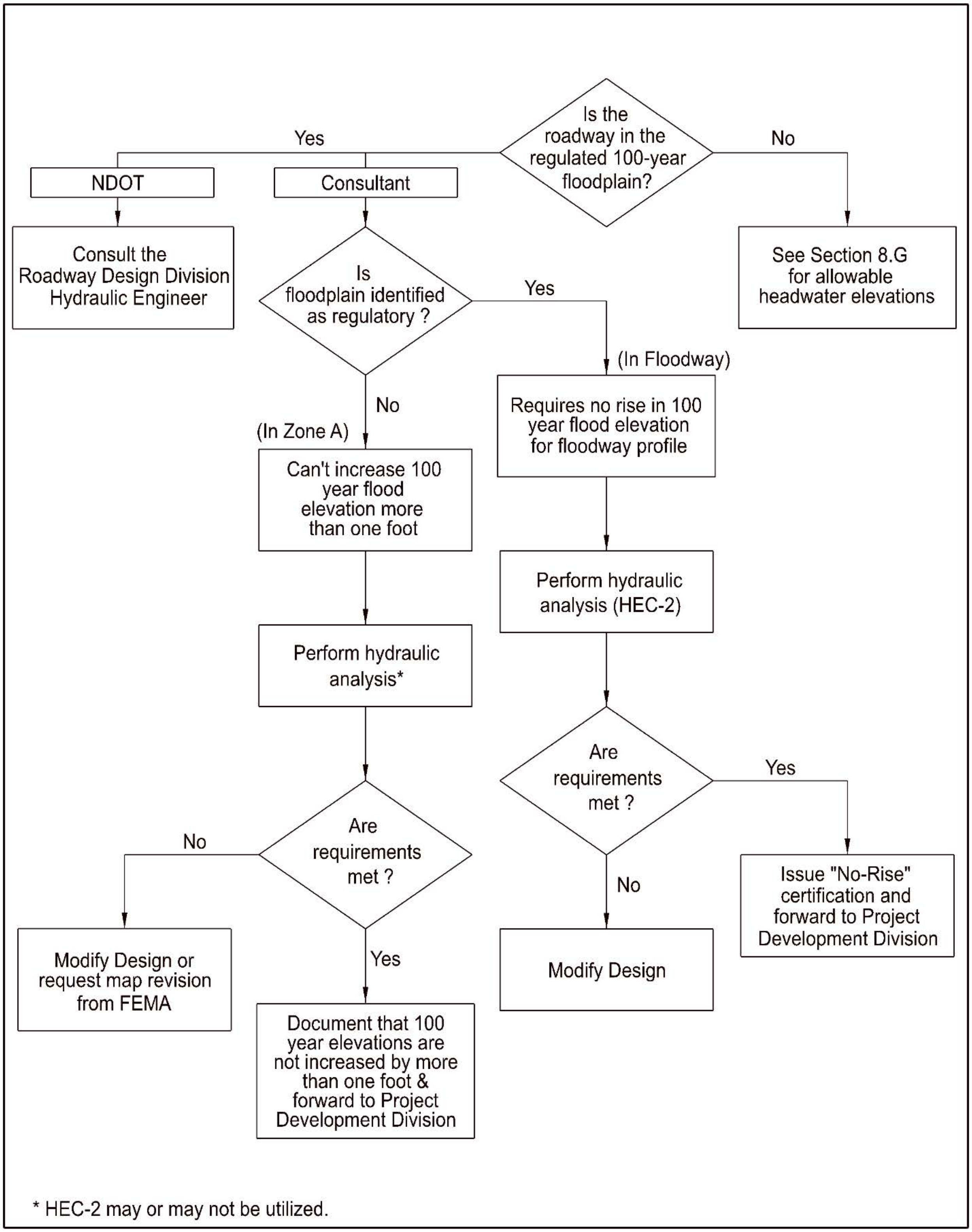


Exhibit 1.1 Floodplain Flow Chart

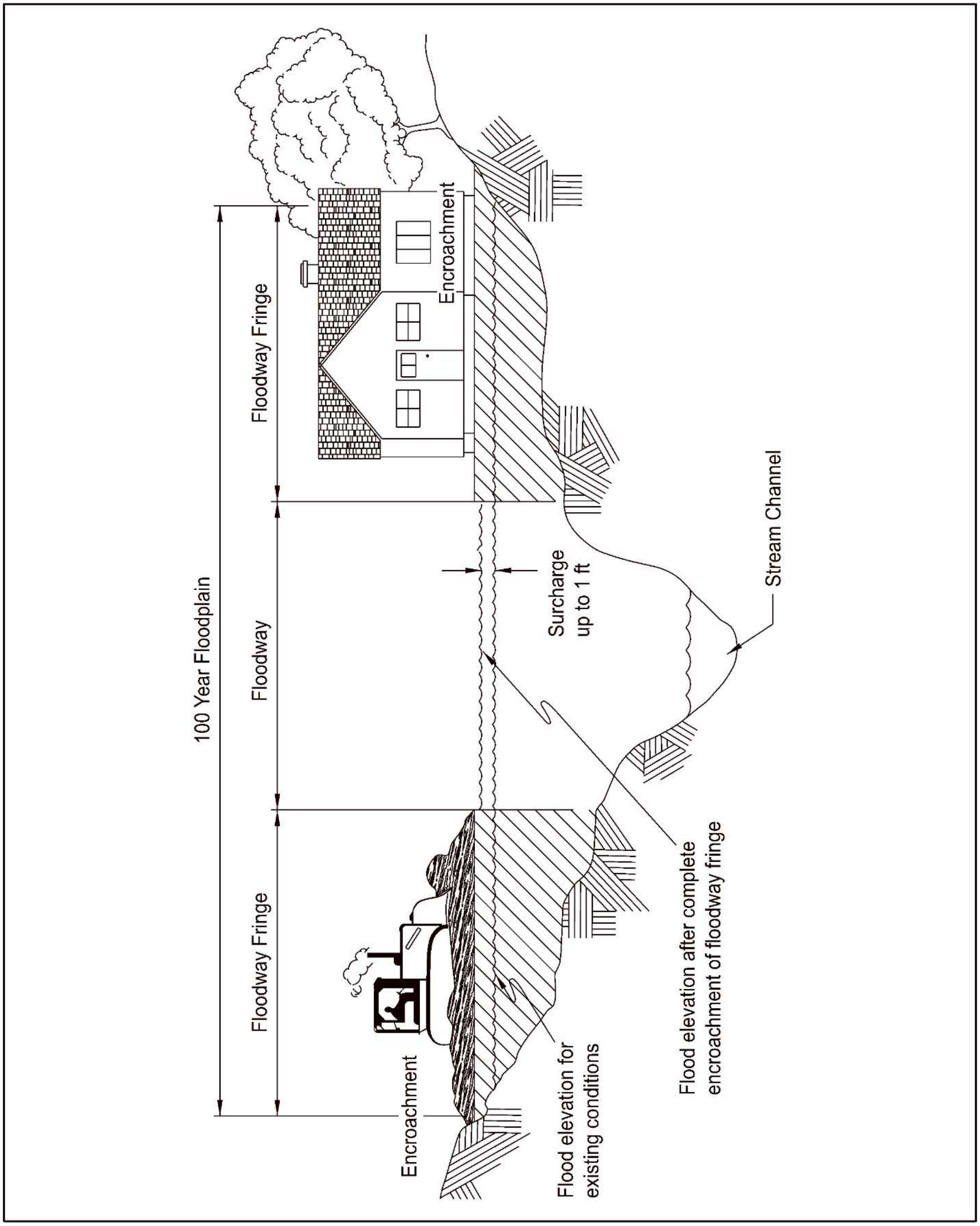


Exhibit 1.2 100-Year Floodplain Schematic
(Source: Reference 1.16)

5.D Utilities

The preliminary survey will include the location of existing utilities (See the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 6, Reference 1.24). Utility conflicts for both existing and proposed installations must be considered during drainage design.

5.E Watershed Characteristics

During preliminary drainage design, the designer needs to obtain an overall understanding of the characteristics of the watershed where the project is located. Watershed characteristics can be primarily determined from studies of existing topographic maps, aerial photos, and soil survey maps. The size, slope, and configuration of the watershed, geometry of the stream network, and location of existing ponds and lakes can be determined from the above listed documents.

5.F Land Use

The designer should be familiar with land use patterns and the nature of physical development in the watershed. Land use maps and aerial photos, indicating existing and future development in the watershed, should be reviewed if available. Changing land use patterns may impact drainage design. Drainage facilities should be designed to accommodate the land use at the time of the highway construction.

5.G Existing Drainage Facilities

Construction drawings indicating the location of existing drainage facilities should be obtained and reviewed. The hydraulic capacity and structural conditions of the facilities should be assessed. The designer should consult the survey for any inadequacies or problems with the drainage facilities.

Ordinarily, 3R projects will not require a hydraulic analysis of culverts since these projects normally involve only the driving surface. Some 3R projects may require extending the culvert beyond the lateral obstacle clearance distance. In these instances the existing culvert may be extended and hydraulic analysis of the culvert will not be necessary.

5.H Preliminary Layout

Preliminary layouts or schematics, featuring the basic components of the intended design, are useful to the designer. Such sketches should indicate watershed areas, a street and driveway layout with respect to the project roadway when needed, underground utility locations and elevations, logical inlet and manhole locations, preliminary lateral and trunkline layouts, and a clear definition of the outfall location and characteristics. It may be useful for the designer to place existing utilities on the cross section drawings. With this layout, the designer can proceed with the detailed process of culvert and storm drainage design calculations, adjustments, and refinements.

5.I Preliminary Design Checklist

A checklist of information and activities should be assembled and completed during preliminary drainage design. See the Roadway Design Manual, Chapter Two: Roadway Design Process, Section 5.E, (Reference 1.24), and **EXHIBIT B** of the Design Process Outline, (Reference 1.25), (http://www.roads.nebraska.gov/media/6761/design-process-outline.pdf), for additional information.

5.J Field Visit

The roadway designer should schedule a field visit for the purpose of investigating the existing drainage conditions and the proposed drainage design. The designer should obtain an understanding of the existing drainage patterns.

6. HYDROLOGY

Hydrology is the study of the movement and distribution of water. Runoff is the drainage that leaves an area as surface flow or pipeline flow. Hydrologic analysis is necessary to determine the peak runoff rate and volume of runoff that a drainage structure will be required to convey or control. Methods for computing peak rates of runoff and criteria for determining design storm frequencies are included in this section.

6.A Factors Affecting Peak Runoff

The designer should be familiar with the many factors or characteristics that affect peak runoff rates. This section provides a listing of these factors.

Basin Characteristics:

- Area.
- Shape.
- Slope (land and stream slope).
- Land use and vegetative cover.
- Soil type and antecedent (existing) soil moisture.
- Storage (interception and depression storage).
- Orientation of basin.

Channel Characteristics:

- Channel configuration.
- Cross-section.
- Stream frequency (number of streams in a drainage area).
- Stream morphology (change in channel shape).

Site Characteristics:

- Deposition of sediment/streambed erosion.
- Debris accumulation.
- Seasonal changes in vegetation.

Storm Characteristics:

- Type of precipitation.
- Frequency of precipitation.
- Rainfall intensity.
- Rainfall duration.
- Distribution of rainfall within basin.
- Direction of storm movement.

6.B Probability and Frequency

Since the hydrology of drainage structures is concerned with future events, designers use probability or frequency concepts with which a specified rate or volume of flow will be equaled or exceeded.

Sometimes storm recurrence is expressed in terms of probability as a percentage rather than in terms of frequency. As an example, a storm having a 50-year frequency can be expressed as a 2% storm or a storm with the occurrence probability of 0.02. This means a 50-year storm has a 2% chance of being equaled or exceeded in a given year. By expressing recurrence intervals in terms of a percentage, it is possible to avoid misinterpretation associated with using a frequency in terms of years. *When dealing with the public, it is usually better to discuss storm recurrence in terms of probability as a percentage since this is generally easier to understand and comprehend.*

6.C Design Storm Frequencies

Design storm frequencies by drainage facility type and design location are given in EXHIBIT 1.3.

6.C.1 Culvert Design Storm

The design storm frequency was chosen to limit the potential and frequency of overtopping occurrences for the highway given the level of service, risk to the public, and other related damages from overtopping. The design storm frequency provided in EXHIBIT 1.3 might need to be adjusted on a case-by case basis when special conditions merit such a change. An example of a special condition might be where the culvert underlies the only practical route to and from a critical area, requiring that the road be open even during low probability (100-year) events.

The Department of Natural Resources (**DNR**) may designate some drainage ways as special flood prone areas. The use of the 100-year storm frequency is required for design of all drainage structures in these specially designated areas, (See Section 5.C for additional information).

Design Location	Interstate	Expressways & Over 7500 ADT	2000 – 7499 ADT	1999 ADT & Under
Culverts	50 year	50 year	50 year	25 year
Storm Sewers*	50 year	50 year	10 year	10 year
Storm Sewer on Depressed Roadways	50 year	50 year	50 year	25 year
Roadway Gutters	50 year	50 year	10 year	10 year
Median Pipe	50 year	50 year	10 year	10 year
Ditch Grade Control Drop	50 year	50 year	25 year	25 year
Intercepting Dike / Backslope Drop Pipe	25 year	25 year	25 year	25 year
Temporary Facilities** (Duration ≤ Two Years)	2 year	2 year	2 year	2 year

* The 10 year design storm for storm sewers does not include cross-drainage culverts.
** These frequencies are used for facilities to remain in place for less than two years. If a facility will be in use for more than two years, other appropriate storm frequencies should be considered.

Exhibit 1.3 Design Storm Frequencies

6.C.2 Storm Sewer Design Storm

Urban storm sewers are generally designed for the 10-year storm. However, adjacent land uses and structures should be considered in low lying areas to see if the 100-year storm will cause water to back up into structures or critical areas. When designing new storm sewer systems the Designer must determine where surface water not collected by the system will be carried. This uncollected surface flow cannot be allowed to impact / directed toward structures or other critical areas.

Occasionally a 10-year frequency for storm sewers in cities or villages located in extremely flat terrain, such as the Platte River Valley, may be virtually impossible or impractical to obtain. In such instances, special consideration may be in order for frequencies as low as two years.

When connecting a proposed storm sewer to an existing municipal system, the designer should make sure the proposed storm sewer does not overload the existing system. If the desired 10-year storm design puts the existing system over capacity, the following alternatives should be investigated on a project-by-project basis:

- **NDOT** will notify the municipality that it should upgrade its municipal drainage facilities. **NDOT** will request, for safety and liability reasons, that the municipality commit to one of the following plans for upgrading their municipal drainage facilities:
  a) The municipality provides the state with reasonable written assurances of a present plan for a future upgrade of its municipal facilities. The municipality shall provide the state with the details of its proposed improvements that will convey the design event determined by the state.
  b) The municipality requests that the project include an upgrade of its municipal drainage facilities to be paid for solely by the municipality, and the municipality shall enter into an agreement with the state concerning this upgrade of its facilities prior to **NDOT** beginning the final design of the project.
- Design for a 10-year frequency, connect to the existing municipal system, and assume the municipality will upgrade their system in the future (this alternative must not increase flood liability downstream).
- Consider detention using 10-year frequency design.

6.D Peak Runoff Design Methods

Drainage design begins with an estimate of the quantity of water that is anticipated to reach drainage inlets. Several methods have been developed for estimating peak runoff quantities. This section presents the two methods recommended for computing peak runoff, the rational method and regression equations.

Hydrology is random and follows a probabilistic behavior. Experience and judgment are important in determining peak runoff estimates since the hydrologic analysis is only an approximation of the complex precipitation-runoff relationship. Peak runoff estimates computed using different methods may vary, sometimes considerably, because present methods use differing assumptions and parameters. The designer may use methods other than the **NDOT** recommended methods for computing peak runoff to compare results, (for other methods consult the **Roadway Design Hydraulic Engineer**). Local (county or city) regulations may require that a specific method be used.

6.D.1 Rational Method

The rational method is the most commonly used method to estimate the peak runoff of a drainage basin. The peak runoff is computed using the assumptions that:

- The peak flow occurs only during the peak rainfall event.
- That it does not occur until the entire basin is contributing to the flow.
- The rainfall rate is uniform over the entire basin.
- The rainfall rate is uniform over the entire time it takes for the entire basin to contribute.

Due to these assumptions the rational method is normally used for computing runoff from drainage basins less than 640 acres in both rural and urban areas.

The rational method is based on the following formula:

$$Q = CiA \qquad \text{Eq. 1.1}$$

where:
Q = Discharge, cfs;
C = Coefficient of runoff;
i = Intensity of rainfall, in./hour;
A = Drainage area, acres.

Section 14.A demonstrates the use of the rational method for computing peak runoff.

6.D.1.a Coefficient of Runoff (C)

The Coefficient of Runoff (C) value in the rational formula is the proportion of the total rainfall, expressed as a decimal, which runs along the ground as surface runoff. The C values given in EXHIBITS 1.4 AND 1.5 were derived from the following variable factors:

- Land use.
- Surface and/or soil type.
- Vegetative cover.
- Degree of imperviousness.
- Existing soil moisture.
- Watershed slope.
- Surface roughness.
- Surface storage.
- Rainfall intensity and duration.

C values for various surface types are given in EXHIBITS 1.4 AND 1.5. The range of values in EXHIBITS 1.4 AND 1.5 allows for variation in land slope and differences in permeability for the same type of cover. C values are also dependent upon the return period. For flat slopes and permeable soil, use the lower values.

Where the drainage area is comprised of several different surface types, a weighted runoff coefficient is used, based on the area of each type of surface present. For example, a given 10-acre developed area has the following characteristics for return period of 10 years:

Area (A) (acres)	Type of Surface	Slope	C (EXHIBIT 1.4)	C x A
1	concrete & roof surface	1-2%	0.83	0.83
4	parks & lawn	0-2%	0.37	1.48
5	well-established grass	> 7%	0.40	2
Total = 10			Total =	4.31

The weighted runoff coefficient (C) equals (Total C x A) / (Total A) = 4.31/10 or 0.43.

Character of Surface	Return Period (Years)						
	2	5	10	25	50	100	500
Asphalt	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass Areas (lawns, parks, etc.)							
Poor Condition (grass cover less than 50% of the area)							
Flat, 0-2% *	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2-7% *	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7% *	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair Condition (grass cover on 50% to 75% of the area)							
Flat, 0-2% *	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7% *	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7% *	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good Condition (grass cover more than 75% of the area)							
Flat, 0-2% *	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2-7% *	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7% *	0.34	0.37	0.40	0.44	0.47	0.51	0.58

* Slopes refer to watershed slope not channel slope.

Exhibit 1.4 Runoff Coefficients for Developed Areas for use in the Rational Method (Source: NDOT Research)

Character of Surface	Return Period (Years)						
	2	5	10	25	50	100	500
Cultivated Land							
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/Range							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0-2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Exhibit 1.5 Runoff Coefficients for Undeveloped Areas for use in the Rational Method (Source: NDOT Research)

6.D.1.b Rainfall Intensity (i)

The rainfall intensity (i) is the average rainfall rate, in in./hr., for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration (See Section 6.D.1.c for a discussion on time of concentration). Nebraska has been divided into three rainfall intensity zones, (See EXHIBIT 1.6). Rainfall intensity duration frequency (IDF) charts, (which relate time of concentration and design storm frequency to rainfall intensity), corresponding to these three zones are shown in EXHIBITS 1.7, 1.8 AND 1.9. The value of rainfall intensity (i) for a particular return period may be read off the appropriate line. These lines are based on the time of concentration; intensities for times of concentration other than those plotted can be interpolated from the graphs. For Example; if the 10-acre developed area in Section 6.D.1.a is in Zone B, has a time of concentration of 23 min., and a return period of 10 years:

Time of Concentration, T_C	10-year Design Storm Rainfall Intensity (in./hr.)
15 min.	4.90
30 min.	3.45

Ratio of Rainfall Intensities: $Ra_i = (i_{15}-i_{30}) / (Tc_{30}-Tc_{15}) = (4.9-3.45)/(30-15) = 0.097$

Interpolated Rainfall Intensity for Time of Concentration of 23 min.:

$i_{23} = i_{15} - (Tc_{23}-Tc_{15}) \times Ra_i = 4.9 - (23-15) \times 0.097 = \underline{4.13 \text{ in./hr.}}$

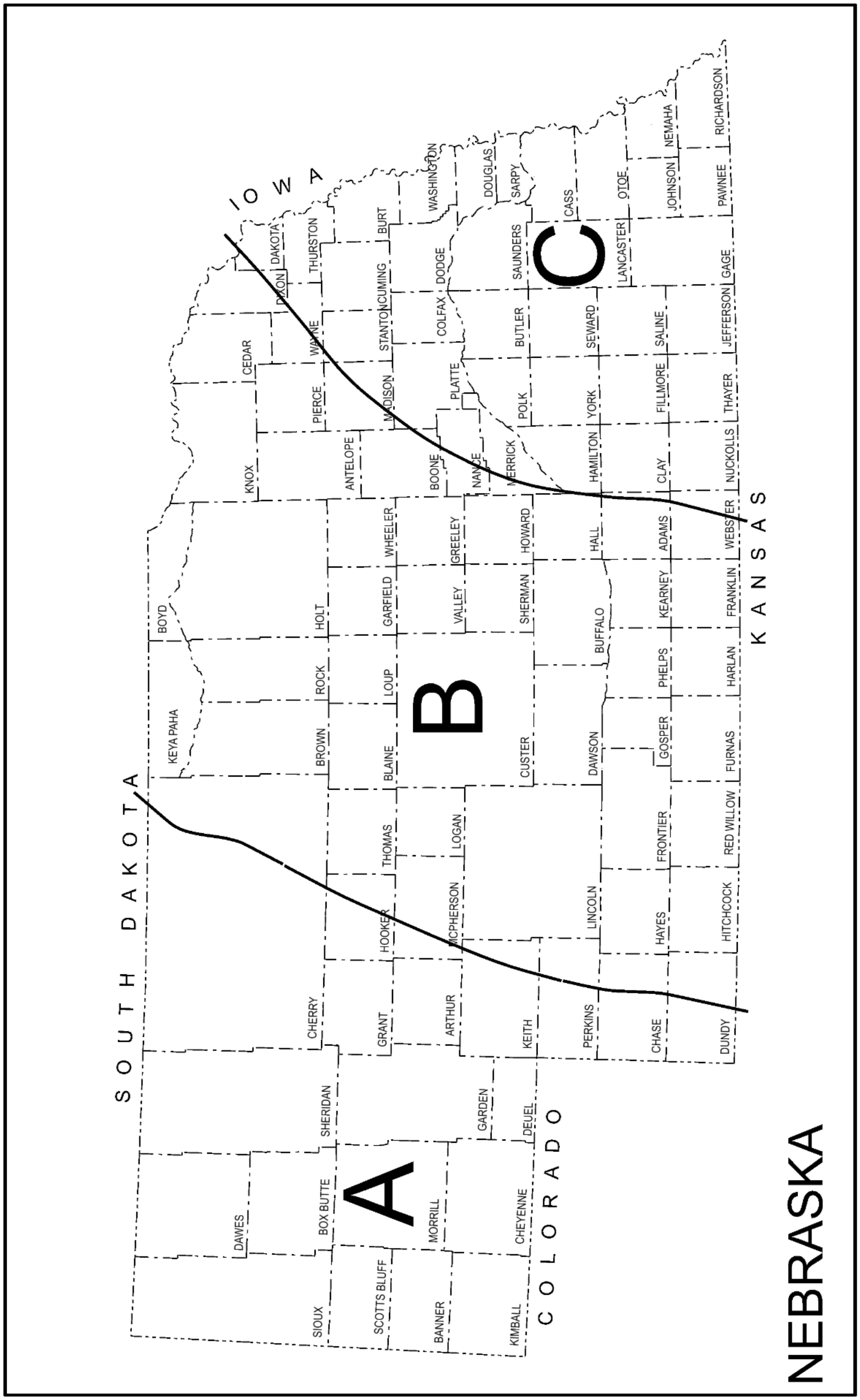
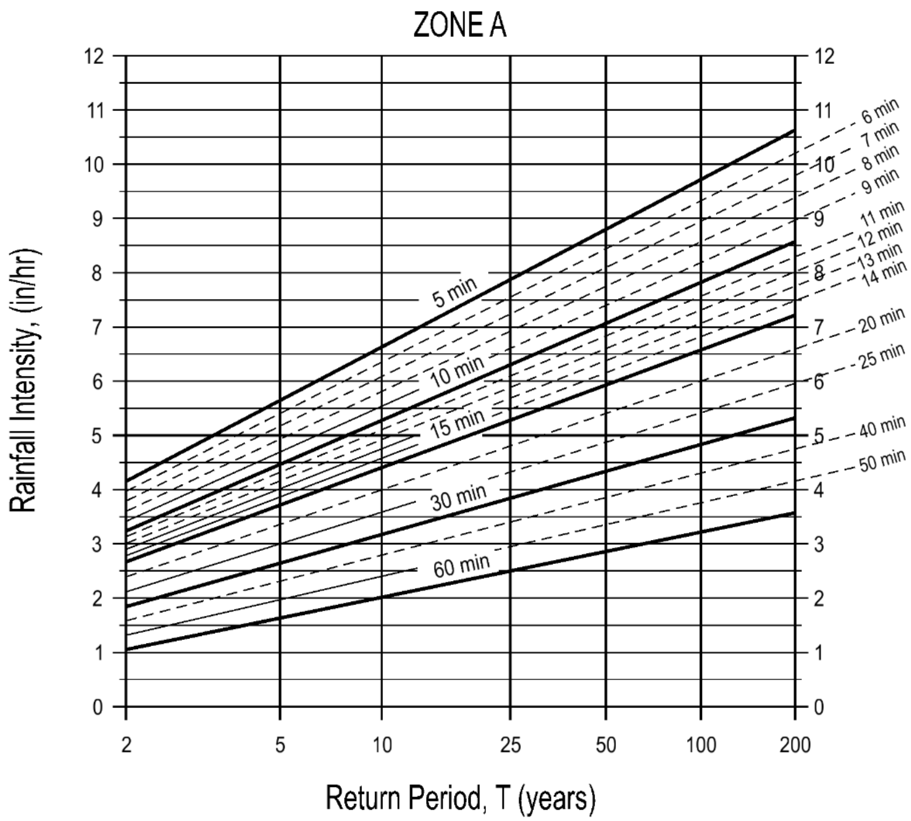


Exhibit 1.6 Nebraska Rainfall Zones for Use with Rainfall Intensities in Rational Method



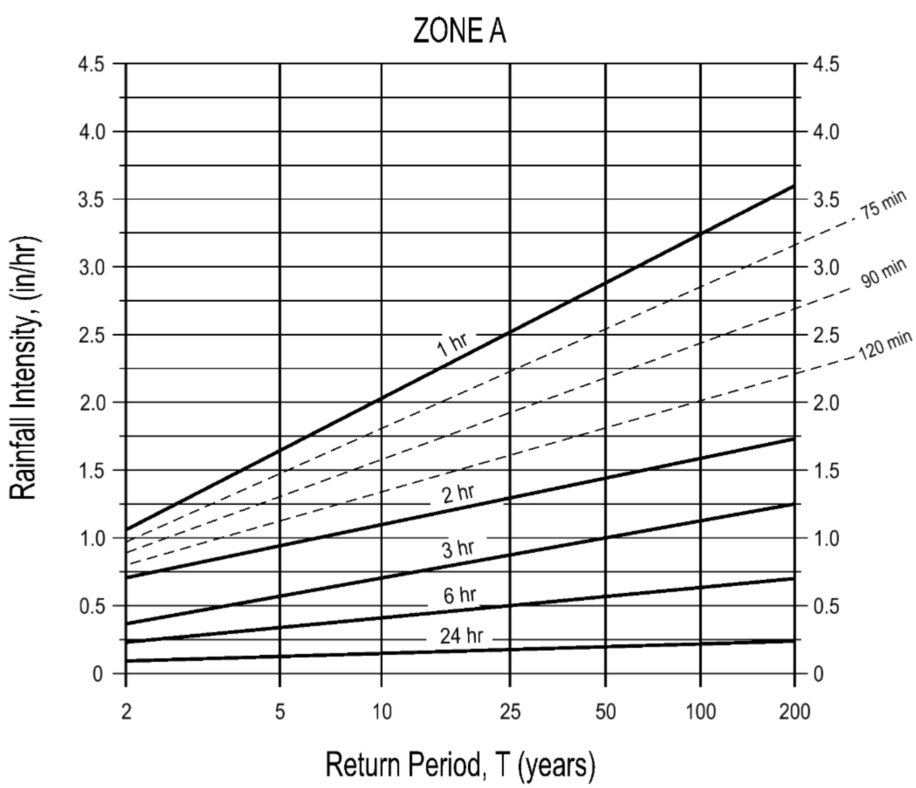
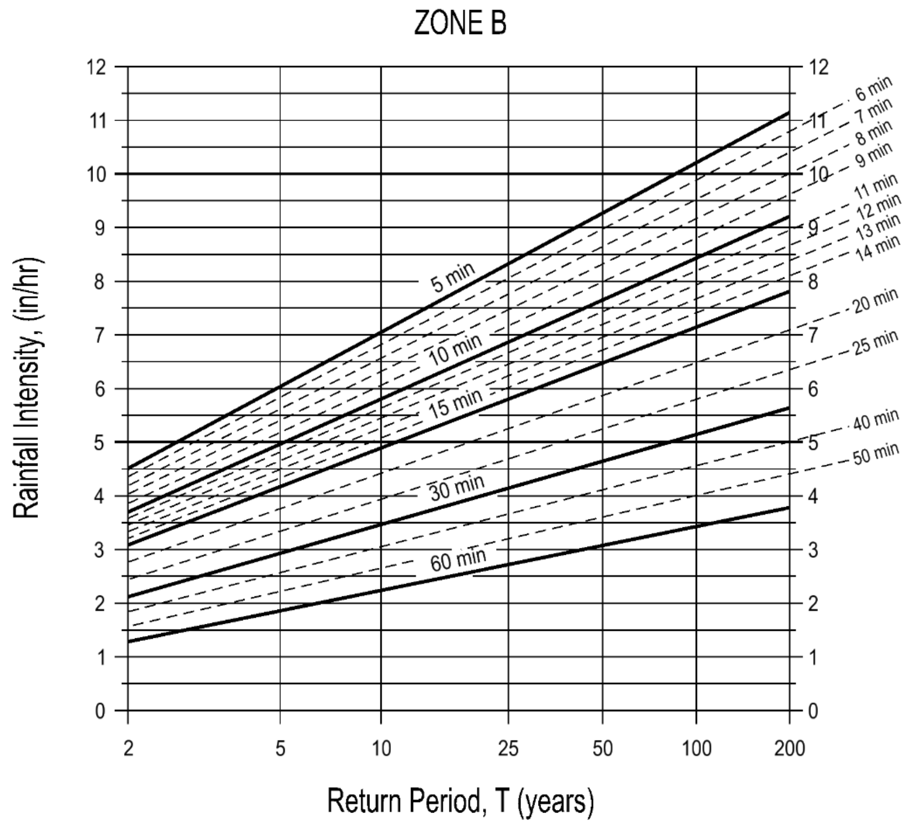


Exhibit 1.7 IDF Charts for Western Nebraska (Zone A)



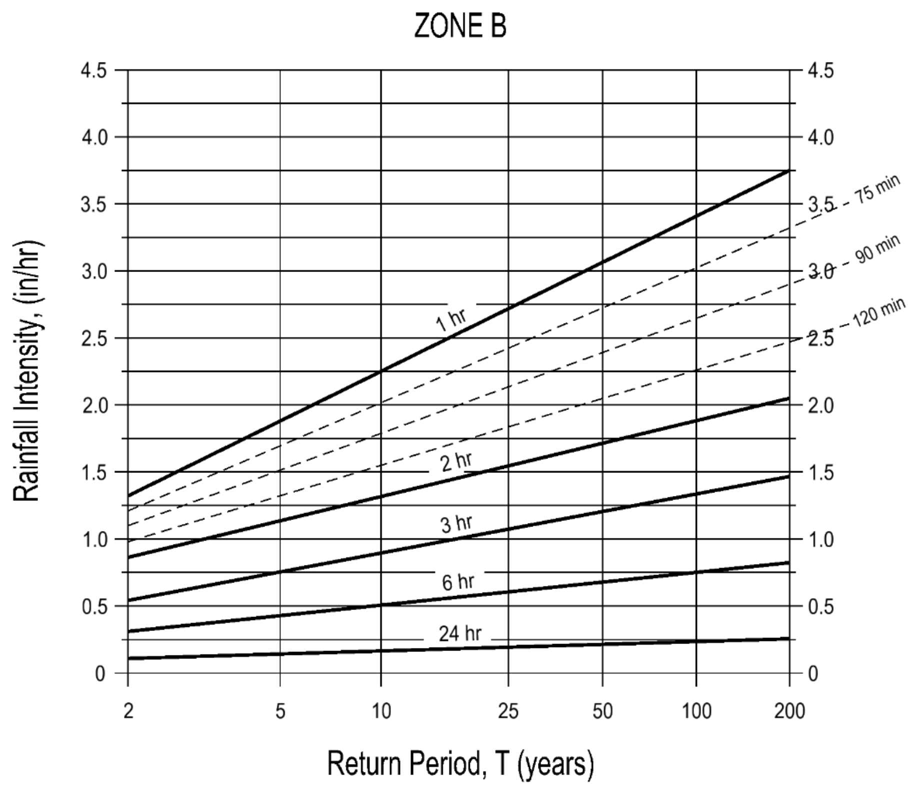


Exhibit 1.8 IDF Charts for Central Nebraska (Zone B)

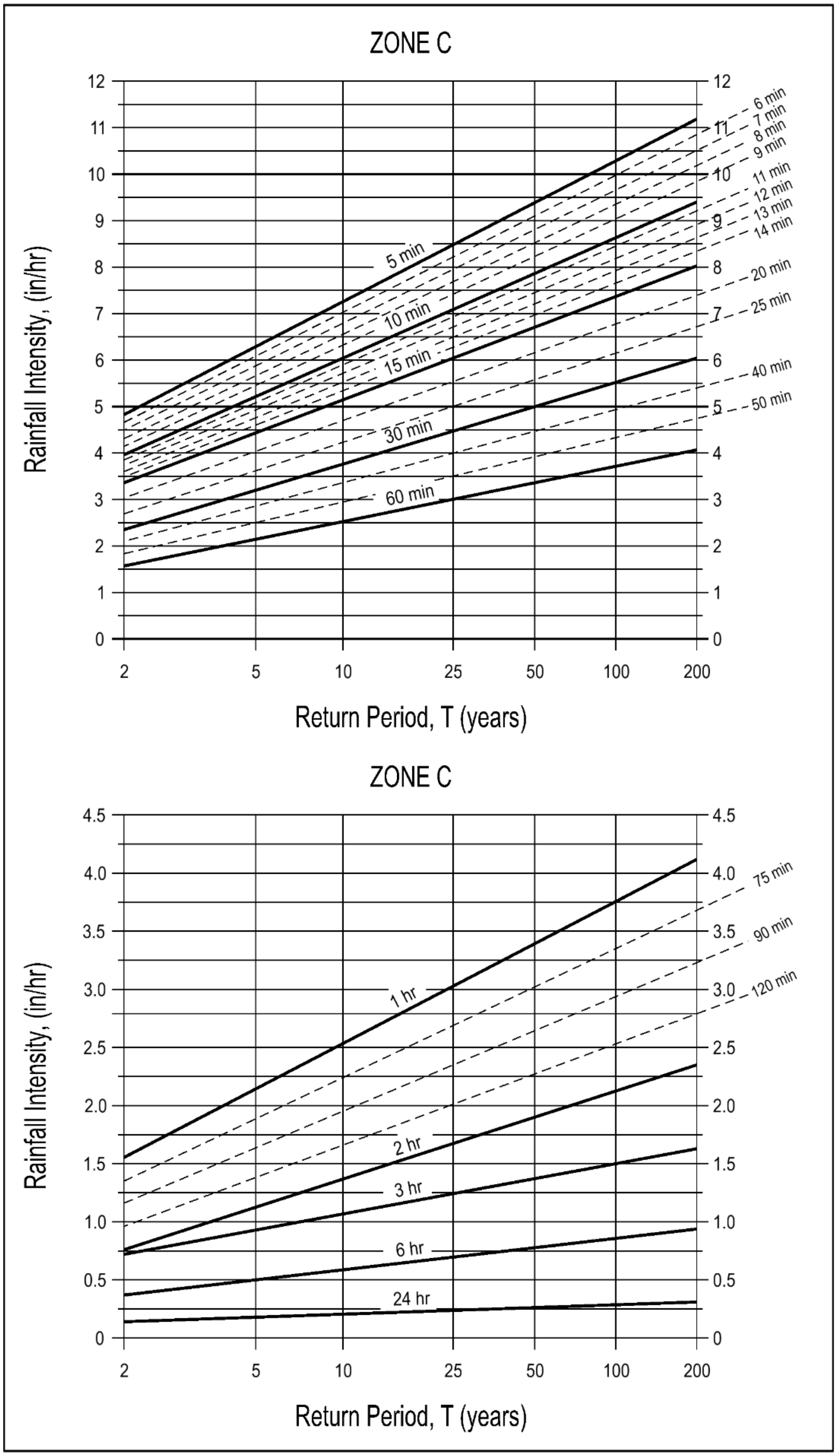


Exhibit 1.9 IDF Charts for Eastern Nebraska (Zone C)

6.D.1.c Time of Concentration (T_c)

The time of concentration (T_c) is the time required for runoff to travel from the hydraulically most remote point (in terms of travel time, not distance) on the watershed boundary to the proposed structure. The T_c value will vary with the size and shape of the drainage area, land slope, type of surface, intensity of rainfall, whether flow is overland or channelized and many other factors. The T_C value can be obtained from any number of methods, but it is recommended that the designer utilize the Kirpich equation to obtain T_C for the rational method. The Kirpich equation is:

$$T_c = 0.0078\, L^{0.77}\, S^{-0.385}\, C_F \qquad \text{Eq. 1.2}$$

where:
- L = Length of drainage path from the hydraulically most remote point of the basin to the culvert inlet, ft.;
- S = Average slope of drainage path = H / L, ft./ft.;
- H = Height of (elevation difference between) the hydraulically most remote point of the basin to the culvert inlet, ft.;
- C_F = Correction Factor for the ground cover of the drainage path (See EXHIBIT 1.10).

Ground Cover Description	Correction Factor, C_F
Natural basins with well defined channels	1.0
Mowed grass roadside channels	1.0
Overland flow, bare earth	1.0
Overland flow, grassed surface	2.0
Overland flow, concrete or asphalt surface	0.4
Concrete channel (e.g. gutter section)	0.2
Cultivated farmland	1.5

Exhibit 1.10 Correction Factor Used in Kirpich Equations

EXHIBIT 1.11, which is a nomograph for the Kirpich equation, can be used to obtain the T_C value. The use of EXHIBIT 1.11 requires the length (L) of the drainage area measured along the principal drainage line and the height (H), which is the difference in elevation between the inlet and the most remote point above the inlet (longest T_c).

Using L and H, enter the nomograph EXHIBIT 1.11 to obtain T_c. Adjust T_c using the correction factors (C_F) given in EXHIBIT 1.10 for ground cover, to obtain the final T_c value. If the time of concentration computed from EXHIBIT 1.11 is less than 5 min., the designer shall use 5 min. for the time of concentration.

The designer should keep the following points in mind regarding T_c:

- The time of concentration to any point in a drainage basin is a combination of the “inlet time” and time of flow in the channel.
- The inlet time is the time required for the water to flow over the surface of the ground to the culvert inlet. The time of flow in the culvert, T_T, may be assumed to be the length of the culvert divided by the velocity of flow:

$$T_T = \text{time of flow in culvert (sec.)} = \frac{\text{length of the culvert, ft. (m)}}{\text{velocity of flow in the culvert when full, ft./sec. (m/sec.)}}$$

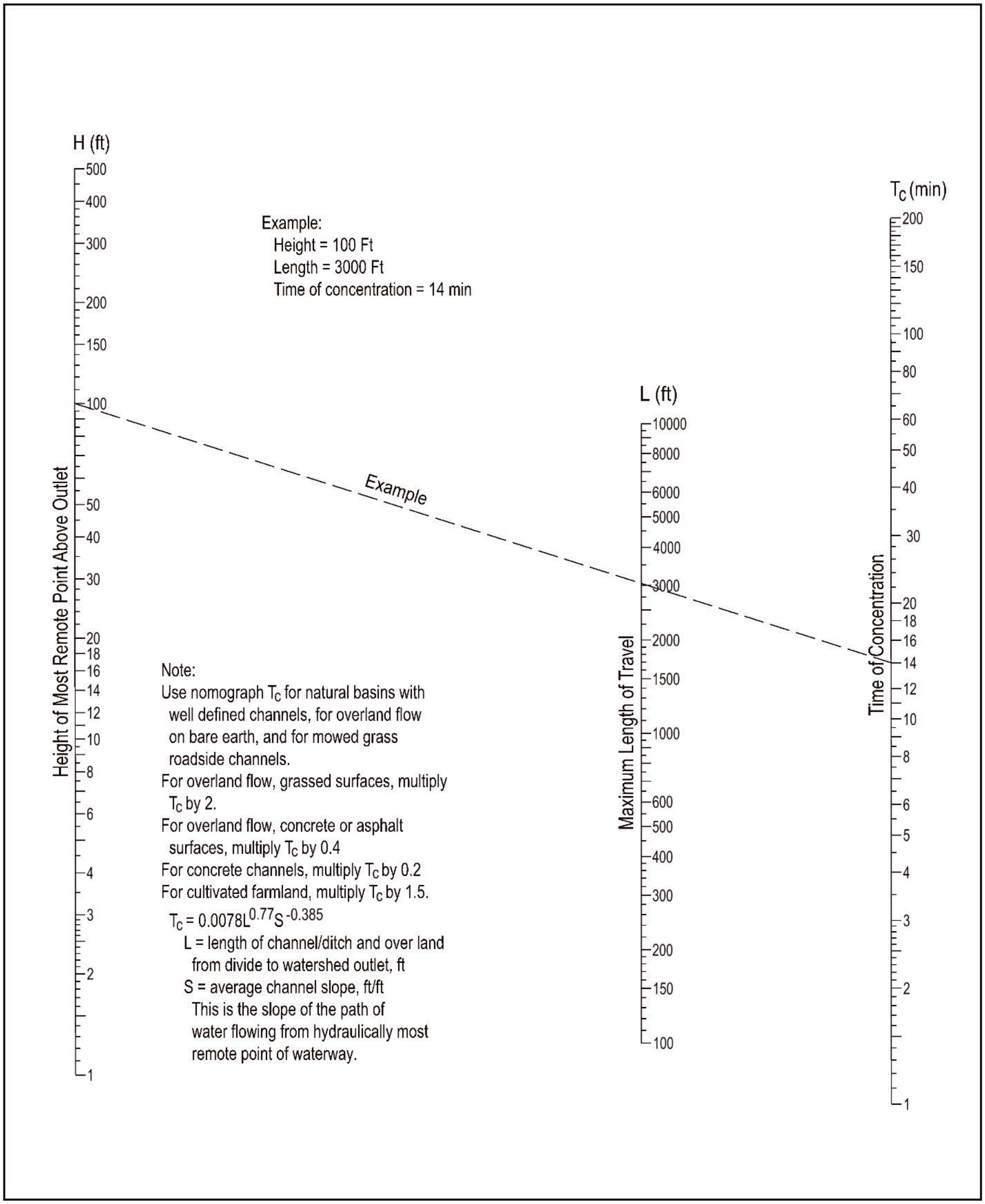


Exhibit 1.11 Time of Concentration of Small Drainage Basins (640 Ac. Or Less)
(Source: Reference 1.5)

6.D.1.d Drainage Area (A)

The drainage area, A, is the plan view area, measured in acres, found to contribute surface water runoff to the point for which the peak discharge is to be determined. The drainage area is normally determined using contour maps in conjunction with aerial and stereo photos, land surveys, and site inspections.

6.D.2 Regression Equations

The **NDOT** Regional Regression Equations have been developed to determine peak discharges for drainage areas greater than 640 acres. These regional regression equations were developed through statistical regression analyses that related various physical and climatological characteristics of watersheds in a given region to the peak-flow data provided by their corresponding gauging stations. The resulting **NDOT** Regional Regression Equations for each region for the return periods of 2, 10, 50, 100, 200 and 500 years and the physical and climatological characteristics used in the development of the regional regression equations are shown in EXHIBIT 1.13.

Using the regional regression equations, peak discharges can be estimated for any location in the State of Nebraska, including those lacking gauge data, by determining a watershed's measurable representative characteristics and using those values in the respective regional regression equation.

As part of the regression analyses, the State of Nebraska was divided into five (5) regions. These regional divisions were selected to reduce the standard of error for the regression equations results. The five (5) hydrologic regions of Nebraska are shown on EXHIBIT 1.12, and are described below.

- Region 1 is the remnant after the other four regions were separated from the entire group. In general, it is along the northern border and in the southwestern part of the state.
- Region 2 basically is made of Sandhill terrain that is not contiguous. The streams, which rise in Sandhill terrain, retain the characteristic of Sandhill streams even though their main stems flow across other types of terrain and soil and have tributaries from areas having characteristics different from Sandhills.
- Region 3 includes almost all of the eastern part of the state. Both the Elkhorn River, (which drains part of Region 2), and the Platte River, (which is a controlled stream), cross Region 3 but are not part of it.
- Region 4 is the loess-hill area that drains into the Loup River and the north side of the central part of the Platte River. This region is traversed by the main-stem streams, which are in Region 2 and must be treated with Region 2 relations. They are the South Loup, Middle Loup, North Loup, and Loup Rivers; also Cedar River and Beaver Creek.
- Region 5 is the Blue River basin.

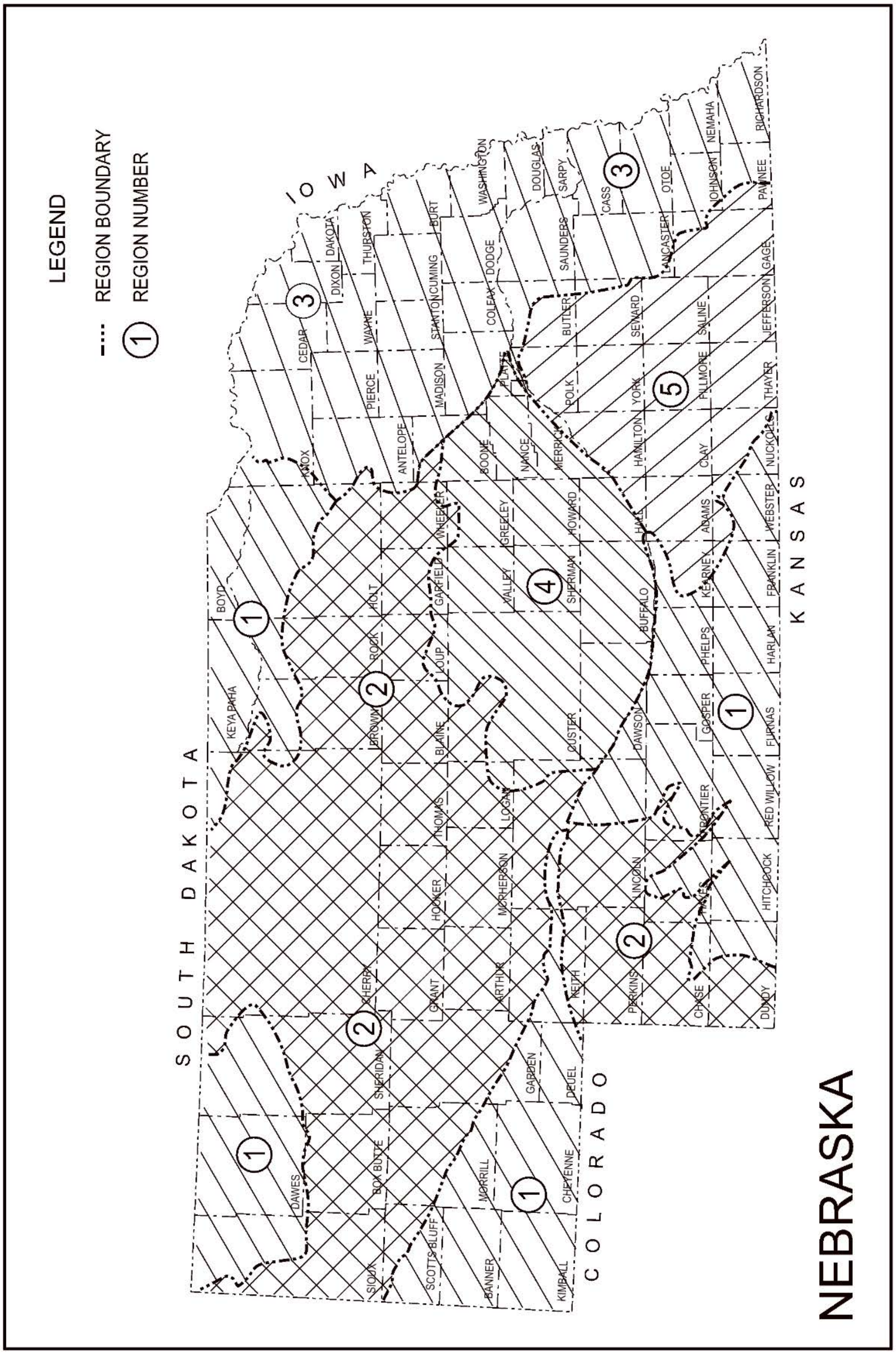


Exhibit 1.12 USGS Hydrologic Regions of Nebraska for Use with Regression Equation Method (640 Ac. or Greater) (Reference 1.7)

Return Period	Region 1	Region 2
2	$Q_2 = 1.965\ A_c^{0.493}\ (P\text{-}13)^{1.44}$	$Q_2 = 0.269\ A_c^{0.912}\ S^{0.967}\ SN10^{2.337}$
10	$Q_{10} = 211.7\ A_c^{0.324}\ (P\text{-}13)^{0.314}$	$Q_{10} = 0.109\ A_c^{0.9917}\ S^{1.653}\ SN10^{2.607}$
50	$Q_{50} = 6366\ A_c^{0.211}\ (P\text{-}13)^{-0.630}$	$Q_{50} = 0.0845\ A_c^{1.036}\ S^{2.005}\ SN10^{2.632}$
100	$Q_{100} = 23553\ A_c^{0.170}\ (P\text{-}13)^{-1.011}$	$Q_{100} = 0.0816\ A_c^{1.051}\ S^{2.119}\ SN10^{2.615}$
200	$Q_{200} = 82183\ A_c^{0.131}\ (P\text{-}13)^{-1.382}$	$Q_{200} = 0.0816\ A_c^{1.064}\ S^{2.216}\ SN10^{2.587}$
500	$Q_{500} = 400713\ A_c^{0.082}\ (P\text{-}13)^{-1.863}$	$Q_{500} = 0.0844\ A_c^{1.079}\ S^{2.326}\ SN10^{2.536}$
	Region 3	**Region 4**
2	$Q_2 = 7.57\text{x}10^{-10}\ A_c^{0.815}\ S^{0.599}\ P^{7.099}$	$Q_2 = 341.4\ A_c^{0.443}\ L^{0.126}\ (T_3\text{-}43)^{-2.062}$
10	$Q_{10} = 2.55\text{x}10^{-8}\ A_c^{0.722}\ S^{0.505}\ P^{6.657}$	$Q_{10} = 4741\ A_c^{0.914}\ L^{-0.783}\ (T_3\text{-}43)^{-1.960}$
50	$Q_{50} = 8.19\text{x}10^{-7}\ A_c^{0.688}\ S^{0.492}\ P^{5.908}$	$Q_{50} = 19516\ A_c^{1.285}\ L^{-1.411}\ (T_3\text{-}43)^{-1.903}$
100	$Q_{100} = 3.26\text{x}10^{-6}\ A_c^{0.681}\ S^{0.497}\ P^{5.581}$	$Q_{100} = 31008\ A_c^{1.433}\ L^{-1.648}\ (T_3\text{-}43)^{-1.876}$
200	$Q_{200} = 1.37\text{x}10^{-5}\ A_c^{0.677}\ S^{0.504}\ P^{5.226}$	$Q_{200} = 46677\ A_c^{1.573}\ L^{-1.871}\ (T_3\text{-}43)^{-1.850}$
500	$Q_{500} = 9.20\text{x}10^{-5}\ A_c^{0.673}\ S^{0.516}\ P^{4.740}$	$Q_{500} = 75811\ A_c^{1.752}\ L^{-2.148}\ (T_3\text{-}43)^{-1.819}$
	Region 5	
2	$Q_2 = 0.00137\ A_c^{0.790}\ S^{0.777}\ I_{24,2}^{8.036}$	
10	$Q_{10} = 0.00126\ A_c^{0.687}\ S^{0.683}\ I_{24,2}^{10.037}$	
50	$Q_{50} = 0.00240\ A_c^{0.632}\ S^{0.640}\ I_{24,2}^{10.467}$	
100	$Q_{100} = 0.00335\ A_c^{0.615}\ S^{0.628}\ I_{24,2}^{10.491}$	
200	$Q_{200} = 0.00464\ A_c^{0.599}\ S^{0.618}\ I_{24,2}^{10.490}$	
500	$Q_{500} = 0.00755\ A_c^{0.581}\ S^{0.606}\ I_{24,2}^{10.393}$	

Symbol	Characteristic	Unit of Measure	Figure
Q	Peak Discharge	cfs	N/A
A_c	Contributing drainage area	Sq. mi.	N/A
L *	Length from station to basin divide along main channel	mi.	N/A
S	Slope, measured from the elevations at .10 and .85 of the length (L) divided by 0.75L	ft./mi.	N/A
P	Mean annual precipitation, 1959 (1)	in.	EXHIBIT 1.14
$I_{24,2}$	Rainfall for a 2-year, 24-hour event, 1961 (1)	in.	EXHIBIT 1.15
SN10	Equivalent moisture content of snow as of March 15, 1964 (1)	in.	EXHIBIT 1.16
T_3	Normal daily March temperature, 1959 (1)	°F	EXHIBIT 1.17

(1) from Reference 1.6
* This length is from basin divide to the point of interest along the main channel.

Exhibit 1.13 Regional Regression Equations for NDOT (640 Ac. or Greater) (Source: Reference 1.7)

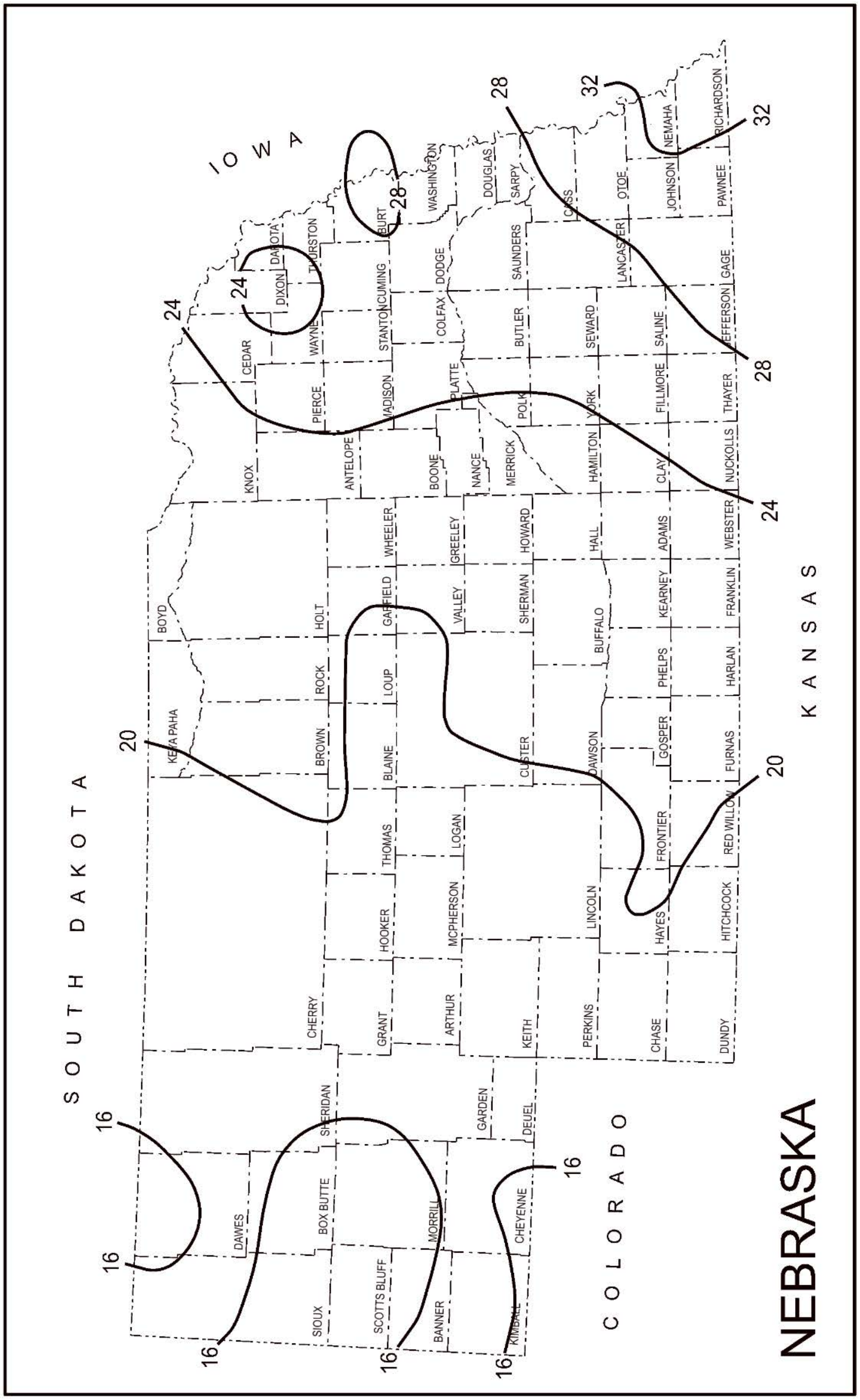


Exhibit 1.14 Mean Annual Precipitation (inches) (640 Ac. or Greater) (Reference 1.6)

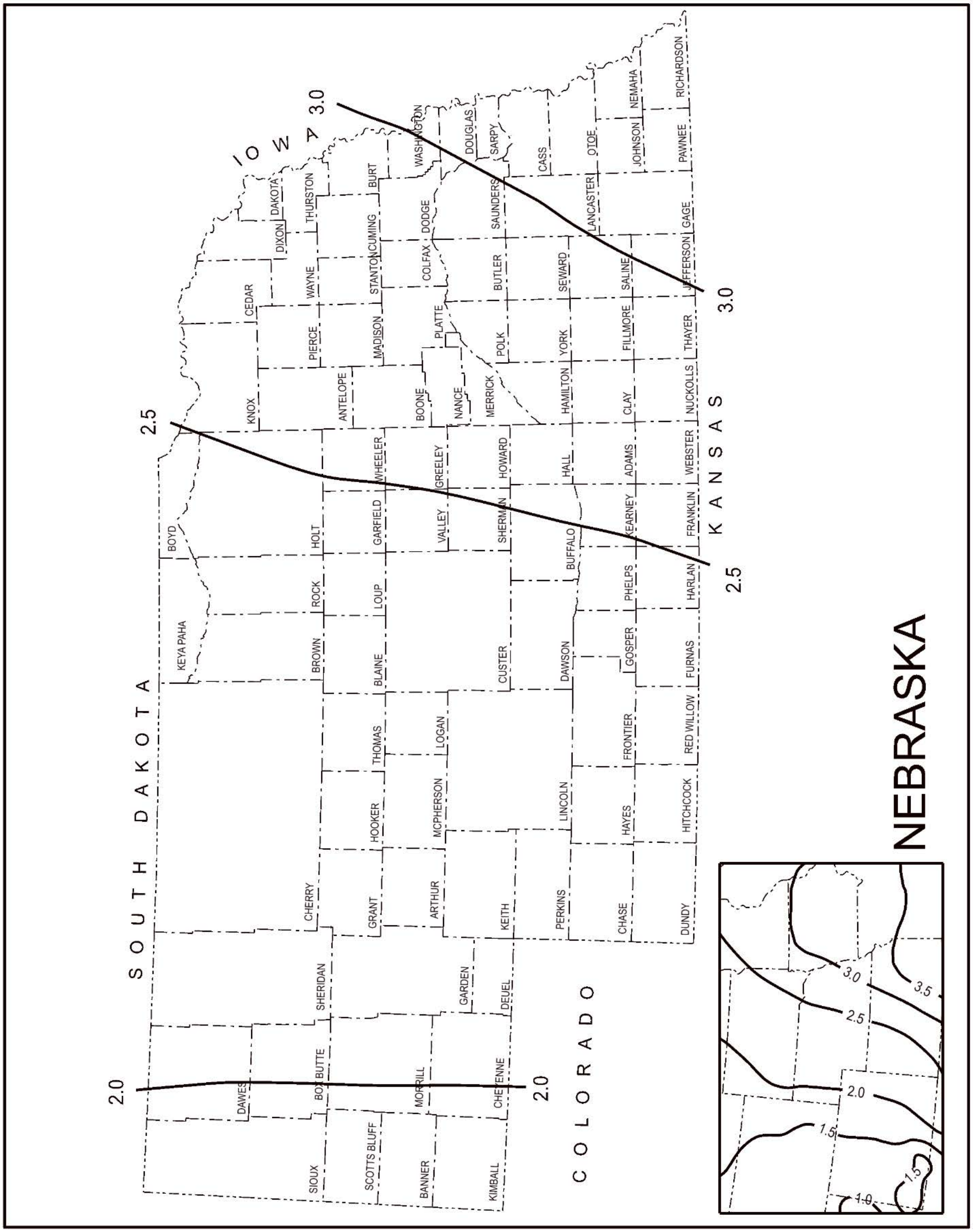


Exhibit 1.15 2-Year, 24-Hour Rainfall (inches) (640 Ac. or Greater)
(Reference 1.6)

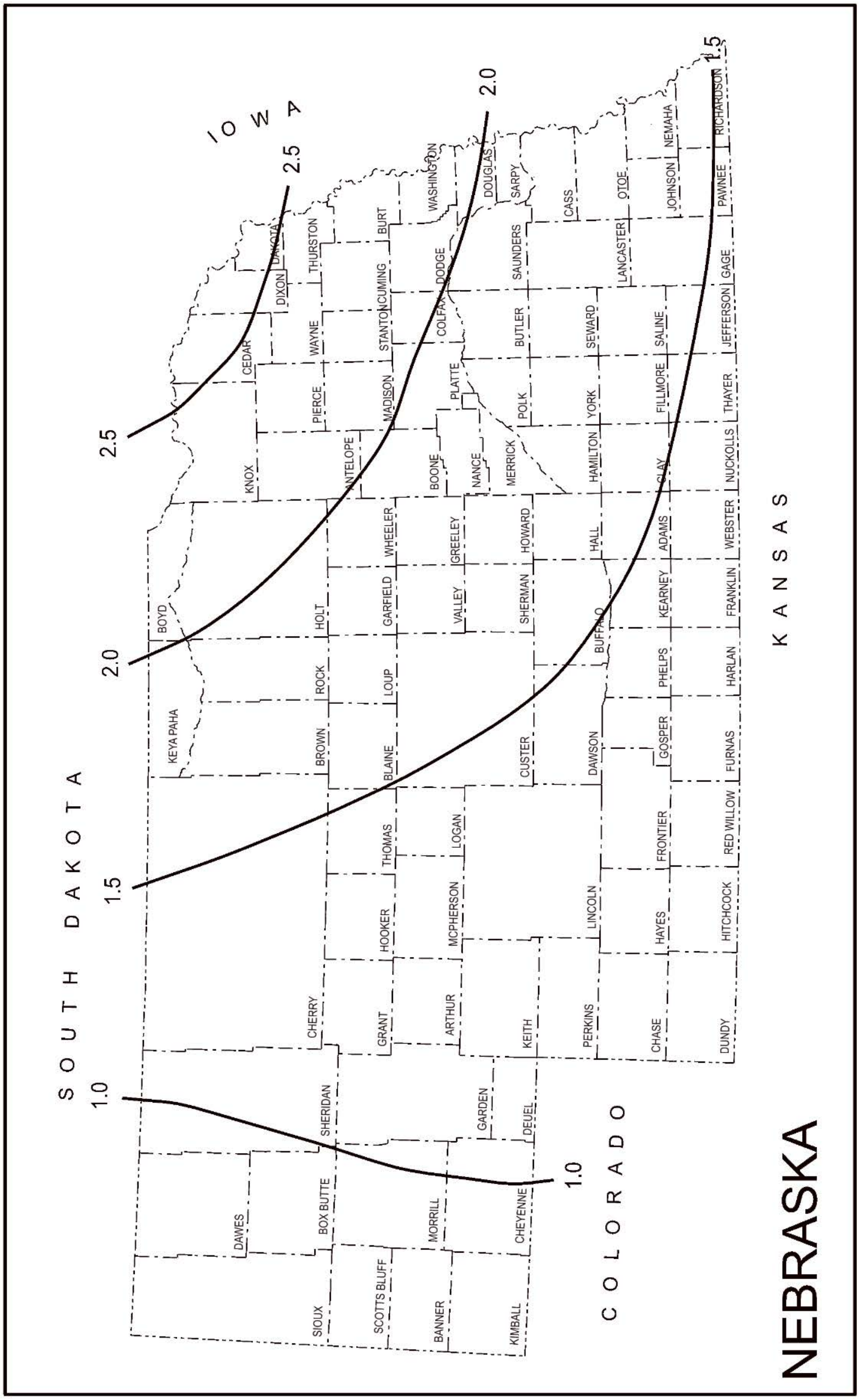


Exhibit 1.16 10% Probability-Equivalent Moisture Content of Snow as of March 15 (inches) (640 Ac. or Greater) (Reference 1.6)

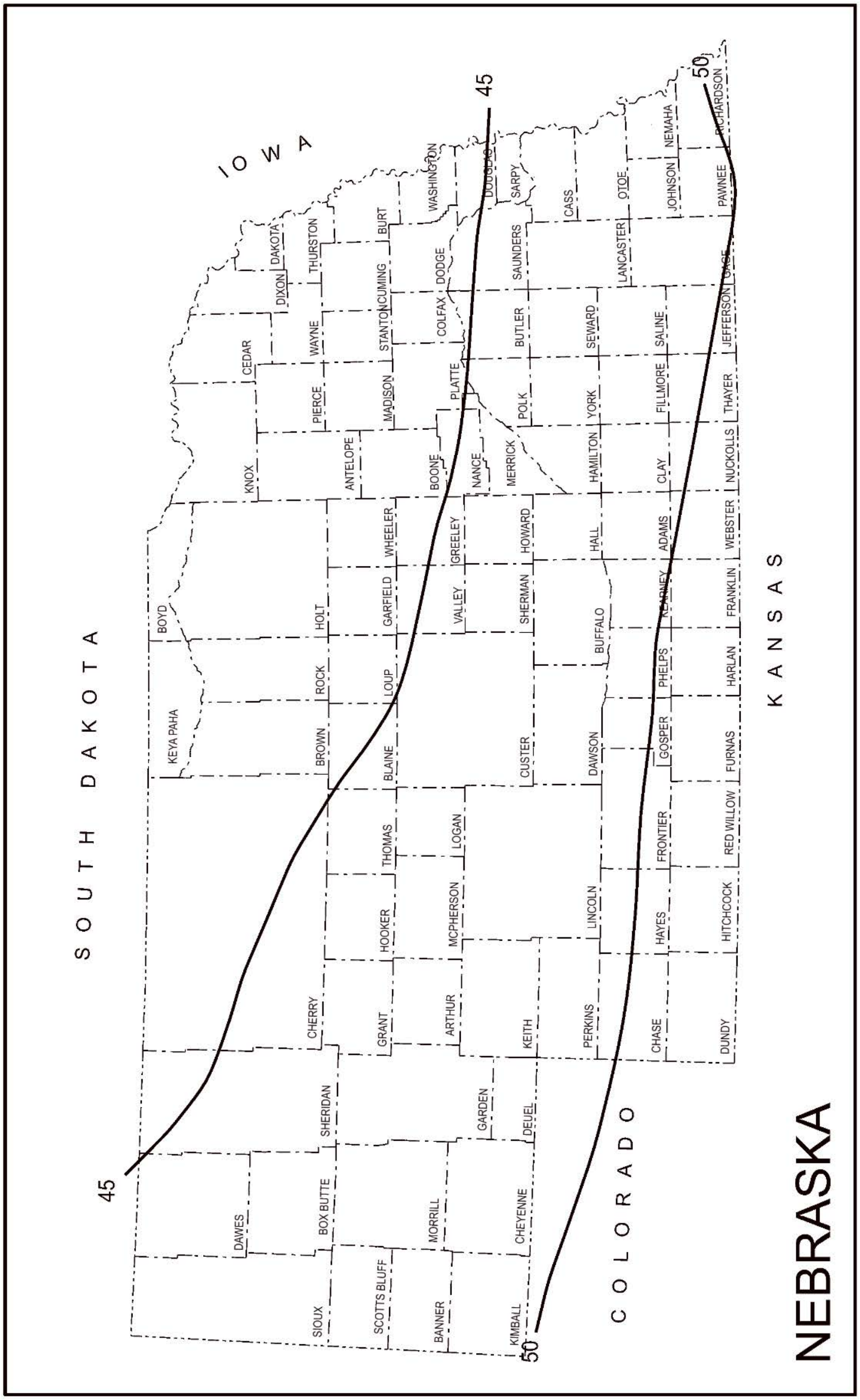


Exhibit 1.17 Normal Daily March Temperature (ºF) (640 Ac. or Greater)
(Reference 1.6)

7. OPEN CHANNELS

7.A General

This section contains a discussion of the basic fundamentals of open channel hydraulics and includes procedures for the design of open channels. The designer should consult Hydraulic Design Series 4: Design of Roadside Drainage Channels, (Reference 1.11), Water Surface Profiles, User Manual, (HEC2), (Reference 1.12), and Hydraulic Design Series 3: Design Charts for Open-Channel Flow, (Reference 1.13), for detailed explanations of specialized procedures and methods pertaining to open channel hydraulics.

The designer's primary considerations in the design of open channels are:

- Water surface elevations.
- Maximum allowable velocities for various channel linings.
- Distribution of flow (main channel, overbanks) within the total channel cross-section.
- Non-erosive channel slope.

For information on channel changes see the **Hydraulics Engineer** and the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 4.B.6, (Reference 1.24).

7.B Types of Open Channel Flow

There are several types of flow possible in open channels. A brief description and discussion of these follow.

Open channel flow is classified as:

- Uniform or non-uniform.
- Steady or unsteady.
- Subcritical, critical or supercritical (See Section 7.B.2).

Non-uniform, unsteady, subcritical flow is the most common type of flow in open channels. However, due to the complexity and difficulty involved in the analysis of this type of flow, most hydraulic computations are made with certain simplifying assumptions that allow the application of steady, uniform (or gradually varied) flow principles.

The use of **steady flow methods** assumes that the discharge at a point does not change with time, and the use of **uniform flow methods** assumes that there is no change in velocity, in magnitude or direction with distance along a streamline. **Steady, uniform flow** is thus characterized by constant velocity and flow rate from section to section along the channel.

Steady, uniform flow is an idealized concept of open channel flow, which seldom occurs in natural channels and is difficult to obtain even in model channels. However, for most practical highway applications the flow is assumed to be steady, and changes in width, depth or direction (resulting in non-uniform flow) are sufficiently small that flow can be considered uniform. For these reasons, use of uniform flow theory is usually within acceptable degrees of accuracy.

7.B.1 Critical Depth

Critical depth is the depth at which a given quantity of water flows with minimum content of energy. In a given channel, critical depth occurs when the specific energy (depth + velocity head) is at a minimum. Critical depth is important as a hydraulic “control point”, which is a location along the channel or culvert where depth of flow can be computed directly.

Critical depth is particularly helpful in the hydraulic analysis of culverts. Since flow must pass through critical depth when changing from subcritical (deeper, tranquil) flow to supercritical (shallow, rapid) flow, critical depth typically occurs at the following locations:

- Abrupt changes in channel or culvert slope when a flat slope is sharply increased to a steep slope (as in broken-back culverts).
- A channel constriction such as a culvert entrance.
- The unsubmerged outlet of a culvert on subcritical slope, discharging into a wide channel or free outfall (no tailwater present at the outlet).
- The crest of an overflow dam or weir.

The following relationship is used to calculate critical depth:

$$A^3/T = Q^2/g \qquad \text{Eq. 1.3}$$

where:
A = Cross-sectional area of channel, ft.^{2};
T = Top width of water surface, ft.;
Q = Discharge cfs;
g = Acceleration of gravity = 32.2 ft./s^2.

As can be seen from this equation, critical depth is dependent on channel geometry (shape) and discharge **only.** It is independent of channel slope and roughness. This means that for a given flow rate, critical depth remains constant throughout the channel or culvert length, even throughout a single or double broken-back culvert.

7.B.2 Froude Number

The Froude number is a dimensionless number that represents the ratio of inertial to gravitational forces. It is defined by the following equation:

$$Fr = V/(gD)^{0.5} \qquad \text{Eq. 1.4}$$

where:
- V = Velocity in the channel, ft./sec.;
- g = Acceleration of gravity = 32.2 ft./s^2;
- D = Hydraulic depth = (Flow Area/Top Width), ft.

- **Critical flow** exists when inertial forces and gravity are equal, (Fr = 1.0).
- **Supercritical flow** (Shallow, Rapid flow) exists when the inertial forces are greater than gravity forces (High Velocity), (Fr > 1.0).
- **Subcritical flow** (Deeper, Tranquil flow) exists when inertial forces are less than gravity forces (Deep pool of slow moving water), (Fr < 1.0).

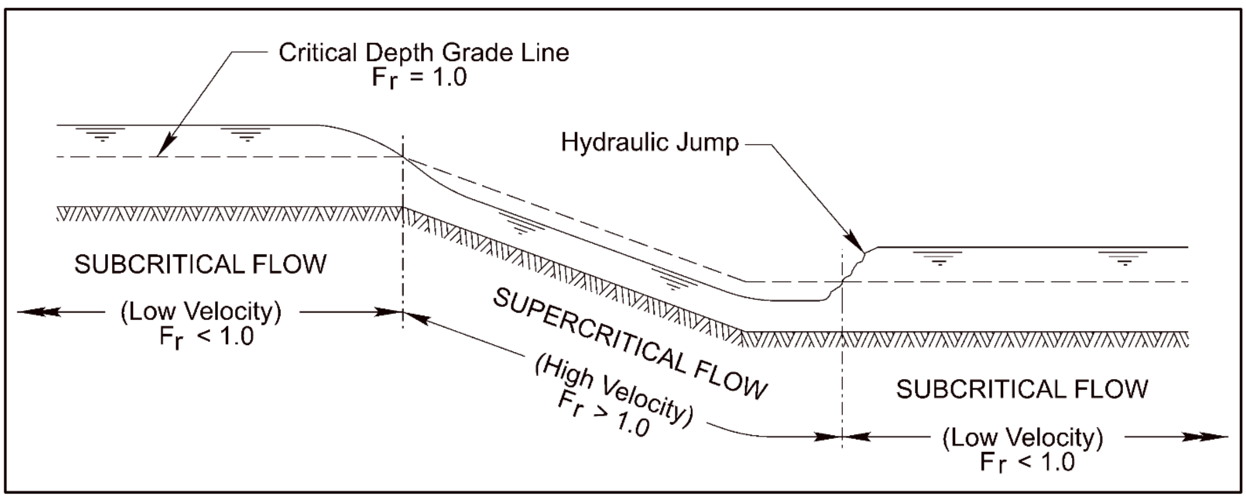


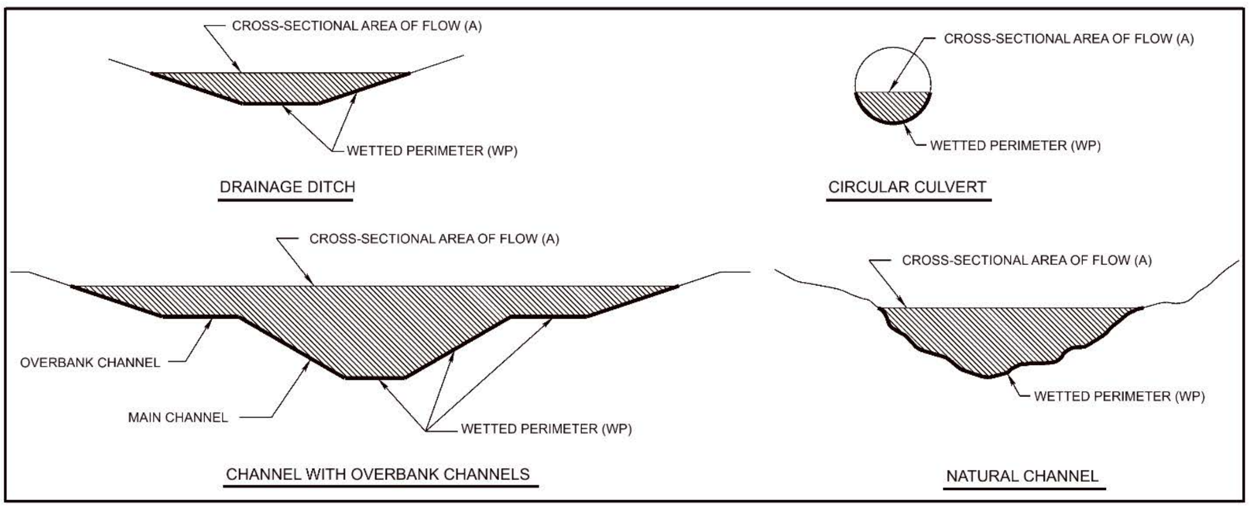
Exhibit 1.18 Water Surface Profile Illustrating Hydraulic Jump

7.C Open Channel Equations

An open channel must be designed to convey the peak runoff rate for the selected design storm frequency. The hydraulic capacity of an open channel can be determined from Manning's equation for evaluating uniform flow in open channels:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{Eq. 1.5}$$

where:
- V = Velocity of flow, ft./sec.;
- R = The hydraulic radius defined as the cross sectional area of flow (A) divided by the wetted perimeter (WP) or (A/WP), ft.;
- S = Slope of the hydraulic grade line, ft./ft.;
- n = Manning's roughness coefficient.



Manning's equation can be solved by using the nomograph presented in EXHIBIT 1.19. Roughness coefficients for use in Manning's equation for open channels are presented in Appendix B, "Manning's Coefficient, n".

If a channel cross section is irregular in shape such as a channel with a relatively narrow, deep main channel and wide, shallow overbank channels, the cross section should be subdivided and the discharge computed separately for the main channel and the overbank channels. The same procedure is used when parts of the cross section have different roughness coefficients. In computing the hydraulic radius of the subsections, the water depth common to adjacent subsections is not counted as wetted perimeter. For additional information see the **AASHTO** Model Drainage Manual, (Reference 1.1), and Hydraulic Design Series No. 3, Design Charts for Open Channel Flow, (Reference 1.13).

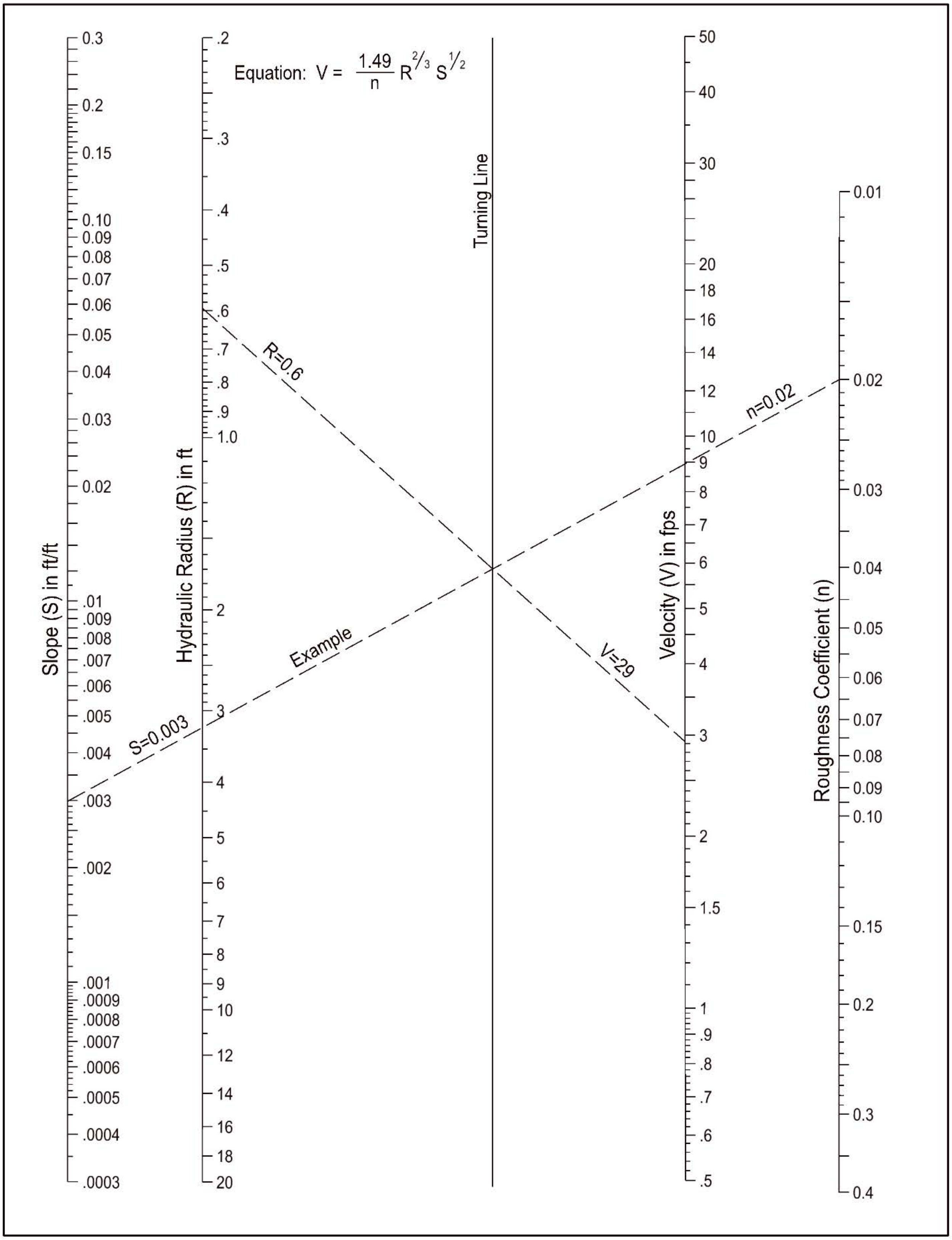


Exhibit 1.19 Nomograph for Solution of Manning's Equation, Open Channel

8. CULVERT DESIGN

A highway generally acts as a barrier to the flow of water in a stream or channel where the highway crosses the watercourse. Culverts are conduits for conveying water from a stream or channel through the highway embankment. In addition to their hydraulic function, culverts must also support construction equipment, highway traffic, and earth loads. Therefore, culvert design involves both hydraulic and structural design.

Culverts are of great importance to adequate drainage and the integrity of the highway facility. Although the cost of individual culverts is usually relatively small, the total cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, the total cost of maintaining highway hydraulic features is substantial, and culvert maintenance may account for a large share of these costs. Improved traffic service and a reduction in the total cost of highway construction and maintenance can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

Any structure which measures less than 20 ft. from the inside face of the exterior wall to the inside face of the exterior wall (including interior walls) along the centerline of the roadway is classified as a culvert; any structure which measures 20 ft. or greater for the same dimensions is classified as a bridge or major structure. The **Bridge Division** will be responsible for the hydrology and design of all major structures.

8.A Hydraulic Analysis

Hydraulic design procedures described in this section are based on Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts, (Reference 1.14). Hydraulics analysis includes computation of the:

- Drainage area.
- Design flow.
- Allowable headwater.
- Headwater at design flow.

8.A.1 New and Reconstructed Projects

Hydraulic analysis of culverts is required for all new and reconstructed projects, even if the existing vertical alignment is used in place. On new and reconstructed projects culvert extensions should be discussed at the plan-in-hand inspection with the **District Engineer**. The existing culvert size should be evaluated to determine if it is still within the allowable range.

8.A.2 3R Projects

3R projects do not require a hydraulic analysis of culverts, unless there is a known hydraulic problem, since these projects normally involve only the driving surface. Some 3R projects may require extending the culvert end beyond the fixed obstacle clear distance. In these instances, the existing culvert may be extended without a hydraulic analysis. For additional information, see the Roadway Design Manual, Chapter Seventeen: Resurfacing, Restoration and Rehabilitation (3R) Projects, (Reference 1.24).

8.A.3 Culvert Design Features

Culvert locations will be noted on the preliminary plans. A design discharge should also be specified. All locations, design discharge computations, and culvert data will be reviewed at the plan-in-hand inspection (See the Roadway Design Manual, Chapter Two: Roadway Design Process, Section 7, Reference 1.24). Additional information is required if the culvert is located in a floodplain (See Section 5.C). See the **Roadway Design Hydraulics Engineer** for additional information on hydraulics analysis and culvert design in a floodplain.

Culvert design involves consideration of the following factors:

- Inlet and outlet control.
- Culvert shape and cross-section.
- Location and material.
- Culvert length and extension.
- End treatments.
- Multiple installations.
- Inlet improvement.
- Outlet velocity.
- Culvert size.
- Slope and alignment.
- Camber.
- Bedding and fill requirements.

EXHIBITS E.1 AND E.2 of Appendix E, "Design Forms and Checklists", show a culvert design checklist and a broken-back culvert checklist. Designers are encouraged to use a separate checklist for each culvert as a documentation tool.

8.B Inlet and Outlet Control

Laboratory tests and field observations show two major types of culvert flow:

1. Flow with inlet control.
2. Flow with outlet control.

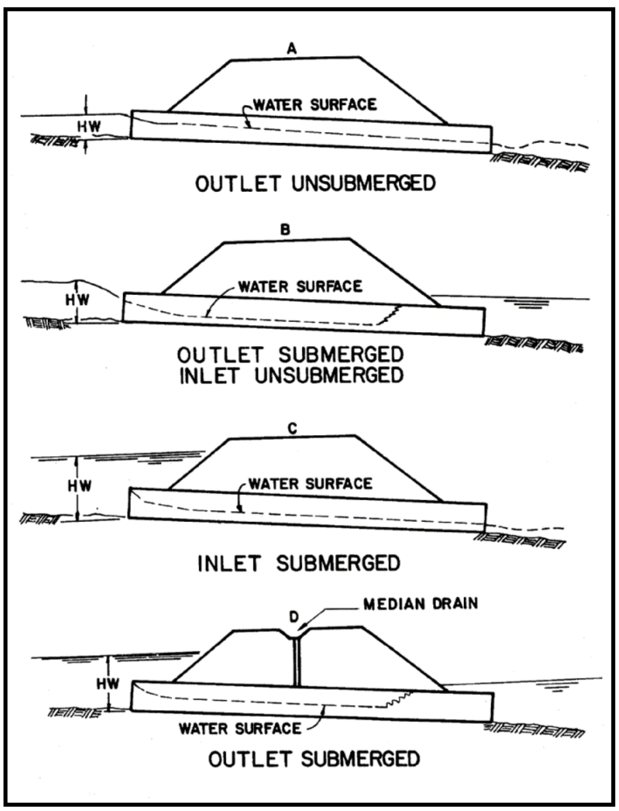
For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Hydraulic analysis of a culvert design includes determining the headwater elevation at the design discharge (See Section 8.A). This is done by comparing the inlet control headwater elevation against the outlet control headwater elevation and selecting the higher value. For additional information see Hydraulic Design Series 5: Hydraulic Design of Highway Culverts, (Reference 1.14) and Hydraulic Engineering Circular No. 13: Hydraulic Design of Improved Inlets for Culverts, (Reference 1.17).

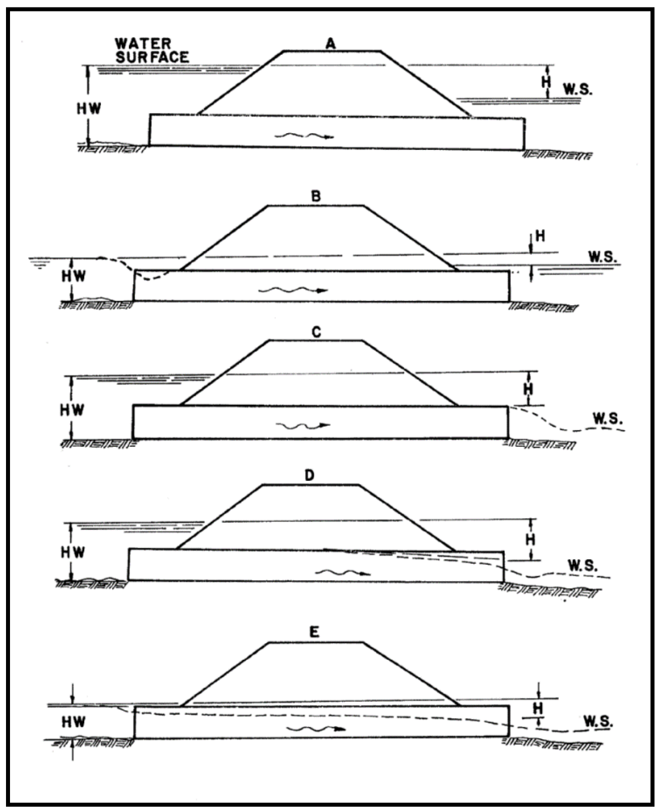
8.B.1 Inlet Control

In inlet control, the discharge capacity of a culvert is controlled by the conditions at the culvert entrance. Flow passes through critical depth shortly after entering the culvert, becoming high-velocity shallow (supercritical) flow in the culvert, (See EXHIBIT 1.20). Under inlet control, the cross-sectional area of the culvert barrel (opening size), the inlet geometry (culvert shape), entrance configuration (projecting, headwalls, wingwalls) and depth of the headwater at the entrance are of primary importance. Inlet control generally occurs when the culvert opening is not capable of accepting as much flow as the culvert barrel is able to convey. The efficiency of the culvert inlet can be greatly enhanced by beveling or tapering the opening, (See Section 8.K.2 and Hydraulic Engineering Circular No. 13: Hydraulic Design of Improved Inlets for Culverts, (Reference 1.17).

8.B.2 Outlet Control

In outlet control, the discharge capacity of a culvert is controlled by the downstream conditions including the head losses incurred entering the culvert, (friction), and exiting the culvert. Water flows through the culvert as low-velocity, deep (subcritical) flow. The culvert may flow completely or partially full (See EXHIBIT 1.20). Under outlet control, in addition to the parameters affecting inlet control, the barrel slope, length, and roughness are important. Also of importance is the tailwater elevation of the outlet. Outlet control generally occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept.





Inlet Control **Outlet Control**

Exhibit 1.20 Inlet and Outlet Control (Source: Reference 1.14)

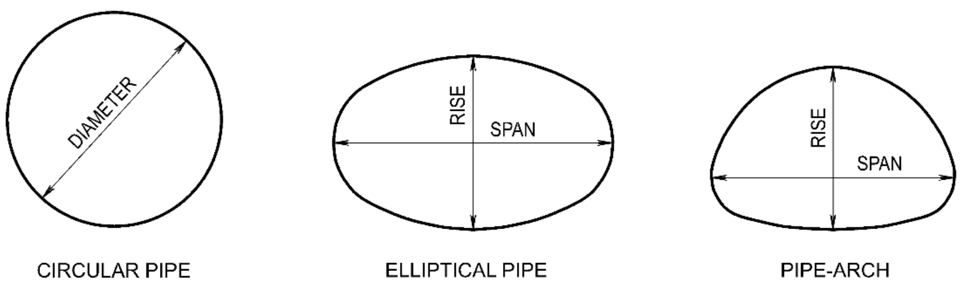
8.C Culvert Type, Material and Location

Culvert type selection includes the shape and cross section, choice of materials and number of culvert barrels or spans. The shapes and cross sections listed in EXHIBIT 1.21 are acceptable for highway culverts. EXHIBIT 1.22 shows equivalent pipe cross sections.

Appendix C, "Pipe Material Policy", describes locations and acceptable pipe materials to use for culverts. Corrugated metal pipe will not be permitted in the southeast counties of Gage, Nemaha, Richardson, Pawnee, Johnson, Otoe or any other locations that are designated by the **Materials and Research Division** (**M&R**) as unsuitable for corrugated metal pipe.

Shape/Cross Section	Comments
Circular	Most common; standard lengths and strength classes available
Pipe arch and elliptical	Used where cover is limited
Box or rectangle	Longer construction time required for cast-in-place construction; precast construction may be considered
Arches	Cause least obstruction to waterway

Exhibit 1.21 Culvert Shapes and Cross Sections



Circular Pipe		Concrete Pipe Horizontal - Elliptical			Concrete Pipe Arch-Type			Corrugated Metal Pipe-Arch ($2^2/_3$ in. x ½ in.)		
Diameter (in.)	Area (ft^2)	Span (in.)	Rise (in.)	Area (ft^2)	Span (in.)	Rise (in.)	Area (ft^2)	Span (in.)	Rise (in.)	Area (ft^2)
15	1.23	---	---	---	---	---	---	---	---	---
18	1.77	23	14	1.84	22	13 ½	1.6	---	---	---
21	2.41	---	---	---	---	---	---	25	16	2.16
24	3.14	30	19	3.28	28 ½	18	2.8	29	18	2.83
27	3.98	34	22	4.14	---	---	---	---	---	---
30	4.91	38	24	5.12	36 ¼	22 ½	4.4	36	22	4.42
33	5.94	42	27	6.31	---	---	---	---	---	---
36	7.07	45	29	7.37	43 ¾	26 5/8	6.4	43	27	6.36
42	9.62	53	34	10.21	51 1/8	31 5/16	8.8	50	31	8.65
48	12.57	60	38	12.92	58 ½	36	11.4	58	36	11.30
54	15.90	68	43	16.6	65	40	14.3	65	40	14.34
60	19.64	76	48	20.5	73	45	17.7	72	44	17.7
66	23.76	83	53	24.8	88	54	25.6	---	---	---
72	28.27	91	58	29.5	---	---	---	---	---	---

Notes: Dimensions do no include the wall thickness.
Refer to manufacturer's literature for larger pipe sizes.

Exhibit 1.22 Equivalent Pipe Cross Sections

8.D Culvert Lengths

Culvert lengths will be computed directly from the drainage cross section, which is drawn along the flow line. These measurement guidelines should be followed:

- All round and round equivalent culvert lengths will be measured along the longitudinal axis of the culvert.
- The length of concrete culvert pipe computed from the drawing should be specified to the next largest whole ft. and next even number for CMP.
- Pay lengths of pipe shall be measured from center to center of structure (e.g., inlet, junction box).
- Flared end sections, and the associated "Y" distances, are not included in the culvert length.
- Box culvert lengths shall be specified to the nearest ft.
- Additional pay allowances are made for connecting bands on extended corrugated metal structures with changes or breaks in horizontal or vertical alignment (See EXHIBIT 1.23).

EXHIBIT 1.24 shows the side slope grading for culvert ends while EXHIBIT 1.25 illustrates the pay length for culvert pipe. Refer to the Roadway Design Manual, Chapter Four: Intersections, Driveways and Channelization, Section 2.A.1, (Reference 1.24), for the driveway culvert length policy.

<table>
<tr><th></th><th>Diameter of Elbow (in.)</th><th>Length Allowance (lin. ft.)</th></tr>
<tr><td rowspan="3">For Both Concrete and CMP</td><td>Less than 36 in. round or equivalent pipe arch 36 in. x 22 in.</td><td>7</td></tr>
<tr><td>36 in. - 54 in. round or equivalent pipe arch 43 in. x 27 in. to 65 in. x 40 in.</td><td>6</td></tr>
<tr><td>Over 54 in. round or equivalent pipe arch 65 in. x 40 in.</td><td>5</td></tr>
<tr><td></td><td>Diameter of Connecting Band (in.)</td><td>Length Allowance (lin. ft.)</td></tr>
<tr><td rowspan="2">CMP only</td><td>21 in. or less round or equivalent pipe</td><td>3</td></tr>
<tr><td>Over 21 in. round or equivalent pipe</td><td>4</td></tr>
</table>

Exhibit 1.23 Additional Pay Length for Elbows and Connecting Bands

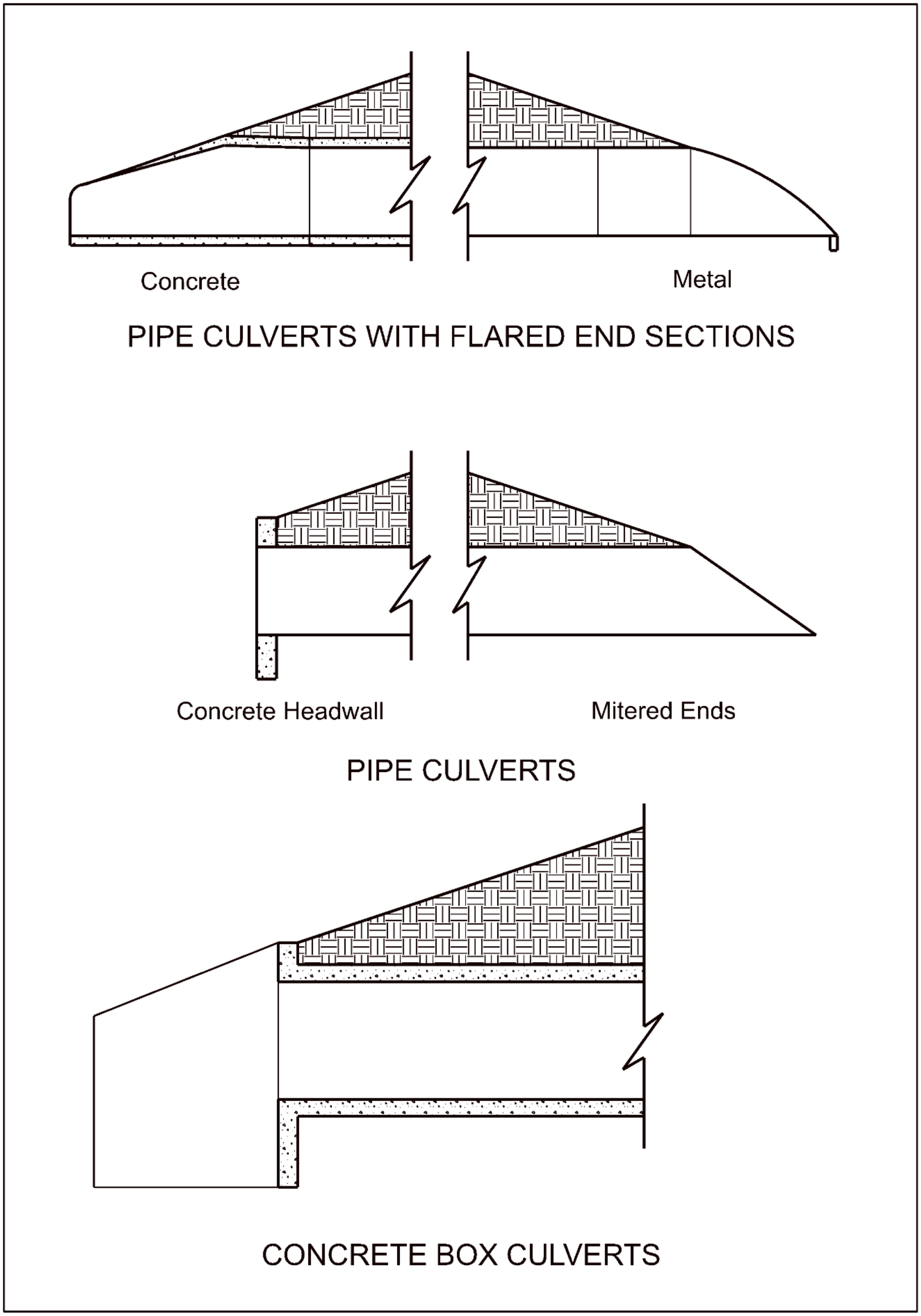


Exhibit 1.24 Side Slope Grading for Culverts

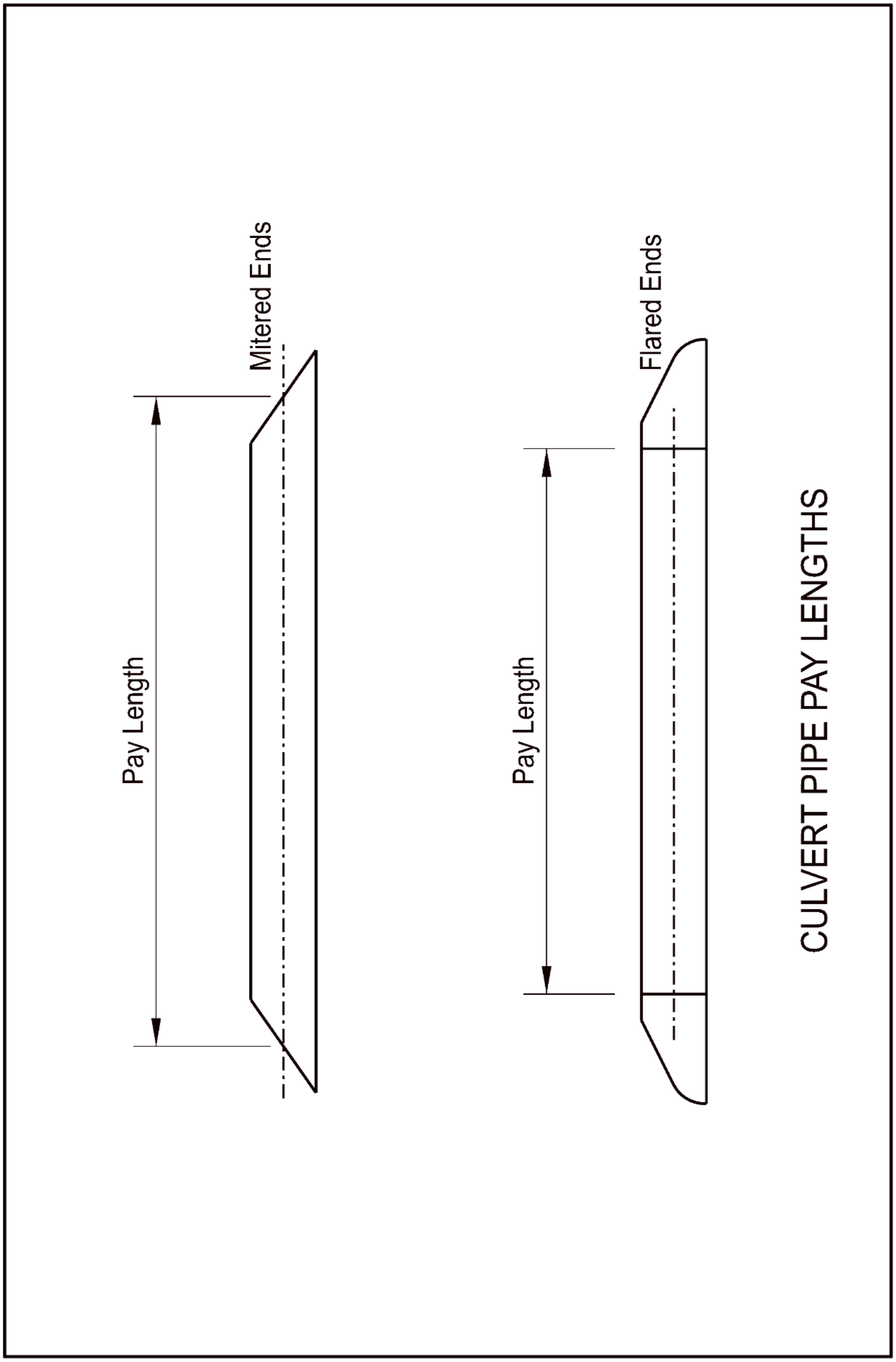


Exhibit 1.25 Pay Lengths for Culvert Pipe

8.E Multiple Barrels and Spans

In the case of box culverts, it is usually more economical to use a multiple span structure than a wide single span, due to a reduction in the thickness of the top slab. In some locations, multiple spans have a tendency to catch debris and clog the waterway. They are also susceptible to ice jams and the deposition of silt in one or more spans. Alignment of the culvert face normal to the approach flow and installation of debris control structures can help to alleviate these problems.

In the case of pipe culverts, multiple pipe installations often exhibit settlement after construction. Use of multiple pipes should be avoided whenever possible. However, if multiple pipes are used, sufficient space between pipes must be provided to allow for proper backfill and compaction to eliminate the settlement problem. Multiple pipe installations should desirably have 5 ft. or greater clearance from outside of pipe to outside of pipe. Backfill material for the minimum clear spacing of 1 ft. shall be flowable fill. Proper indigenous soils may be used for backfill material where spacing is greater than 5 ft.

Headwalls are preferred to flared end sections for multiple pipe installations where the headwall does not present an obstacle, (e.g. is outside of the clear zone). Flared end sections are also available (if concrete is not available) that permit 1 ft. minimum clear spacing between pipes.

8.F End Treatments

A flared end treatment is preferred over a headwall for pipe culverts from a safety standpoint. Flared end sections shall be used on rural culverts whenever feasible. The material of the flared end section generally shall match the pipe material unless plastic pipe is used, which requires a metal flared end section. Flared end sections may prove to be unsatisfactory for skewed culverts with low fills and the use of a headwall may be necessary. Installation of flared end sections on multiple pipe installations is preferred over cast-in-place concrete headwalls within the lateral obstacle clearance.

Headwalls on the inlet of a culvert help control erosion slightly better than flared end sections. Headwalls can function as a drop structure on the outlet of a culvert in locations where rapid flow and change in direction need to be addressed. Headwalls shall be designed in accordance with special plans requested from the **Bridge Division** and may be used for:

- Multiple pipe installations.
- Culverts with skews of 30° or over.
- Culverts with slopes too steep for flared end sections.
- Broken-back culverts where the possibility of slippage exists, e.g., drop pipes in backslopes.

Headwalls with a deeper footing (3 ft.) are needed for culverts placed on steep grades or in areas of potential headcutting. If deep footings are necessary, they should be included in the request to the **Bridge Division**.

8.G Headwater Elevation

Any culvert which constricts the natural stream flow will cause a rise in the upstream water depth to some extent. The depth of water in the stream measured from the culvert inlet invert (flowline) is termed headwater.

The minimization of potential damage to adjacent property should be of primary concern in the design of all culverts. Additional areas of concern in the design of culverts are:

- The potential for damage to adjacent property is greater in urban areas because of the number and value of properties that can be affected.
- If roadway embankments are low, flooding of the roadway and delay to traffic are usually of concern, especially on highly traveled routes.
- If roadway embankments are high, the potential for flooding upstream of the roadway must be considered.

The desirable maximum allowable headwater for New and Reconstructed Projects will be the lower of the following, (known as the D + 1 policy):

- One foot above the top of the culvert (See EXHIBIT 1.26, Condition A).
- One foot below the top of the outside edge of the shoulder (See EXHIBIT 1.26, Condition B).

If headwater depth lower than the D + 1 policy maximum cannot be achieved, the designer should obtain approval from the **Roadway Design Unit Head** and document the decision in the project file. Risk assessment should be considered at all times.

Culvert installations under high fills may allow the designer an opportunity to use a high headwater or ponding to attenuate flood peaks. If high headwater is necessary, the designer should investigate the need to acquire additional right-of-way. If deep ponding is considered the possibility of catastrophic failure should be investigated because a breach in the highway fill could be quite similar to a dam failure. Whenever the design of a detention pond is considered, the roadway designer shall consult with the **Roadway Design Hydraulic Engineer**.

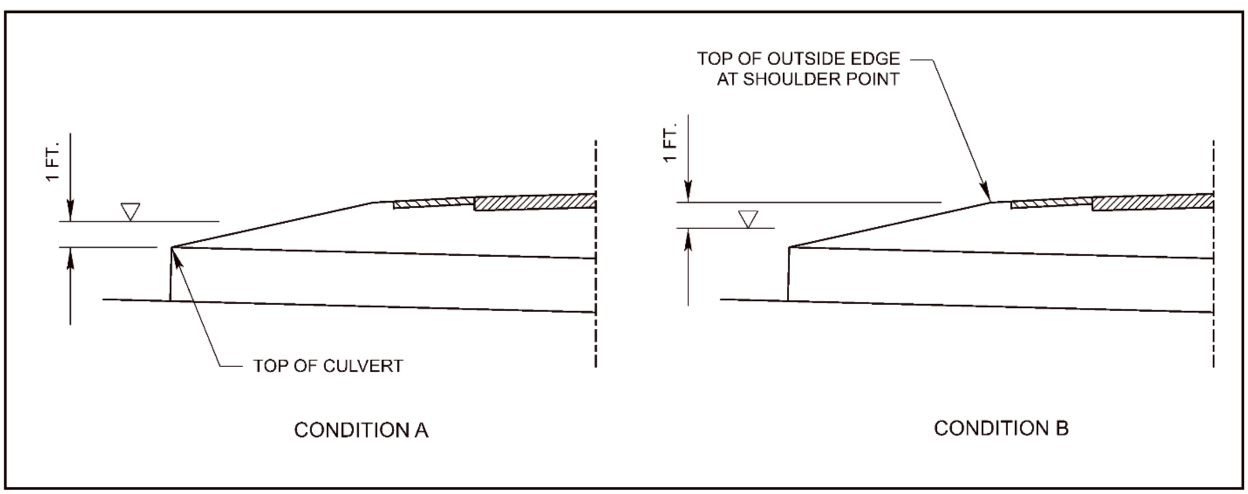


Exhibit 1.26 Maximum Allowable Headwater

8.H Tailwater Elevation

Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

A field inspection of the downstream channel should be made to determine whether there are obstructions that will influence the flow depth. Tailwater depth may be controlled by the stage in another stream, headwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

8.I Minimum Culvert Sizes

Culvert sizes will be determined in accordance with the charts and methods contained in Hydraulic Design Series 5: Hydraulic Design of Highway Culverts, (Reference 1.14), or from computer programs based on these circulars. For the input consisting of hydrological and topographical data, these programs assist the designer with the number of pipes or barrels, the dimensions of the pipes or barrels (diameter or span and rise) and headwater and outlet velocities for both inlet and outlet control conditions.

A minimum velocity of 2 ft./sec. should be maintained in the culvert to preclude settlement of silts and other solids. Velocities greater than 10 ft./sec. should be avoided when possible. See Chapter Two: Erosion and Sedimentation Control, Sections 6 and 7, for erosion control measures for high outlet velocities.

EXHIBIT 1.27 shows the minimum sizes of culverts used on state highways. The minimum allowable culverts on roads and streets under other jurisdictions shall be governed by the policy of those jurisdictions.

Type of Structure	Minimum Culvert Size
Cross Drain Pipe	24 in.
Median Drain Pipe	18 in.
Flume Pipe	15 in.
Drive Pipe	18 in. *
Box Culvert	4 ft. x 4 ft.
Storm Sewer	
Transverse	15 in.
Longitudinal	18 in.

*Normally 24 in. is used for drive pipe.

Exhibit 1.27 Minimum Culvert Sizes

8.J Hydraulic Design Procedure

Nomographs and other information for the hydraulic design of culverts are provided in Appendix F, "Nomographs and Charts for Culvert Design". EXHIBIT 1.28 provides entrance loss coefficients for different culvert edge configurations. An example procedure for concrete box culvert design is presented in Section 14.C.

The entrance loss coefficient, k_e, applies to the velocity head, $\frac{V^2}{2g}$, for determination of head loss at the entrance to a culvert operating full or partly full with control at the outlet.

$$\text{Entrance head loss, } H_e = k_e \frac{V^2}{2g}$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end, flared end section	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square edged	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope[1]	0.5
Pipe, or Pipe-Arch Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square-edge, flared end section	0.5
Mitered to conform to fill slope	0.7
End section conforming to fill slope[1]	0.5
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	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	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7

[1] Such end sections, made of either metal or concrete, are commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using EXHIBIT F.3 in Appendix F, "Nomographs and Charts for Culvert Design".

Exhibit 1.28 Entrance Loss Coefficients

8.K Culvert Entrance Configurations

The culvert entrance configuration is the cross sectional area and shape of the culvert face, and the type of inlet edge. When a culvert operates in inlet control, headwater depth and the entrance configuration determine the culvert capacity and the culvert barrel usually flows only partially full. Entrance geometry refinements or inlet improvements can be used to reduce the flow contraction at the inlet and to increase the capacity of the culvert without increasing the headwater depth.

Culverts operating in outlet control usually flow full at the design flow rate. Therefore, inlet improvements on these culverts only reduce the entrance loss coefficient, which results in only a small decrease in the required headwater elevation.

Even though the construction of a bevel-edged inlet, a side-tapered inlet or a slope-tapered inlet will increase the labor and material costs for the inlet portion of a new culvert (See Section 8.K.2), a substantial savings may be attained by a reduction in the size of the barrel which represents the major portion of the structure. Improved inlets may also be installed on existing culverts with inadequate flow capacity, thus avoiding the replacement of the entire structure or the addition of a new parallel structure. The greatest savings usually result from the use of improved inlets on culverts with long barrels. Short barrels, however, should also be checked especially when an improved inlet might increase the capacity sufficiently to avoid replacement of an existing structure.

One of the more important developments in hydraulic design of improved inlets for culverts is described in Hydraulic Engineering Circular No. 13, Hydraulic Design of Improved Inlets for Culverts, (Reference 1.17). This circular provides a good manual approach for hydraulic design of improved inlets for culverts.

Improved inlets for box culverts may be considered if any of the following conditions exist:

- A culvert barrel with slope greater than the slope for critical depth of flow at design discharge.
- An inlet-outlet flowline elevation differential that, by old criteria, would dictate a double broken-back culvert or a culvert with an energy-dissipating structure at its outlet.

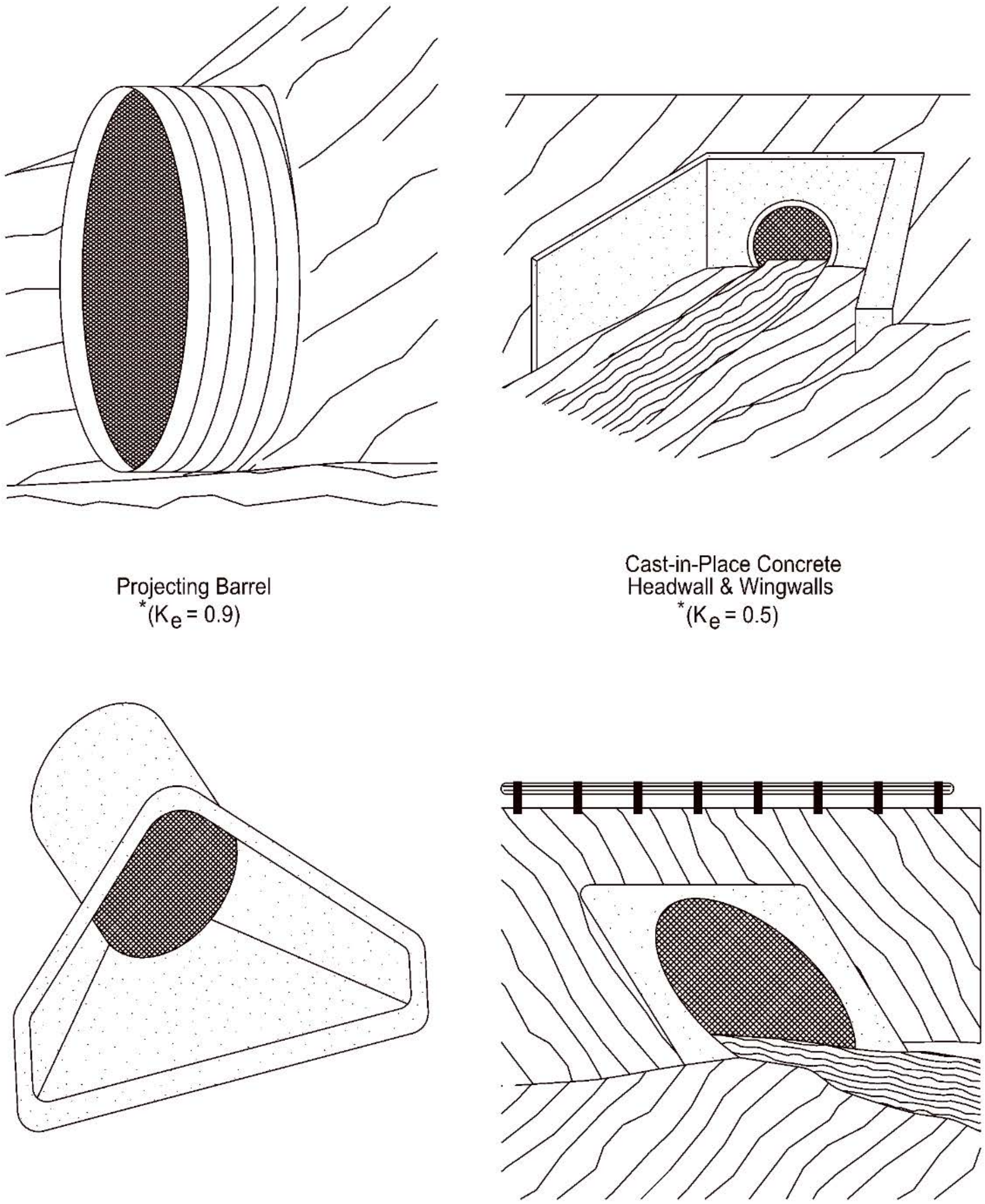
A standard code sheet shall be used when ordering a special design from the **Bridge Division** for the improved inlet.

8.K.1 Conventional Culvert Inlets

An overview of the various types of culvert entrance configurations is presented in the following sections. Conventional culvert entrances include the following:

- Thin-edge projecting inlet.
- Groove-end projecting inlet.
- Square edge inlet in headwall with wing walls.
- Mitered inlet with slope paving.
- Flared-end.

EXHIBIT 1.29 depicts various types of conventional culvert inlets.



Projecting Barrel
*(K_e = 0.9)

Cast-in-Place Concrete
Headwall & Wingwalls
*(K_e = 0.5)

Precast End Section
*(K_e = 0.5)

End Mitered to the Slope
*(K_e = 0.7)

* K_e = Entrance Loss Coefficient, (See Exhibit 1.28).

Exhibit 1.29 Conventional Culvert Inlets (Source: Reference 1.14)

8.K.2 Improved Inlets

Improved inlets include bevel-edged, side-tapered or slope-tapered inlets. The first type of inlet improvement is a bevel-edged inlet. A bevel is similar to a chamfer.

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. Tapered inlets may be either side-tapered or slope-tapered inlets. Side-tapered and slope-tapered box culvert inlets are shown in EXHIBITS 1.30 AND 1.31, respectively.

Improved inlets increase culvert performance primarily by reducing the contraction at the inlet control section. Also, a slope tapered inlet configuration depresses the inlet control section below the streambed to improve performance.

The designer should consult Hydraulic Engineering Circular No. 13: Hydraulic Design of Improved Inlets for Culverts, (Reference 1.17), for detailed discussion and design procedures for improved side tapered and slope tapered inlets.

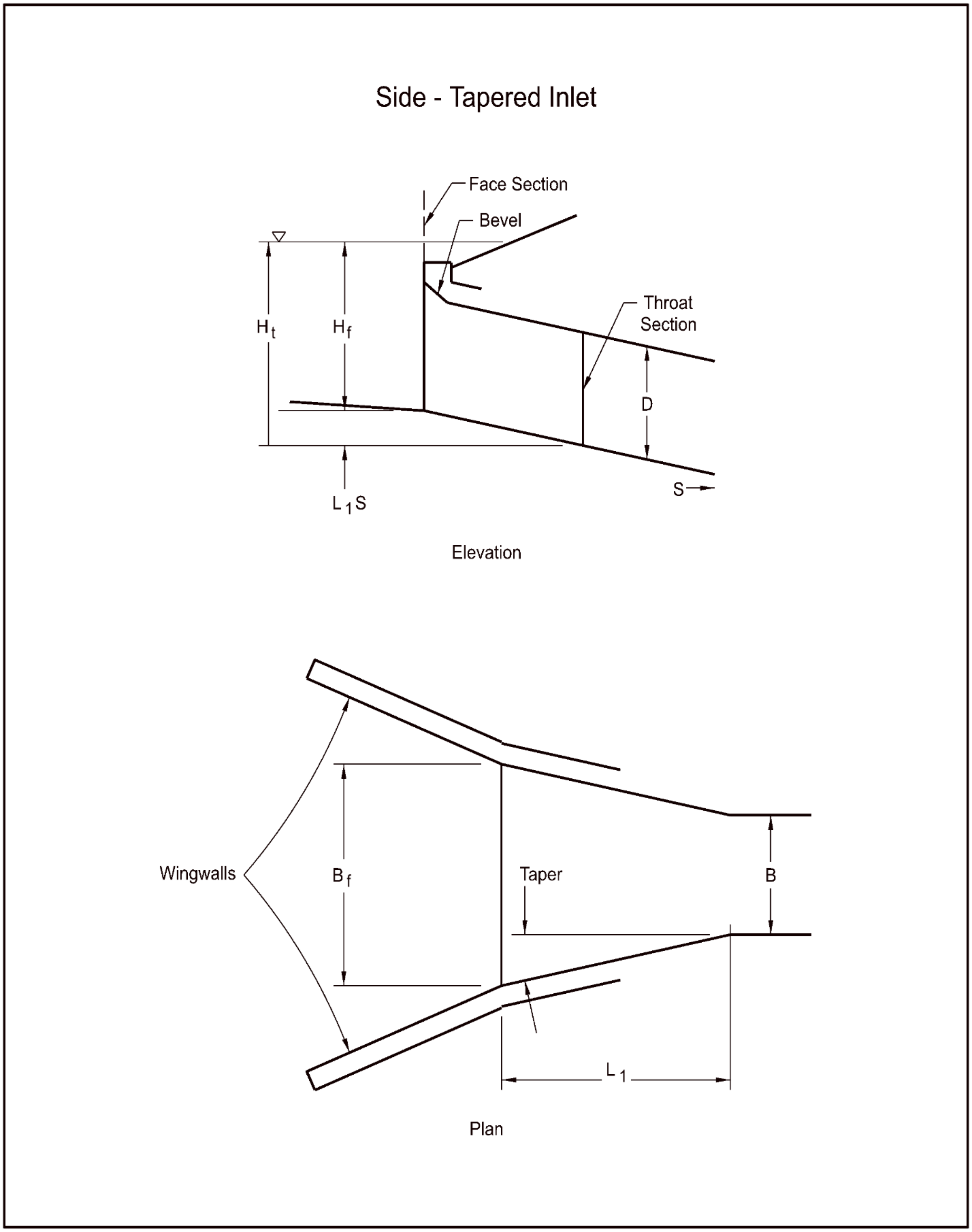


Exhibit 1.30 Side-Tapered Inlets (Source: Reference 1.17)

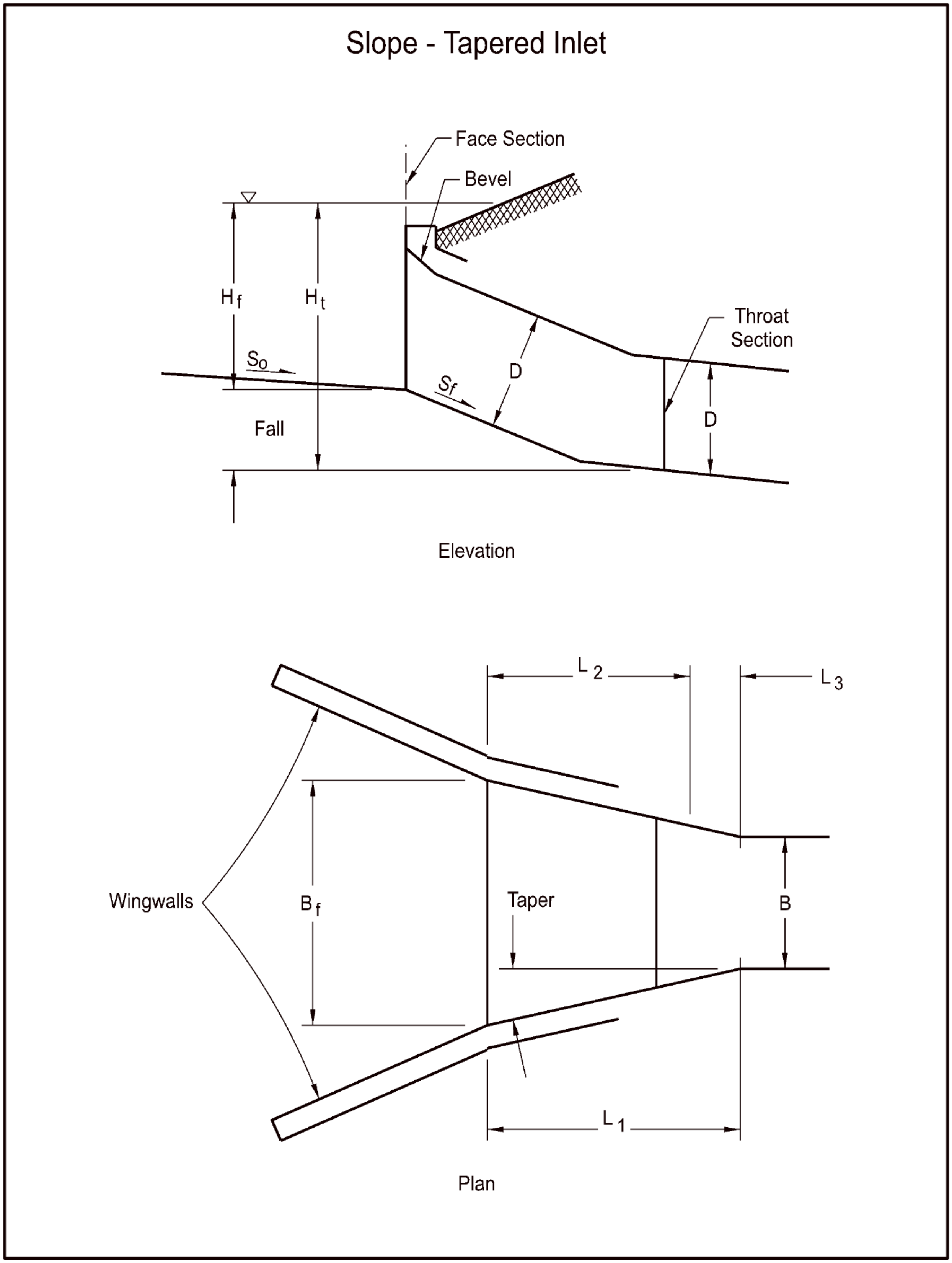


Exhibit 1.31 Slope-Tapered Inlets (Source: Reference 1.17)

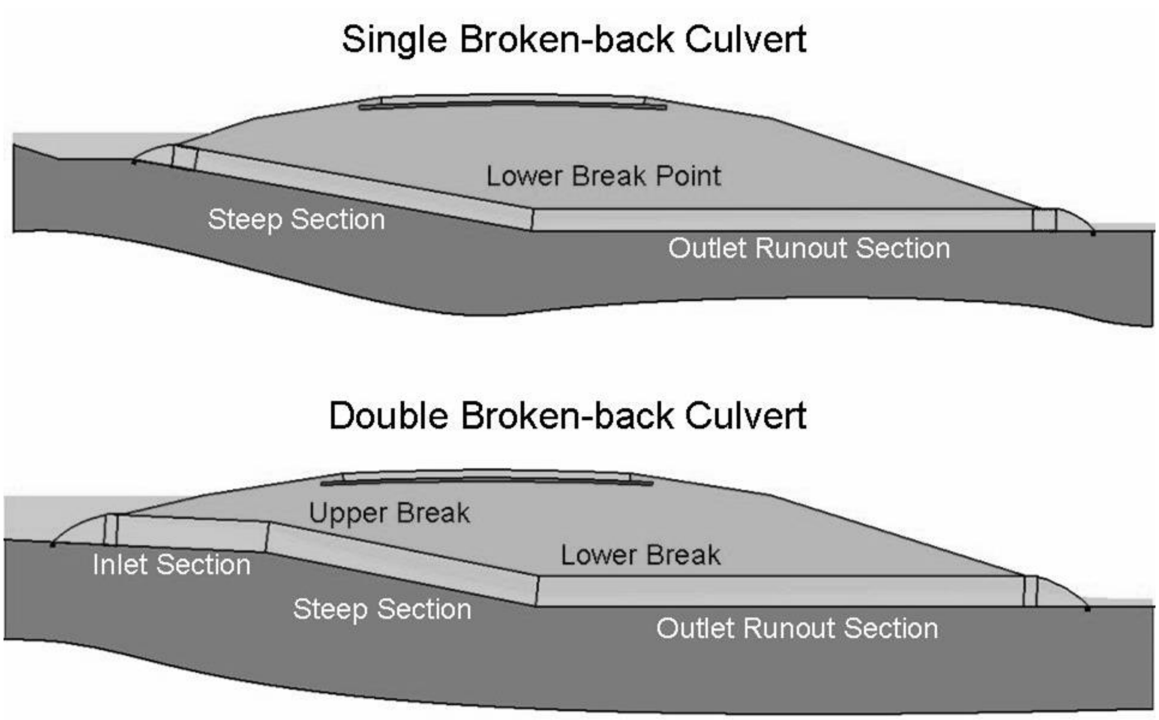
8.L Special Hydraulic Considerations

In addition to the hydraulic considerations discussed in the preceding sections, the designer must consider other factors in order to ensure the integrity of culvert installations under the highway:

- Broken-back Culverts, (See Section 8.L.1).
- Irregular profile and alignment, (See Section 8.L.2).
- Anchorage, (See Section 8.L.4).

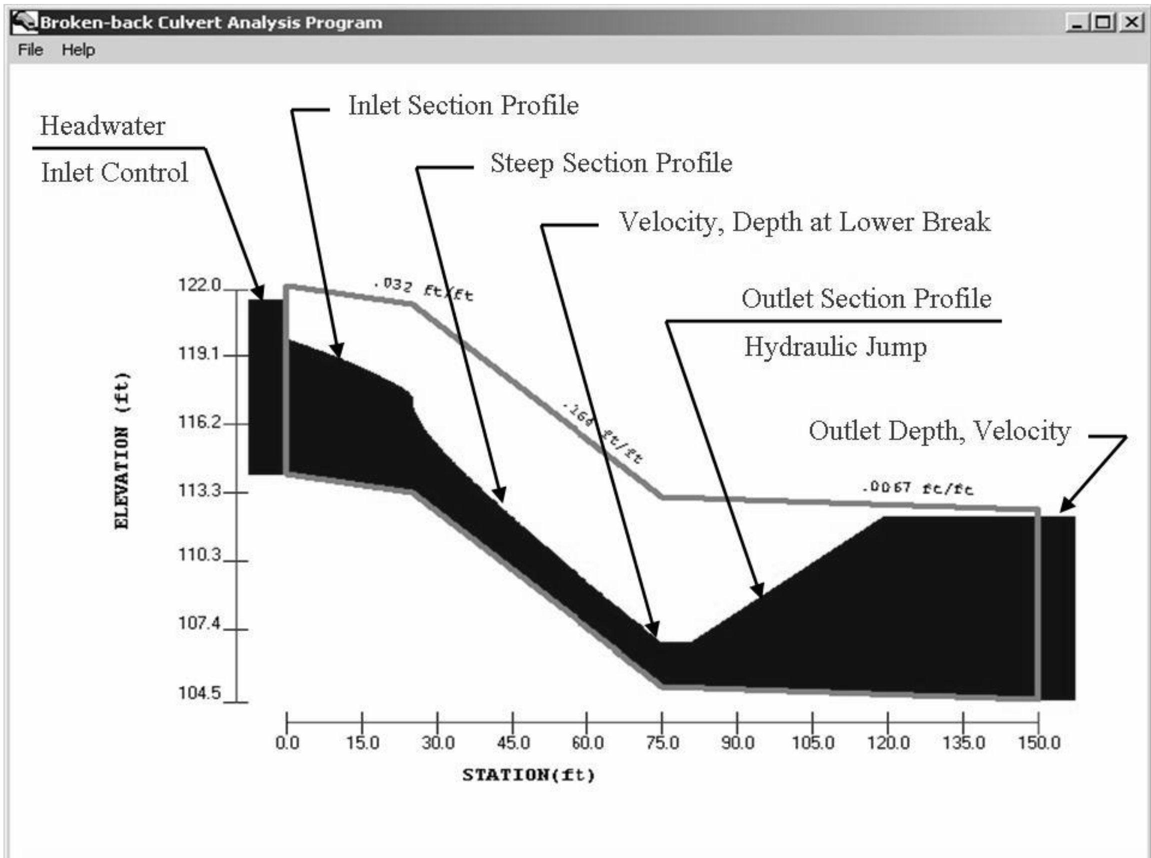
8.L.1 Broken-Back Culverts

Abrupt changes in slope or direction are not desirable from a maintenance and construction standpoint. However, at locations where the inlet is substantially higher than the outlet, culverts referred to as "broken-back", (with either one or two breaks in the vertical alignment), are commonly constructed.



The total hydraulic performance of these culverts is difficult to analyze without the aid of a computer program written for such a purpose. In 1998, research was completed at the University of Nebraska to develop the Broken-back Culvert Analysis Program (BCAP) for the **Nebraska Department of Transportation**. BCAP automates the procedures necessary to design and analyze a broken-back culvert.

BCAP uses energy equations to compute the water surface profiles (WSP) for each section within the culvert. The boundary conditions for the upstream and downstream end of each culvert segment is evaluated to determine the proper WSP for that segment. Each time the program computes a WSP elevation it also tests if conditions are such that a hydraulic jump may occur. A hydraulic jump is an almost instantaneous rise in the water surface as flow changes from shallow, rapid (supercritical) flow to deeper, tranquil (subcritical) flow. Subcritical flow is desirable at the culvert outlet to reduce the erosive potential of the flow as it exits the culvert. A hydraulic jump will occur within a broken-back culvert when there is sufficient roughness within the culvert barrel, sufficient tailwater at the outlet, or both.



Typical BCAP Output Data

Many broken-back culverts are constructed to control head cut erosion, when there is great differential between the inlet and outlet elevations. In broken-back culverts, velocity of flow is greatest at the lower break due to acceleration in the steeply sloped segment. It is often advantageous to specify corrugated interior pipes for these culverts to help reduce velocity between the lower break point and the culvert outlet. In many cases, BCAP can be used to optimize the length of the outlet section and provide for velocity reduction between the lower break point and the culvert outlet.

Through testing and extensive use of the BCAP software, it has been shown that culverts with a smooth interior, having a Manning's n-value of 0.013 or less, do not provide sufficient hydraulic roughness to effectively slow the velocity in the outlet runout section of a broken-back culvert. It is suggested that, whenever possible, pipe material with a Manning's n-value of 0.022 or greater be used for the outlet runout section of broken-back culverts.

BCAP is available for free download at: (http://www.roads.nebraska.gov/business-center/design-consultant/custom-apps/).

8.L.2 Irregular Profile and Alignment

EXHIBIT 1.32 shows slope adjustments for skewed angles, (See the Roadway Design Manual, Chapter Ten: Miscellaneous Design Issues, Section 2.B, Reference 1.24). These figures are used to:

- Determine the length of pipe.
- Draw the cross-section for pipe extensions.

For example, if a culvert is placed on a 25° skew and the foreslope is 6:1, the resultant slope is 6.62:1.

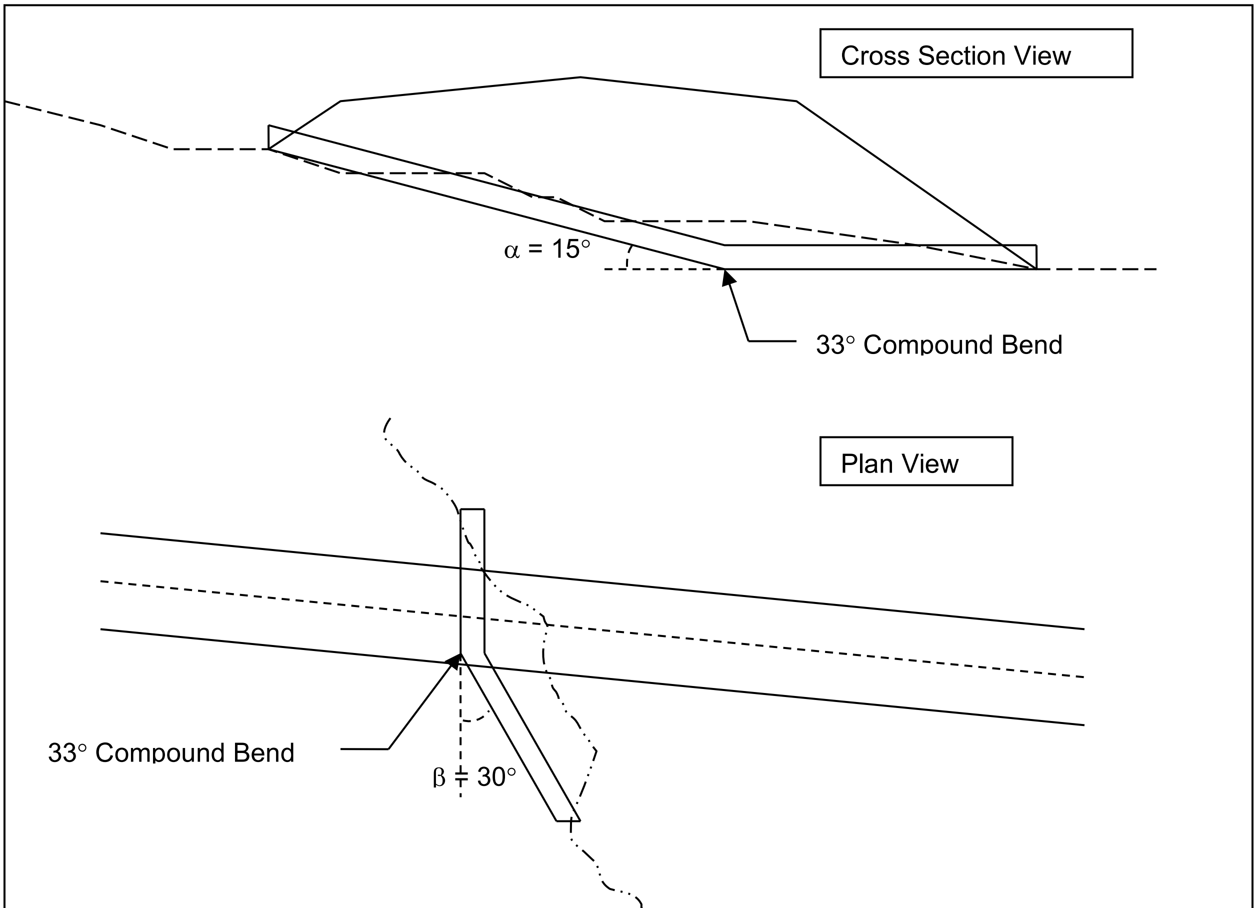
Skew Angle	Foreslope				
	1.5:1	2:1	3:1	4:1	6:1
5°	1.51	2.01	3.01	4.02	6.02
10°	1.52	2.03	3.05	4.06	6.10
15°	1.55	2.07	3.11	4.14	6.22
20°	1.60	2.13	3.19	4.26	6.38
25°	1.66	2.21	3.31	4.41	6.62
30°	1.73	2.31	3.46	4.62	6.92
35°	1.83	2.44	3.66	4.88	7.32
40°	1.96	2.61	3.92	5.22	7.84
45°	2.12	2.83	4.24	5.66	8.48
50°	2.33	3.11	4.67	6.22	9.34
55°	2.62	3.49	5.23	6.97	10.46

Exhibit 1.32 Slope Adjustment for Skewed Angles

8.L.3 Compound Bend Angle

When designing broken back culverts it is sometimes necessary to add a horizontal bend to the outlet section to direct water to a swale. When the horizontal bend occurs at the same location as a vertical break, a three dimensional compound bend occurs. This compound bend angle can be determined from the vertical and horizontal bend angles, (see EXHIBIT 1.33). The bend angle may be calculated using Equation 1.6.

Compound Angle, γ , = $180° - \text{Cos}^{-1}(-\cos(\alpha) \times \cos(\beta))$ Eq. 1.6



Calculation:

$\text{Cos}(15°) = 0.966$; $\text{Cos}(30°) = 0.866$

$\text{Cos}^{-1}(-0.966 \times 0.866) = 146.8°$

$180° - 146.8° = 33.2°$ **Compound Bend Angle = 33.2°**

EXHIBIT 1.33 Compound Bend Angle Example

8.L.4 Anchorage

Anchorage at the culvert entrance and also at the outlet of the culvert is necessary for the following:

- Protect the inlet and especially the outlet from undermining by scour.
- Protect against buoyant forces or uplift.
- Protect against separation of concrete pipe joints.

End anchorage can be in the form of headwalls, slope paving or piling. These techniques protect the slope from scour and preclude undermining of the culvert end. The culvert barrel, however, must be anchored to the end treatment in order to be effective.

Buoyant forces are produced when the pressure outside the culvert is greater than the pressure in the barrel. This condition can occur in a culvert in inlet control with a submerged upstream end and in culverts placed in areas of high groundwater.

Culvert ends projected through levees are susceptible to failure from buoyant forces if flap gates are used on the end. Generally, flexible barrel materials are most vulnerable to this type of failure because of their light weight and lack of resistance to longitudinal bending. Installation of headwalls and wingwalls will increase the dead load on the end of the culvert and protect it from uplift.

Rigid concrete pipe susceptible to separation of the pipe joints can be protected by installation of pipe couplers.

Inclusion of anchorage as a pay item should be discussed with the **Assistant Design Engineer**.

8.M Debris Control

Accumulation of debris at a culvert inlet can result in the culvert not functioning as designed. The consequences may include damages from inundation of the road and upstream property. The designer has two options for dealing with the debris problem: retain the debris upstream of the culvert or attempt to pass debris through the culvert.

If debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. If debris is to be passed through the structure or retained at the inlet, a relief opening should be considered, either in the form of increased barrel size, a vertical riser, or a relief culvert placed higher in the embankment.

It is often more economical to construct debris control structures after problems develop since debris problems do not occur at all suspected locations. The designer should consult Hydraulic Design Series 5: Hydraulic Design of Highway Culverts, (Reference 1.14) for design of debris control structures.

8.N Corrosion and Abrasion

Culvert material durability is as important a consideration to culvert installation as proper hydraulic and structural design. The two largest factors affecting durability of culverts are corrosion and abrasion. Proper attention must be given to these factors during design.

Acidic and alkaline conditions in soil and water, and the electrical conductivity of the soil contribute to corrosion of metal culvert pipe. Sulfates can cause rapid deterioration of concrete culverts. Abrasion is the erosion of culvert materials by sediment carried by streams. Plastic pipe is superior to corrugated metal and concrete pipe in corrosion resistance. Refer to the **Materials and Research Division** soil and situation report for soils information (See the Roadway Design Manual, Chapter Seven: Earthwork, Section 8.A.3, Reference 1.24).

8.O Multiple-Use Culverts

Culverts may often serve other purposes in addition to providing drainage. The cost advantages of multiple-use culverts should be evaluated against the possible advantages of separate facilities.

In situations where a culvert is installed to function as a cattle pass, the minimum size reinforced concrete box culverts will have a span of 5 ft. and a 7 ft. rise. The designer should be aware that a clear line of sight from beginning to end of the culvert must be provided or cattle will not enter the culvert. In addition, if the cattle pass is located in a drainage way, the hydraulic adequacy of the culvert must be verified. The design of all cattle passes shall give the contractor the option of furnishing a precast unit. Existing cattle passes should only be perpetuated if they are still in use. The designer shall consult with the **R.O.W. Design Division** to determine if there is a legal right to use the culvert as a cattle pass.

When a culvert is to be used as a dedicated pedestrian underpass, the minimum reinforced concrete box size shall be 12 ft. by 8 ft. for lengths of 60 ft. or less, with a recommended width of 14 ft. if the facility is shared with bicycles. If equestrian use is expected, the culvert size should be a minimum of 12 ft. by 12 ft. For culverts over 60 ft. in length, both dimensions should be increased. The designer shall consider the safety of the underpass users, (including location, sight distance, and lighting), and the drainage of the facility while designing the underpass. Bicycle/pedestrian underpass design shall be coordinated with the **Bicycle/Pedestrian Coordinator** in the **Project Development Division**. For additional information, see the Roadway Design Manual, Chapter Sixteen: Pedestrian and Bicycle Facilities, Section 5, (Reference 1.24).

8.P Culvert Extensions

Culverts to be extended should be in good condition. They should be extended with like material, (See Appendix C, "Pipe Material Policy"). Concrete pipe culverts shall be extended using a concrete collar to join old and new sections (See EXHIBIT 1.23). Elbows can be used for a change in direction of 3° or more where both pieces are new pipe. A contractor can use a collar for changing concrete pipe direction to angles not commonly used for elbows as shown on Standard Plan Number 4250 in the Standard/Special Plans Book, (Reference 1.8). Corrugated metal pipe culverts shall be extended using a connecting band (See EXHIBIT 1.23). Vitrified clay or cast iron pipe may be extended by use of a concrete collar. The **Bridge Division** shall be contacted for a recommendation before extending a box culvert with a span greater than 20 ft.

Roadway designers should follow these procedures when extending existing culverts:

1. Check the as-built culvert books and/or as-built plans for the existing culvert size. If the culvert size doesn't match the size given on the survey, the designer should check with the **District** to determine the correct size.
2. Check the stream meander for the determination of skew angle of the extended portion. If the survey does not show meanders for a culvert, look for it at the plan-in-hand inspection and request an additional survey if necessary.
3. Identify the causes of siltation and design accordingly. Consider the following options:
   a. Remove (or plug and abandon, if proper) the silted culvert and build a new culvert on the existing natural flowline. This option is available when there is enough clearance available above the culvert for a properly sized culvert.
   b. Keep the existing culvert flowline and extend the culvert without any cleanout efforts. This option is usually possible when siltation is minor, i.e., silt depth is less than 25% of culvert rise or diameter. Occasionally culverts are designed to have buried flowline.
   c. Design the inlet elevation to match the existing (upstream) flowline and do a channel cleanout on the outlet (downstream) end. Also decide what to do with the silt inside the culvert.
4. Use channel cleanout in lieu of small channel changes. If the culvert is silted, channel cleanout may not help. If right-of-way is not sufficient, a temporary easement is required.
5. Make sure that the limits of construction for culvert extensions are shown on the limits of construction plans and be certain there is sufficient room between the end of the culvert and the state right-of-way line.
6. If the embankment over a culvert is increased to or beyond the maximum fill heights as given in Appendix C, "Pipe Material Policy", before extending the culvert request that the **Bridge Division** perform a structural study and verify the structural adequacy of the existing culvert and bedding.

Unless a problem is noted at plan-in-hand or by the **District**, hydraulic analysis is not required for culvert extensions on 3R projects.

8.Q Structural Requirements

An embankment exerts more load on the foundation at the center of the embankment than at the toe of the slope, so more settlement will occur in the center area. A corresponding settlement of the culvert will occur, therefore the bedding profile should be cambered. Drainage structures constructed in fill sections with an anticipated settlement greater than 6 in. (0.2 m) will need to be cambered. The height of camber required is based on the settlement and length of the structure. The method of calculating this can be found in Standard Plan Number ___ in the Standard/Special Plans Book, (Reference 1.8). Control joints will be used in concrete box culverts with fills over 10 feet when settlement is anticipated (See Reference 1.8, Standard Plan 404). In an area where the embankment approaches 20 ft. or more, consult the **Soils Engineer** in the **Materials and Tests Division** for recommendations.

8.R Culvert Excavation Measurement

Quantities for required culvert excavation shall be computed for culverts in accordance with the following criteria:

- For pipe culverts and pipe arches, the width of the culvert trench shall be equal to the nominal inside diameter of the pipe or the maximum nominal inside clear span of the pipe arch, plus 36 in. The length of excavation shall extend for the length of the pipe plus 18 in. at each end, and will not be stepped at the culvert ends.
- When bedding material is not specified, the flow line of the culvert shall be the bottom limit of culvert excavation.
- When bedding material is specified, the additional depth and width shall be measured as culvert excavation.
- For box culverts, excavation shall be measured 18 in. outside of the neat lines of the concrete to the bottom of the box floor or footings. On box curtain walls below the bottom of the floor, and the footing beneath the lower break of broken-back boxes, excavation shall be measured as the neat lines of the concrete curtain wall or footing.
- When an existing box culvert is entirely removed on a project, the pay item is "Remove Structure". Excavation is not paid for in this situation.
- When an existing box culvert is replaced by a larger box culvert, (e.g., replacing a 4 x 4 CBC with a 6 x 6 CBC), the excavation quantity for the new box culvert shall be reduced by the volume of the opening of the existing box culvert if the opening has an average cross sectional area of 16 sq. ft. or more within the limits of the culvert excavation.
- When an existing bridge structure is replaced with a concrete box culvert, earthwork will be a shared responsibility of the culvert contractor and the grading contractor. See the Roadway Design Manual (Ref. 1.24), Chapter Seven: Earthwork, Section 3.A for the demarcation between the culvert and the grading contractor's responsibilities.
- For headwalls, excavation shall be measured 18 in. outside the neat lines of the concrete and to the bottom of the headwall.
- In deep cut sections, the upper limits of culvert excavation shall be the top of the roadway prism. It is assumed that roadway excavation shall be completed before culvert excavation is performed.
- In fill sections, the upper limit of culvert excavation shall be the original ground line.
- When placing multiple culverts, the minimum distance between the culverts (inside wall to inside wall) shall be 1 ft., with flowable fill used as backfill material. When indigenous soil is used as backfill material, this minimum distance increases to 5 ft. Special FES or headwall is required for 1 ft. culvert spacings.

EXHIBIT 1.34 illustrates the separation of excavation for pipe culverts and headwalls, and excavation for box culverts. EXHIBIT 1.35 illustrates the measurement of additional horizontal allowance for culvert excavation for broken-back box culverts.

When excavation depths are greater than 4 ft., additional quantities of culvert excavation shall be provided. EXHIBIT 1.36 provides the excavation depth ranges and additional horizontal allowances.

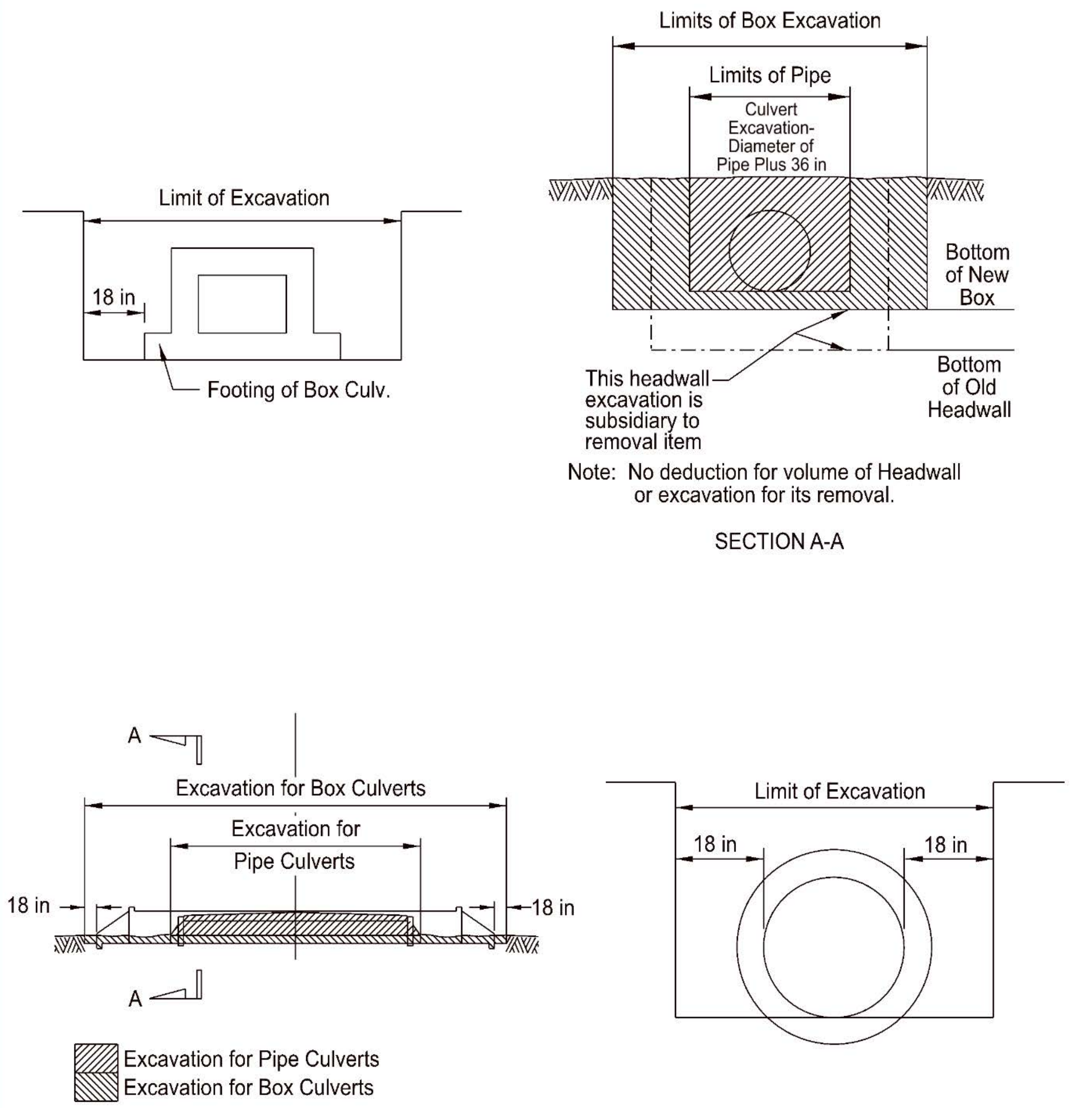


Exhibit 1.34 Separation of Excavation for Pipe Culverts and Headwalls, and Excavation for Box Culverts

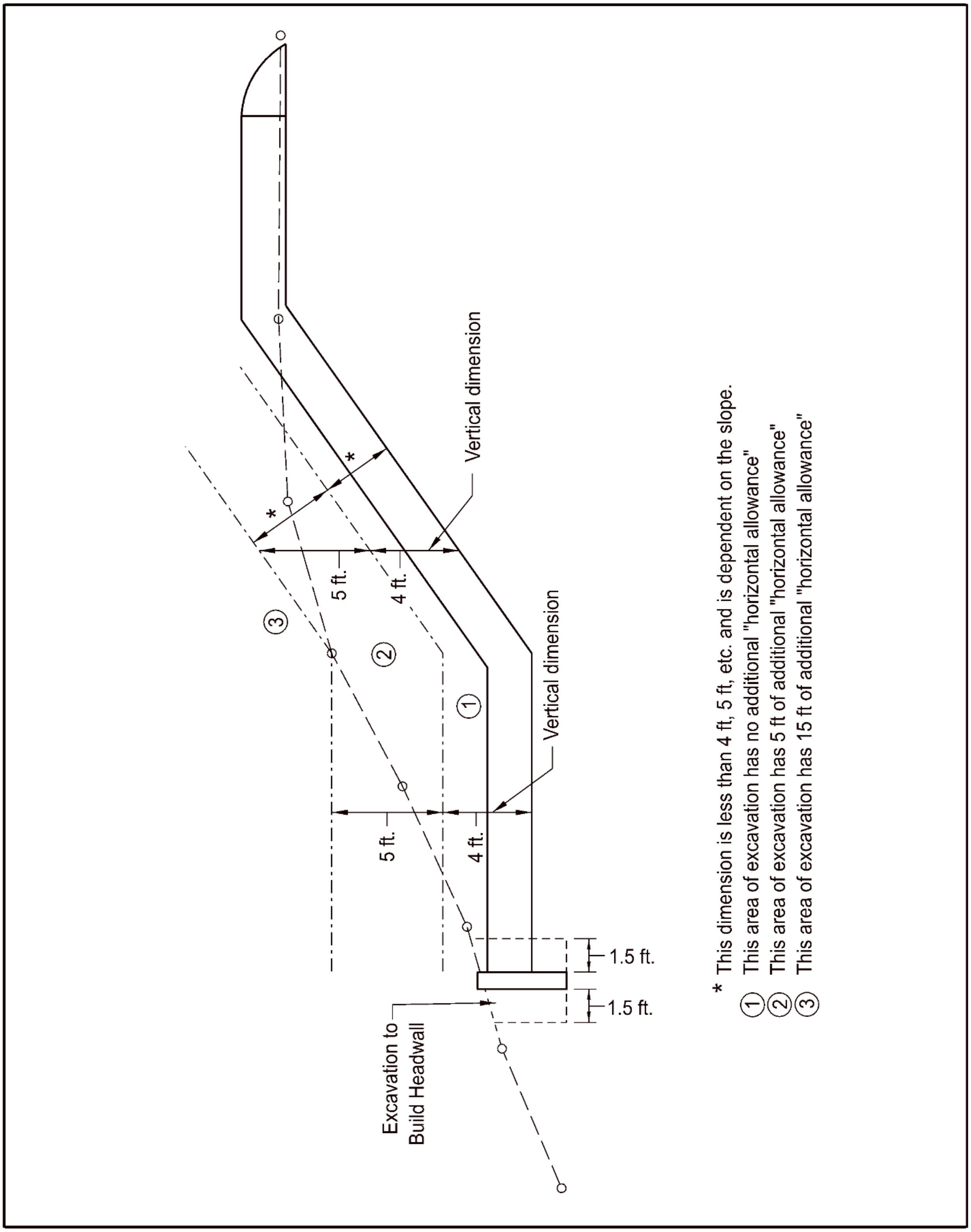


Exhibit 1.35 Excavation for Broken-Back Culverts

Excavation Depth Range (ft)	Additional Horizontal Allowance (ft)
4 to 9	5
9 to 14	15
14 to 19	25
Over 19	35

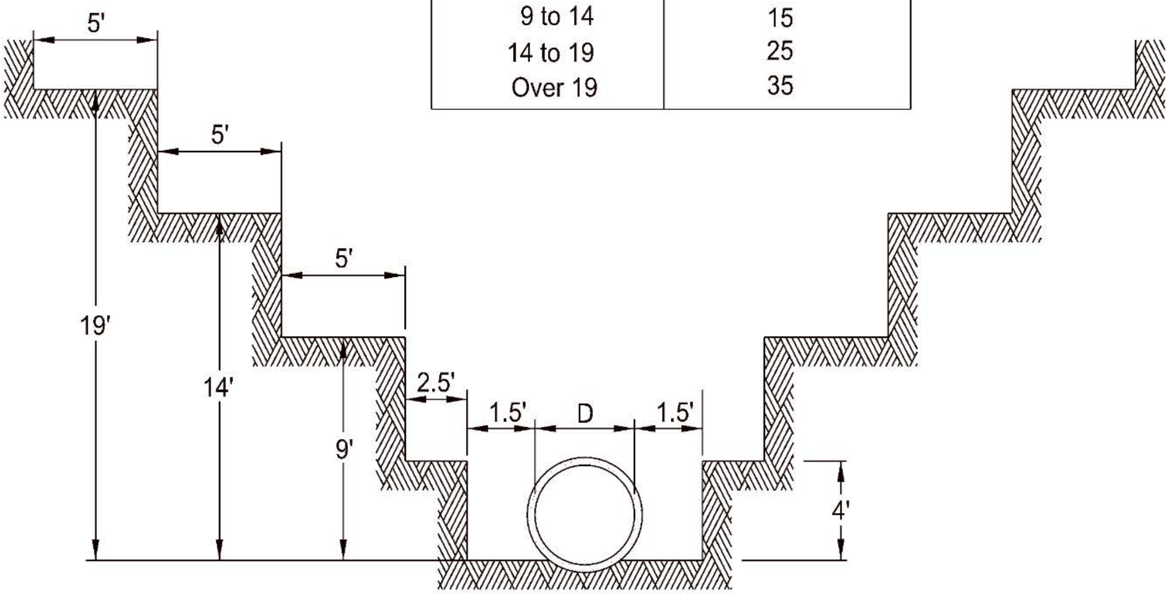


Exhibit 1.36 Culvert Excavation

9. PAVEMENT DRAINAGE

Runoff from large areas draining toward the highway pavement should be intercepted prior to reaching the roadway. Curbed pavement sections of highways and pavement drainage inlets are inefficient means for handling large amounts of runoff and extraneous drainage should be intercepted before it reaches the highway pavement. This applies to drainage from residential neighborhoods, commercial or industrial property, long cut slopes, side streets, and other areas along the pavement. If extraneous drainage cannot be intercepted prior to reaching the highway, it should be included in the pavement drainage design.

Pavement drainage is comprised of two distinct elements. The first involves sheet flow across the pavement surface. The second element or mode occurs where curbs contain and channel runoff within the roadway gutter and a portion of the pavement until the stormwater can be removed from the surface through inlets.

Hydraulic design procedures described in this section are based upon information in Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements, (Reference 1.2). The following sections discuss surface drainage of pavements and curb and gutter flow. Storm sewer inlets are discussed in Section 10.B.

9.A Surface Drainage of Pavements

Stormwater runoff from pavements is dependent on the following combination of pavement conditions:

- Profile grade.
- Cross slope.
- Width.
- Surface texture.

Alignment and drainage design must be coordinated to ensure compatibility between roadway geometrics and drainage design. For general coordination of mainline alignment and drainage design, the following design considerations apply:

1. Superelevation: Horizontal and vertical alignment shall be coordinated so that roadway sections with minimal pavement cross slope (due to superelevation transition) do not coincide with flat segments on vertical curves, (See the Roadway Design Manual, Chapter Three: Roadway Alignment, Section 2.B, Reference 1.24).

2. Profile: The longitudinal gradient is an important consideration in the surface drainage of pavements. Flat gradients on curbed pavement can lead to excessive spreading of the stormwater runoff flowing along the curb. Longitudinal gradients can be a concern on both crest and sag vertical curves. Vertical curves should be designed to produce a longitudinal grade of at least 0.35% at a point approximately 50 ft. either side of the apex of a crest vertical curve or the low point of a sag vertical curve. This corresponds to a value of A = 0.7, for which K = 100/0.7 = 143 ft. per percent change in grade. Where conditions do not allow the desired 0.35% value, a minimum longitudinal grade of 0.20% may be used, corresponding to a K value of 250 (See the Roadway Design Manual, Chapter Three: Roadway Alignment, Section 3.B.2, Reference 1.24). Spread widths should be carefully evaluated at the apex of crest vertical curves and at the low point of sag vertical curves when the K value exceeds 143, (See Section 10.A). A minimum longitudinal gradient of

0.20% should be maintained on straight sections of the curbed roadway. For uncurbed sections of highways, drainage should not be a problem with properly crowned pavements.

3. Curb and Gutter Flow: When curb and gutters are placed on highways, (or are expected to be added to the highway in the future), they should have a minimum profile grade of at least 0.2% to allow for adequate drainage. This minimum grade may be created in very flat terrain by rolling the highway profile or by using a rolled gutter line. Rolling the gutter line may be accomplished by either using a separate gutter line profile or by providing pavement grades on the project 2L sheets (See the Roadway Design Manual, Chapter Eleven: Highway Plans Assembly, Section 4.I, Reference 1.24). The roadway designer should maintain the 2% minimum pavement cross slope by deepening the grade when rolling the gutter line and should avoid level stretches of gutter and low spots in the gutter line which do not have outlets. The use of valley gutters should generally be avoided but may be used at non-signalized intersections and at intersections with stop control on the minor leg(s). Drainage runoff should, generally, be intercepted on the upstream side of an intersection.

4. Bridge Decks: The designer should coordinate with the **Bridge Division** regarding drainage from bridge decks. The roadway designer should ask the **Bridge Division** if runoff from the bridge deck will be handled separately or if runoff from the deck will be discharged to the roadway gutter. If a closed bridge rail is being used, drainage from the bridge deck will be discharged into the roadway drainage system and is the concern of the roadway designer.

Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians. Side slopes and ditches should be designed to accommodate surface drainage in rural sections.

10. STORM SEWER SYSTEMS

A storm sewer is an underground pipe system that carries runoff collected from the roadway surface, discharging this runoff into a stream or river without treatment. Storm sewer design consists of three phases:

- Surface Drainage (surface flow and drainage inlets).
- The conveyance network (pipes and structures).
- The conveyance network capacity (the hydraulic gradeline).

The surface drainage phase of design determines the spread of water on the highway pavement; the locations, types, and capacities of the drainage inlets and the impacts of surface waters that are not collected by the drainage inlets. A significant aspect of this phase is the determination of the effects of inlet bypass, including major storm flows that exceed the design of the conveyance network. Consideration should be give to the flow path of surface waters that exceed and bypass the capacity of the drainage inlets in order to limit damage to adjacent property.

The design of the conveyance network determines the required capacity of the individual pipes and structures through the use of the Manning's Equation (See Section 10.D.1). Initial pipe and structure sizes and slopes are determined in this phase of design by assuming 100% capture of the surface flow by the drainage inlets and full pipe flow. Pipe and structure sizes shall remain the same or increase as they progress downstream.

The conveyance network capacity takes the initial pipe and structure sizes, derived in the design of the conveyance network, and applies head losses due to pipe friction, bends, junctions, and other network conditions to calculate the hydraulic gradeline. The pipe sizes and slopes are then adjusted to establish the capacity of the conveyance network and the hydraulic gradelines of the network (See Section 10.E).

Hydraulic design procedures described in this section are based on information found in Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements, (Reference 1.2), and Hydraulic Engineering Circular No. 22: Urban Drainage Design Manual, (Reference 1.26). See Appendix D, "Storm Sewer Policy", for additional information.

10.A Storm Sewer Curb and Gutter Flow

The curb and gutter form a triangular channel that can efficiently carry rainfall runoff to collection points, such as curb inlets. However, when a design storm event occurs, the width of the runoff spread may include not only the gutter width but also parking lanes, shoulders, and/or portions of the traffic lanes. This spread width is what should concern the roadway designer the most about curb and gutter flow.

In urban sections with curb and gutter, it is preferable to intercept runoff from areas draining toward the roadway prior to the flow reaching the roadway. This may be accomplished via ditches, area inlets, etc. At appropriate intervals, water that collects along the curb may be channeled into curb inlets, grate inlets, flumes, or carried into side ditches or pipe drains. These structures should be provided at strategic locations along the gutter to limit the spread of water across the travel lanes. See EXHIBIT 1.37.

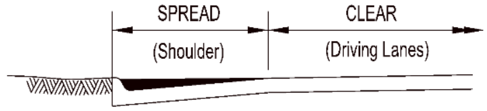
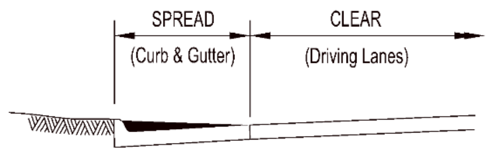
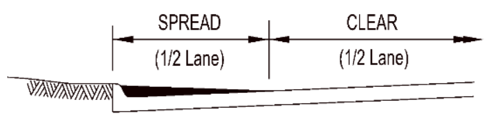
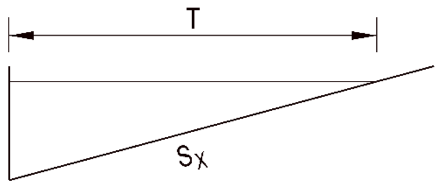
LOCATION	LIMITS OF SPREAD	ALLOWABLE SPREAD	DESIGN Q
HIGH-SPEED ROADWAYS (OVER 45 MPH) WITH/SHOULDER		LOWEST EDGE OF DRIVING LANES	Q_{50}
HIGH-SPEED ROADWAYS (OVER 45 MPH) WITHOUT SHOULDER		LOWEST EDGE OF DRIVING LANES (CREATE CURB & GUTTER OUTSIDE OF DRIVING LANE)	Q_{50}
LOW-SPEED ROADWAY (45 MPH AND UNDER) WITHOUT SHOULDER		½ LANE IN EACH DIRECTION	Q_{10}

Exhibit 1.37 Maximum Allowable Spread Width

10.A.1 Gutter Flow Equations

Chart 3 of Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements, (Reference 1.2), may be used to determine the flow in triangular gutter sections.

The following modified forms of Manning's equation should be used to evaluate gutter flow hydraulics for 6 in. curbs:



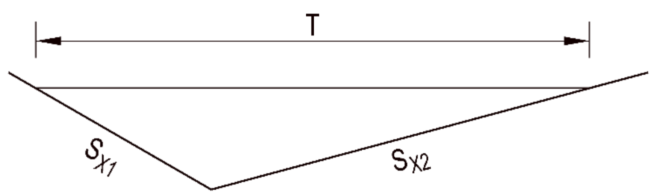
$Q = 0.56(z/n)S^{1/2}d^{8/3}$ (Use when depth (d) is known). Eq. 1.7a

or

$Q = (0.56/n)\ S_x^{5/3}S^{1/2}T^{8/3}$ (Use when width of spread (T) is known). Eq. 1.7b

where:
- Q = Gutter flow rate, cfs;
- z = Reciprocal of pavement cross slope = $1/S_x$;
- n = Manning's roughness coefficient (Appendix B, "Manning's Coefficient, n");
- S_x = Pavement cross slope, ft./ft.;
- S = Longitudinal slope, ft./ft.;
- d = Depth of flow at the curb, ft.;
- T = Width of flow or spread, ft.

For 4 in. lip curbs, S_X is determined by:



$S_X = S_{X1}S_{X2} / (S_{X1} + S_{X2})$ Eq. 1.8

This modified Manning's equation, Eq. 1.7, shall be used when the gutter cross slope(s) is less than 10%. Manning's equation (unmodified) shall be used when gutter cross slope(s) are equal to or greater than 10%. A Manning's roughness coefficient (n) of 0.016, (See Appendix B, "Manning's Coefficient, n"), shall be used for concrete gutter and 0.015 for asphalt gutter in Equation 1.7.

Equation 1.7 can also be solved using the nomograph presented in EXHIBIT G.2 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design". Section 14.D demonstrates the use of Eq. 1.7 in determining the width of spread. EXHIBIT G.1 provides instructions for computing flows in triangular gutter sections, shallow V-shaped channels and composite gutter sections.

10.A.2 Gutter Slopes

A minimum longitudinal slope is important for a curbed pavement since it is an important factor in stormwater spread. Desirable gutter slopes should be 0.35% or greater for curbed pavements with a minimum 0.20% in very flat terrain. Minimum grades may be created in very flat terrain by rolling the highway profile or by using a rolled gutter line. Rolling the gutter line may be accomplished by either using a separate gutter line profile or by providing pavement grades on the project 2L sheets (See the Roadway Design Manual, Chapter Eleven: Highway Plans Assembly, Section 4.I, Reference 1.24. The roadway designer should maintain a minimum 2% pavement cross slope by increasing the pavement cross-slope when rolling the gutter line and should avoid level stretches of gutter and low spots in the gutter which do not have outlets. Depressed gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement.

10.B Storm Sewer Inlets

The following section discusses the location of inlets, type, materials and structural design. The location and spacing of inlets depends primarily on the following factors:

- Amount of runoff.
- Longitudinal slope.
- Pavement cross slope.
- Location and geometrics of intersections and driveways.
- Inlet capacity.
- Allowable spread width (See EXHIBIT 1.37).
- Underground utilities.

Inlets used for the drainage of highway pavements can be divided into four major types. These types are:

1. Curb inlets: vertical openings in the curb covered by a top slab (EXHIBIT 1.38a).
2. Grate inlets: openings in the gutter covered by one or more grates (EXHIBIT 1.38b).
3. Slotted pipe inlets: slotted openings with bars perpendicular to the opening used on multilane facilities or at gore areas. (Slotted inlets function similarly to curb opening inlets, i.e., as weirs with flow entering from the side, EXHIBIT 1.38c).
4. Slotted vane inlets: slotted vane and vane grates used in combination (EXHIBIT 1.38d).

The capacity of an inlet depends upon the following:

- Inlet geometry.
- Cross slope.
- Longitudinal slope.
- Depth of slope.
- Total gutter flow.
- Pavement roughness.

Place flanking inlets on each side of the inlet at the low point in the sag where significant ponding can occur (e.g., in underpasses and in sag vertical curves in depressed sections). Flanking inlets intercept the flow prior to the flat point of the vertical curve to reduce water spread. A procedure for locating the flanking inlets may be found in Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements, (Reference 1.2), page 89.

The depth of water next to the curb is the major factor in the interception capacity of both curb-opening inlets and grate inlets. The quantity of stormwater that bypasses an inlet is carried over and must be accounted for and included in the design of the next downstream inlet. Clogging of the inlet should also be accounted for in the design. Design is generally based on a clean inlet with reduction factors applied for clogging.

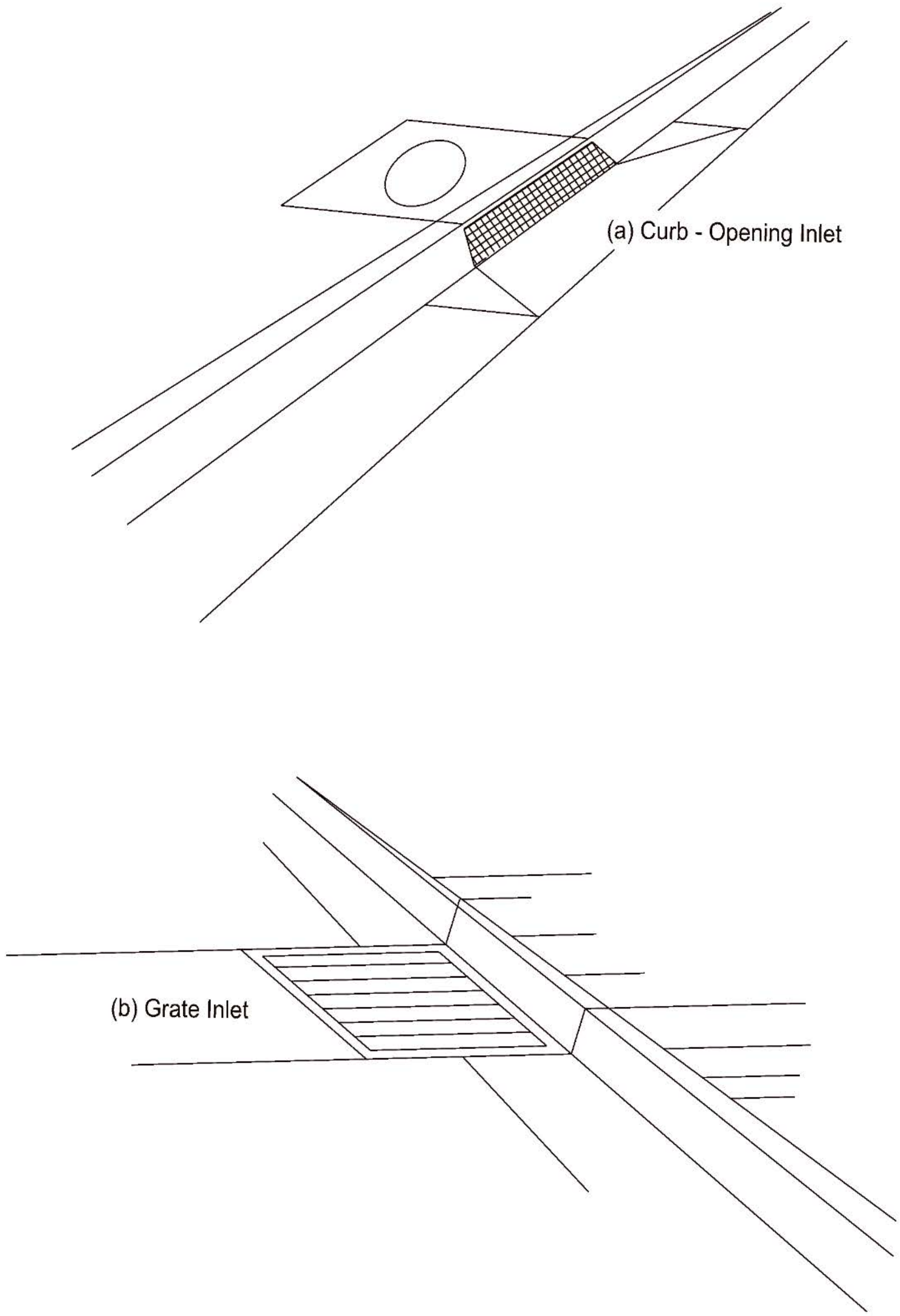
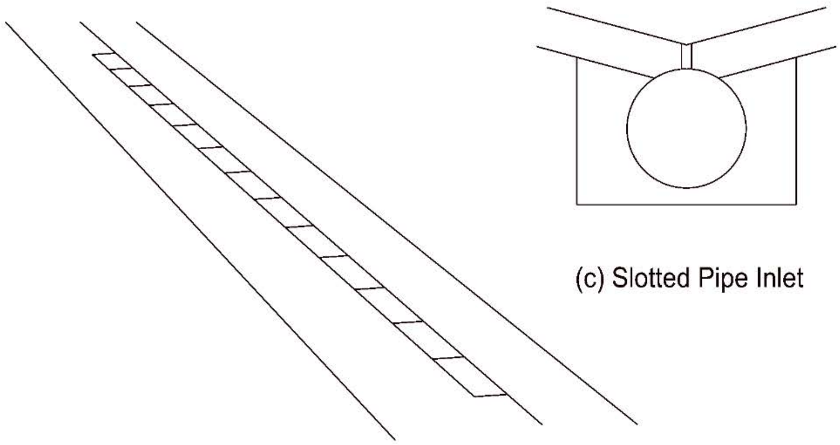
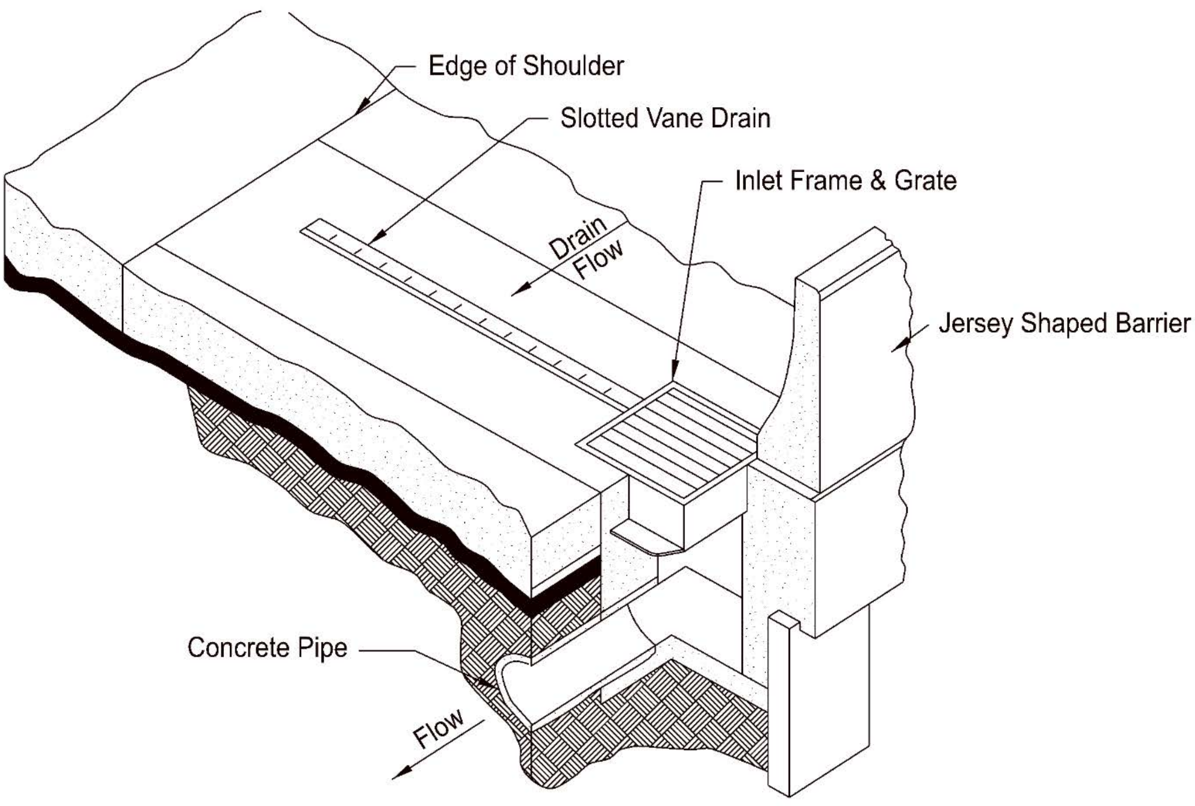


Exhibit 1.38 Types of Inlets



(c) Slotted Pipe Inlet



(d) Slotted Vane Inlet

Exhibit 1.38 (Cont.) Types of Inlets

10.B.1 Inlet Placement

Drainage inlet locations are often established by the roadway geometry, as well as by the need to reduce the spread of water on the roadway surface. When designing inlet spacing, the roadway designer should begin at the high point (the crest of the hill) and work downwards. In general, inlets should be placed:

- At low points in the gutter grade.
- Upgrade of intersections, median breaks, and pedestrian crosswalks.
- On side streets where drainage would flow onto the highway pavement.
- Where pavement surfaces are warped, such as at cross slope reversals and ramps.
- Upgrade of bridges.
- Downgrade of bridges.
- When gutter flow exceeds allowable spread widths.

Inlets are placed at low points in the gutter grade to keep water from ponding, which can damage the pavement and cause icing in the winter. Placement of inlets upgrade of intersections, median breaks, crosswalks, and where pavement surfaces are warped to keep water from flowing across the intersection and highway pavement.

Bridge deck drainage is frequently constrained by structural considerations. Inlets should be placed upgrade of a bridge to intercept water before it flows onto the bridge deck. The roadway designer should check with the **Bridge Division** to determine if a bridge structure has open or closed bridge railing. If a bridge has closed bridge railing, the drainage of the bridge deck is the responsibility of the roadway designer and inlets should be placed downgrade of the bridge to intercept the water flow.

In the absence of other design considerations, inlets spacing will be designed based on the allowable spread width (See EXHIBIT 1.37). For continuous grades the placement of inlets will be fairly uniform. In sag locations flanking inlets should be placed. Flanking inlets are generally placed upgrade and to both sides of the inlet at the bottom of the sag location. These inlets intercept stormwater runoff prior to the low point of the sag in order to control the approach spread. The approach spread is a gutter flow calculation that is computed for the flow that approaches from either side of a sag inlet; frequently the approach spread will exceed the spread width of the combined flows at the sump. A minimum grade of 0.20% is used to determine the spread width. The flanking inlets can also act as relief inlets to control the total depth of ponding in cases where the sag inlet becomes clogged or where a storm event exceeds the design storm. This is of concern in sag locations where significant ponding can occur and water cannot escape except through the curb inlets, such as underpasses and depressed roadways.

10.B.2 Curb Inlets

Curb inlets are the preferred installation when 6 in. curbs are used. Curb inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb inlets are relatively free of clogging tendencies and offer little interference to traffic operation.

EXHIBIT G.5 in Appendix G, “Nomographs and Charts for Gutter Flow & Inlet Design”, contains performance curves for the curb inlets detailed on Standard Plan No. 443 in the Standard/Special Plans Book (Reference 1.8). The designer should avoid placing curb inlets on intersection radii.

They are difficult to construct and maintain. Designers also should avoid using curb inlets with a Y dimension of 6 ft. due to the difficulty in constructing the trough on this inlet.

10.B.2.a Capacity of Curb Inlets on Continuous Grade

The capacity charts in EXHIBIT G.3 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design", are for curb inlets on continuous grades. The capacity of the inlet depends upon the length of opening and depth of flow at the opening. This depth in turn depends upon the amount of depression of the gutter at the inlet, the cross slope, longitudinal slope and roughness of the gutter.

To determine the curb inlet capacity using EXHIBIT G.3 the following information is needed:

- L = Length of inlet throat opening;
- a = Depth of gutter depression, if any, at the inlet (the gutter depression is 5 in. for Standard Plan 443, Standard/Special Plans Book, Reference 1.8);
- Q_a = Design discharge in the gutter or information as to drainage area, rainfall intensity, and runoff coefficients from which a design discharge can be estimated (any carryover from a previous inlet must be included);
- d = Depth of flow in normal gutter for the particular longitudinal and cross slopes above the inlet in question which may be determined from EXHIBIT G.2.

The procedure is as follows:

1. Enter Chart A in EXHIBIT G.3 with depth of flow, d, and gutter depression at the inlet, a, and determine Q_a/L_a, the interception per ft. of inlet opening if the inlet were intercepting 100% of the gutter flow.

2. Determine the length of inlet which is required to intercept 100% of the gutter flow, L_a:

$$L_a = Q_a/(Q_a/L_a) \qquad \text{Eq. 1.9}$$

where:
L_a = Length of inlet required for 100% interception, ft.;
Q_a = Total gutter flow, cfs.

3. Compute the ratio, L/L_a where L equals the actual length of the inlet in question.

4. Enter Chart B in EXHIBIT G.3 with L/L_a and a/d. Determine the ratio, Q/Q_a, the proportion of the total gutter flow intercepted by the inlet in question.

5. Flow intercepted, Q, is the ratio, Q/Q_a, times the total gutter flow, Q_a.

6. Flow carried over to the next inlet equals Q_a minus Q.

10.B.2.b Capacity of Curb Inlets in a Low Point or Sump

The nomograph in EXHIBIT G.4 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design", for the capacity of curb inlets in a low point or sump is based on the following conditions:

- The curb inlet is located at a low point in the grade.
- All flow coming into the inlet must eventually enter the inlet and will pond until sufficient head is built up so that the outflow through the inlet will equal the peak inflow from the gutters.

In EXHIBIT G.4 the following values are shown:

h = Total height of opening, ft. (height of opening is 5 in. for the Standard Plan);
L = Total length of opening, ft.;
H = Depth of water at the entrance, ft.;
Q = Total peak rate of flow to the inlet, cfs.

To use EXHIBIT G.4 enter the nomograph with any two of the three values, h, Q/L, or H/h. Then read the third value. Normally, Q, L and h are known, and the nomograph is used to determine the depth of water, H, at the inlet. The spread of the water on the street will depend upon the cross slope of the pavement. EXHIBIT G.4 is based on orifice and weir hydraulics. For further information, see Hydraulic Engineering Circular No. 22: Urban Drainage Design Manual, (Reference 1.26).

10.B.3 Grate Inlets

Generally, at low velocities all of the water flowing in the gutter section occupied by the grate, called frontal flow, is intercepted by grate inlets. A small portion of the flow along the length of the grate, termed side flow, is intercepted as well. On steep slopes where velocity is high and splashover occurs, only a portion of the frontal flow may be intercepted. Splashover can also occur if the grate is short. Curved vane grates are hydraulically more efficient. For grates less than 2 ft. long, intercepted flow is small.

Inlet interception capacity has been investigated by various agencies and grate manufacturers. For inlet efficiency data for various sizes and shapes of grates, see Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements, (Reference 1.2), and also inlet grate capacity charts prepared by grate manufacturers.

Grate inlets subject to bicycle traffic should be bicycle safe. Grate inlets subject to vehicular traffic should be rated to the anticipated load. Appropriate frames should be provided. A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. In some locations where leaves may create constant maintenance problems, the parallel bar grate may be used more efficiently if bicycle traffic is prohibited.

10.B.3.a Capacity of Grate Inlets On Continuous Grade

The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is expressed by the following equation:

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.7} \qquad \text{Eq. 1.10}$$

where:
- E_o = Ratio of frontal flow to total gutter flow;
- Q = Total gutter flow, cfs.;
- Q_w = Flow in width W, cfs.;
- W = Width of depressed gutter or grate, ft.;
- T = Total spread of water in the gutter, ft.

EXHIBIT G.6 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design", provides a graphical solution of E_o for either straight or cross slopes or depressed gutter sections.

The ratio of side flow to total gutter flow, Q_s, is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \qquad \text{Eq. 1.11}$$

where: Q_s = ratio of side flow to total gutter flow and the rest as defined in Eq. 1.10.

The ratio of frontal flow intercepted to total frontal flow, R_f (frontal flow interception efficiency), is expressed by the following equation:

$$R_f = 1 - 0.09\,(V - V_o) \qquad \text{Eq. 1.12}$$

where:
- R_f = Ratio of frontal flow intercepted to total frontal flow;
- V = Velocity of flow in the gutter, ft./sec.;
- V_o = Gutter velocity where splashover first occurs and not all frontal flow is intercepted, (Assumed V_o = 6 ft. per sec.)

Equation 1.12 can be solved using the nomograph presented in EXHIBIT G.7, which takes into account grate length, bar configuration and gutter velocity at which splashover occurs. The gutter velocity needed to use EXHIBIT G.7 is total gutter flow divided by the area of flow.

The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \qquad \text{Eq. 1.13}$$

where:
- R_s = Ratio of side flow intercepted to total side flow;
- V = Velocity of flow in gutter, ft./sec.;
- S_X = Pavement cross slope, ft./ft.;
- L = Length of the grate, ft.

Equation 1.10 can be solved using the nomograph presented in EXHIBIT G.8.

The efficiency of a grate, E, is expressed as:

$$E = R_f E_o + R_s(1-E_o) \quad \text{Eq. 1.14}$$

where:
- E = Efficiency of a grate inlet;
- R_f = Ratio of frontal flow intercepted to total frontal flow;
- E_o = Ratio of frontal flow to total gutter flow;
- R_s = Ratio of side flow intercepted to total side flow.

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)] \quad \text{Eq. 1.15}$$

where:
- Q_i = Interception capacity of an inlet;
- E = Efficiency of an inlet;
- Q = Total gutter flow, cfs (Eq. 1.11).

The solution of Eq. 1.15 is provided in Section 14.G.

10.B.3.b Capacity of Grate Inlets In a Low Point or Sump

A grate inlet in a sag operates as a weir up to a certain depth dependent on the bar configuration and size of the grate, and as an orifice at greater depths. For standard gutter inlet grates, weir operation continues to a depth of about 0.4 ft. above the top of grate. When the depth of water exceeds approximately 1.4 ft., the grate begins to operate as an orifice. Between depths of approximately 0.4 ft. and about 1.4 ft., a transition from weir to orifice flow occurs.

Weir Condition

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

$$Q_i = CPd^{1.5} \qquad \text{Eq. 1.16}$$

where:
- Q_i = Interception capacity of an inlet;
- C = Coefficient equal to 3.0;
- P = Perimeter of grate excluding the side against the curb, ft.;
- d = Depth of water above the grate, ft.

The solution of Eq. 1.16 is provided in Section 14.H.1.

Orifice Condition

The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA(2gd)^{0.5} \qquad \text{Eq. 1.17}$$

where:
- Q_i = Interception capacity of an inlet;
- C = Orifice coefficient equal to 0.67;
- A = Clear opening area of the grate, ft^2.;
- g = 32.2 ft./s^2;
- d = Depth of water above top of the grate, ft.

Equations 1.16 and 1.17 can be solved using the nomograph presented in EXHIBIT G.9 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design".

The effects of grate size on the depth at which a grate operates as an orifice is apparent from the nomograph. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or orifice equation. This capacity can be approximated by drawing a curve between the lines representing the perimeter and net area of the grate to be used. Refer to Section 14.H.2 for an example problem.

10.B.4 Slotted Pipe Inlets

Slotted pipe inlets (See EXHIBIT 1.38c) have a variety of applications. Slotted pipe inlets can be used on curbed or uncurbed sections and offer little interference to traffic operations. Since debris deposition in the pipe is a common problem, cleanout openings at both ends of the sewer pipe are usually provided.

10.B.4.a Capacity of Slotted Pipe Inlets On Continuous Grade

Flow interception by slotted pipe inlets and curb opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. The length of a slotted pipe inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = 0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6} \qquad \text{Eq. 1.18}$$

where:
- L_T = Length of slotted pipe inlet required to intercept 100% of the gutter flow;
- Q = Gutter flow rate, cfs;
- S = Longitudinal slope, ft./ft.;
- n = Manning's roughness coefficient (Appendix B, "Manning's Coefficient, n");
- S_X = Pavement cross slope, ft./ft.;

The efficiency of slotted pipe inlets shorter than the length required for total interception is expressed by:

$$E = 1 - (1 - L/L_T)^{1.8} \qquad \text{Eq. 1.19}$$

where:
- E = Interception efficiency of slotted pipe inlet;
- L = Length of slotted pipe inlet, ft.;
- L_T = Length of slotted pipe inlet required to intercept 100% of the gutter flow.

Equations 1.18 and 1.19 can be solved by using the nomographs presented in EXHIBITS G.10 AND G.11, respectively, in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design".

The actual gutter flow intercepted can be found using the following equation:

$$Q_i = EQ \qquad \text{Eq. 1.20}$$

where:
- Q_i = Gutter flow intercepted, cfs;
- E = Interception efficiency of slotted pipe inlet;
- Q = Gutter flow rate, cfs

10.B.4.b Capacity of Slotted Pipe Inlets in a Low Point or Sump

Slotted pipe inlets in sag locations perform as weirs to depths of about 0.2 ft., dependent on slot width and length. At depths greater than about 0.4 ft., they perform as orifices. Between these depths, flow is in a transition stage.

Weir Condition

EXHIBIT G.12 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design", provides a nomograph for solutions for weir flow and a plot representing data at depths between weir and orifice flow. For an example problem, see Section 14.I.1.

Orifice Condition

The interception capacity of a slotted pipe inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8LW(2gd)^{0.5} \quad \text{Eq. 1.21}$$

where:
- Q_i = Interception capacity of an inlet;
- L = Length of slot, ft.;
- W = Width of slot, ft.;
- g = 32.2 ft./s^2;
- d = Depth of water at slot, ft.

For a slot width of 1.75 in., the above equation becomes:

$$Q_i = 0.94Ld^{0.5} \quad \text{Eq. 1.22}$$

The interception capacity of slotted pipe inlets at depths between 0.2 ft. and 0.4 ft. can be computed by the use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted pipe inlet. See Section 14.I.2 for an example problem.

10.B.5 Slotted Vane Inlet

The slotted vane inlet indicated in EXHIBIT 1.38d is a slotted vane drain in conjunction with a curved vane grate.

Curved vane grates are hydraulically more efficient than conventional grates. Slotted vane drains also offer the advantage of increased water capture efficiency when compared to slotted inlets. When the above are used in combination, as shown in EXHIBIT 1.38d nearly all of the side flow is captured, allowing increased spacing of inlets.

Refer to Section 10.B.3 for hydraulic design procedures for determining the capacity of curved vane grates.

The capacity of a slotted vane drain can be determined from the following equation developed by Neenah Foundry Company:

$$Q = Kd^{5/3} \quad \text{Eq. 1.23}$$

where:
- Q = Interception capacity of slotted vane drain, cfs;
- K = Slotted vane drain coefficient;
- d = Depth of flow in gutter upstream of drain, ft.

Values of K can be obtained from EXHIBIT G.13 in Appendix G, “Nomographs and Charts for Gutter Flow & Inlet Design”.

The designer should note that EXHIBIT G.13 has been specifically developed by Neenah Foundry Company for Slotted Vane Drain R-3599. *K values indicated in EXHIBIT G.13 are not applicable to other slotted vane drains.*

10.B.6 Multiple Grate Inlets

Multiple inlets are made up of two or more inlets, immediately adjacent to each other, acting as a single unit. They may be constructed in either a longitudinal or transverse direction, depending on the roadway situation. Multiple inlets are used to both increase the interception of the gutter flow and to reduce the impact of debris clogging. The increase in the interception of gutter flow depends on both the location of the inlet (sag or continuous grade) and the direction of the inlet extension.

On a continuous grade the placement of multiple grates transverse to the flow greatly increases the inlet's interception by spanning a greater amount of the gutter spread and increasing the frontal interception efficiency. Placing the grates transverse to the flow also reduces the impact of debris clogging, which primarily occurs next to the curb. Placing the grates longitudinally on a continuous grade increases the inlet's interception, somewhat, by increasing the side flow interception efficiency and capturing any potential splashover water.

In a sag condition, the placement of multiple grates longitudinally to the flow provides better interception with reduced spreads, limits the impact of debris clogging, and increases the length of curb line covered by the inlet. The interception rate of a longitudinal grate is an improvement over a lateral grate since placement adjacent to the curb produces a greater depth of water, which is uniform across the inlet, compared to a laterally placed grate that has a decreasing water depth as it extends away from the curb. The increased weir length and open area of ~~a~~ multiple grates reduce the overall percentage of the inlet covered by debris, thus reducing its impact. By placing the grates longitudinally the likelihood that the inlet covers the actual low point of the sag increases, reducing the possibility that water will continue to pond after the storm.

10.C Storm Sewer Manholes and Junction Boxes

Manholes and junction boxes are utilized to provide access to storm drains for inspection and cleanout and are used for changing direction or convergence. The Standard/Special Plans Book, (Reference 1.8), may be consulted for details of Manholes and Junction Boxes commonly used by **NDOT**.

10.C.1 Location

Manholes or junction boxes are typically installed at the following locations:

- Convergence of two or more storm sewers.
- Intermediate points along tangent sections.
- Change in pipe size.
- Change in pipe alignment.
- Change in pipe grade.

The maximum spacing of manholes and junction boxes should be 300 ft. for pipes less than or equal to 48 in., (junction boxes shall not be located in the roadway).

Manholes shall be used if a structure must be located in the roadway. When it is impossible to avoid locating a manhole in a traffic lane, care should be taken to avoid placing the manhole in the normal vehicle path. Where feasible, curb or area inlets should be used in lieu of manholes, allowing access to the system at the inlet while providing the benefit of extra stormwater interception with minimal additional cost.

10.D Storm Sewer Pipe

Hydraulic design of storm sewers shall be in accordance with the following criteria:

- All storm sewer pipe shall have smooth interior walls.
- Storm sewers shall be designed using a Manning's n value of 0.012.
- Appendix B, "Manning's Coefficient, n", lists values for storm sewer pipe materials and should be used for evaluating the hydraulic capacity of existing systems.
- Storm sewers should be designed for open channel flow and not as pressure conduits. Storm sewers may operate under pressure conditions provided certain hydraulic grade line criteria are observed. Refer to Section 10.E, Hydraulic Grade Line, for a discussion of the above criteria.
- Storm sewers should be designed to flow full at the design runoff. Minimum and maximum velocities of flow should be 2 ft./sec. and 10 ft./sec., respectively. In order to prevent silt accumulation, a velocity of 3 ft./sec. should be maintained in the storm sewer.
- The minimum pipe size for storm sewers is 15 in. for transverse pipe and 18 in. for longitudinal pipe.
- Storm sewers should be constructed on a straight (tangent) alignment between manholes. Storm sewers should not be constructed on curves unless extenuating circumstances warrant construction on a curved alignment. The designer must obtain approval from the **Roadway Design Unit Head** to construct a storm sewer on a curved alignment.
- Utilities and deep cuts should be avoided.
- The storm sewer trunk line should not be located beneath the traveled way. The placement of the storm sewer trunk line, in descending order of preference, is behind the curb, in the shoulder, or in the roadway median.
- Storm sewers should be laid a minimum of 10 ft. horizontally (plan view) from any existing or proposed water main (measured edge to edge). In cases where it is not practical to maintain a 10 ft. separation, the **Nebr. Dept. of Health** may allow installation of the sewer closer to the water main, provided that the sewer is laid in a separate trench or on an undisturbed earth shelf located on one side of the water main or at such an elevation that the bottom of the sewer is at least 18 in. above the top of the water main.
- When crossing a water main, the edge of the storm sewer shall be a minimum vertical distance of 18 in. from the outside edge of the water main. This shall be the case whether the sewer is above or below the water main. At crossings, one full length of water pipe shall be located so that both joints will be at least 10 ft. from the sewer, or 20 ft. of the water main shall be enclosed by casing centered on the sewer.
- The **Nebr. Dept. of Health** must specifically approve any variance from the requirements of these instructions when it is impossible to obtain the specified separation distances.
- Where sewers are being installed and these instructions cannot be met, the sewer materials shall be water main pipe or equivalent and shall be pressure treated to insure water tightness.

For additional information see the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 6, (Reference 1.24).

10.D.1 Manning's Equation for Open Channel Flow

The most widely used formula for determining the hydraulic capacity of storm sewers is Manning's equation expressed by:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{Eq. 1.5}$$

where:
- V = Velocity of flow, ft./sec.;
- n = Manning's roughness coefficient;
- R = The hydraulic radius defined as the cross sectional area of flow (A) divided by the wetted perimeter (WP) or (A/WP), ft.;
- S = Slope of the hydraulic grade line, ft./ft.

By combining Eq. 1.5 with the continuity equation (Q = VA), Manning’s equation can be used to directly compute discharge as indicated below:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad \text{Eq. 1.24}$$

where:
- Q = rate of flow, cfs.;
- A = cross sectional area of flow, ft^2 and the rest as before.

For storm sewers flowing full, the above equations become:

$$V_{full} = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad \text{Eq. 1.25}$$

$$Q_{full} = \frac{0.463}{n} D^{8/3} S^{1/2} \qquad \text{Eq. 1.26}$$

where:
- D = Diameter of pipe, in ft., and the rest as before.

Maximum discharge from a circular conduit actually occurs when the conduit is slightly less than full. Specifically, maximum flow occurs when the depth of flow in the conduit equals 0.96 D.

Manning’s equation can be solved using the nomographs presented in Appendix H, ”Nomographs and Charts for Storm Sewer Design”.

10.E Storm Sewer Hydraulic Grade Line

In order to determine if design flows can be accommodated by the storm sewer system without causing flooding, or causing flows to exit the system at unacceptable locations, it is necessary to determine the hydraulic grade line.

Computing the hydraulic grade line will determine the elevations, under design conditions, to which water will rise within inlets, manholes and junction boxes. Computer programs provided by **NDOT**, (e.g., Hydraflow), can be used to compute the hydraulic grade line. The following sections provide reference information necessary to determine the elevation of the hydraulic grade line.

The following design criteria should be followed when determining the elevation of the hydraulic grade line:

- The hydraulic grade line shall be 0.75 ft. below the intake lip of any affected inlet, any manhole cover, or the flow line of the highest pipe of any entering non-pressurized system.
- The energy grade line shall not rise above any such intake lip or manhole cover, or above the flow line of any such entering non-pressurized system.

10.F Storm Sewer Energy Losses

After runoff enters the storm sewer system, it will be conveyed through a variety of conduits and hydraulic structures such as manholes, inlets, enlargements and transitions. All of these elements of the conveyance system cause energy losses including friction losses and velocity head losses. The following sections discuss the various types of energy losses.

10.F.1 Friction Losses

Energy losses from pipe friction may be determined by modifying Manning's equation as indicated:

$$S_f = [Qn/1.486\ AR^{2/3}]^2 \qquad \text{Eq. 1.27}$$

where:
- S_f = Friction slope, ft./ft.;
- Q = Rate of flow, cfs;
- n = Manning's roughness coefficient;
- A = Cross sectional area of flow, ft^2;
- R = The hydraulic radius defined as the cross sectional area of flow (A) divided by the wetted perimeter (WP) or (A/WP), ft.

Head losses due to friction may be determined by the formula:

$$H_f = S_f L \qquad \text{Eq. 1.28}$$

where:
- H_f = Friction head loss, ft.;
- S_f = Friction slope, ft./ft. (Eq. 1.27);
- L = Length of conduit, ft.

10.F.2 Velocity Head Losses

Velocity head losses may be expressed in general form from Bernoulli and Darcy-Weisbach equations:

$$H = K(V^2)/2g \qquad \text{Eq. 1.29}$$

where:
- H = Velocity head loss, ft.;
- K = Coefficient for the particular type of velocity head loss under consideration (EXHIBIT 1.28);
- V = Velocity of flow, ft./sec.;
- g = Acceleration due to gravity, 32.2 ft./s^2.

Velocity head losses can be subdivided into two categories:

1. Terminal and entrance losses.
2. Junction losses.

10.F.2.a Terminal and Entrance Losses

Equations used for terminal and entrance losses are:

$$H_{tm} = (V^2)/2g \qquad \text{Eq. 1.30}$$

and

$$H_e = K(V^2)/2g \qquad \text{Eq. 1.31}$$

where:
- H_{tm} = Terminal loss at beginning of run, ft.;
- H_e = Entrance loss for end of run, ft.;
- K = 0.5 (assuming square edge 0.2 for beveled etc.); and other terms as defined earlier.

10.F.2.b Junction Losses

Junction losses can be subdivided into the following categories: incoming opposing flows, changes in direction of flow, and several entering flows.

10.F.2.b.1 Incoming Opposing Flows

The head loss at a junction, H_{j1}, for two almost equal and opposing flows meeting head on with the outlet direction perpendicular to both incoming directions, is considered as the total velocity head of outgoing flow.

$$H_{j1} = (V^2)\text{ (outflow)}/2g \qquad \text{Eq. 1.32}$$

where:
- H_{j1} = Junction loss, ft., and other terms as defined earlier.

10.F.2.b.2 Changes in Direction of Flow

When main storm drain pipes or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90°), the more severe the energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$H_b = K(V^2)\text{ (outlet)}/2g \qquad \text{Eq. 1.33}$$

where:
- H_b = Bend loss, ft.;
- K = Junction loss coefficient (EXHIBIT 1.39);
- V = Velocity of flow, ft./sec.;
- g = Acceleration due to gravity, 32.2 ft./s^2.

Degree of Turn (A) In Junction	K
15	0.19
30	0.35
45	0.47
60	0.56
75	0.64
90 and greater	0.7

Exhibit 1.39 Values of K for Change in Direction of Flow in Lateral Lines

EXHIBIT 1.40 lists the values of K for Eq.1.33 for various junction angles. EXHIBIT H.7 in Appendix H, "Nomographs and Charts for Storm Sewer Design", can be used for a graphic solution for determining values of K for degrees of turns not listed in EXHIBIT 1.39.

10.F.2.b.3 Several Entering Flows

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy, involving both position energy (elevation of water surface) and momentum energy (mass times velocity head). Thus, for a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction. Losses in a junction with several entering flows can be determined from Eq. 1.34, (See EXHIBIT H.7 in Appendix H, "Nomographs and Charts for Storm Sewer Design").

$$H_{j2} = [(Q_4V_4^2)-(Q_1V_1^2)-(Q_2V_2^2)+(KQ_1V_1^2)]/(2gQ_4) \qquad \text{Eq. 1.34}$$

where:
- H_{j2} = Junction losses, ft.;
- Q = Discharges (see subscript nomenclature below);
- V = Horizontal velocities, ft./sec.;
- V_3 = Assumed to be zero;
- g = Acceleration due to gravity, 32.2 ft./sec^2;
- K = Bend loss factor.

Subscript nomenclature for Eq. 1.34 is as follows:

- Q_1 = 90° lateral, cfs.;
- Q_2 = Straight through inflow, cfs.;
- Q_3 = Vertical dropped-in flow from an inlet, cfs.;
- Q_4 = Main outfall = total computed discharge, cfs.

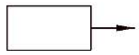
V_1, V_2, V_3, V_4 are the horizontal velocities of foregoing discharges, respectively, ft./sec.

Also assume:

- $H_b = K(V_1^2)/2g$ for change in direction of flow.
- No velocity head of an incoming line is greater than the velocity head of the outgoing line.
- Water surface of inflow and outflow pipes in junction is level.

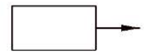
When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for any conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

A summary of energy losses is included in EXHIBIT 1.40. See Section 14.L for an example problem.



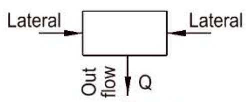
$$H_{tm} = \frac{v^2}{2g}$$

TERMINAL LOSSES
(at beginning of run)
Where g = gravitational constant,
32.2 feet per second per second.



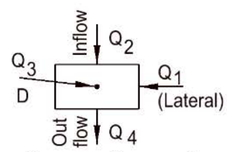
$$H_e = \frac{0.5\ v^2}{2g}$$

ENTRANCE LOSSES
(at end of run)
Assuming square - edge



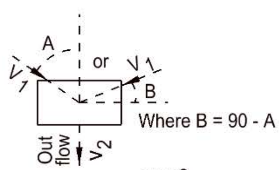
$$H_{j1} = \frac{v^2 \text{(Outflow)}}{2g}$$

JUNCTION LOSSES
(Incoming-opposing Flow)
Use only where flows are
identical to above, otherwise
use H_{j2} Equation.



$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ\ \ V_1\ I^2}{2gQ_4}$$

JUNCTION LOSSES
(Several Entering Flows)
Total losses to include H_{j2} plus losses for
changes in direction of less than 90^ (h_b).
Where K = Bend loss coefficient
Q_3 = Vertical dropped-in flow from an inlet
V_3 = Assumed to be zero



$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSSES (H_f)

$$H_f = S_f \times L$$

Where H_f = friction head loss
S_f = friction slope
L = length of conduit

Where $$S = \left(\frac{Q_n}{1.486AR^{\frac{2}{3}}}\right)^2$$

Q = Discharge of conduit
n = Mannings coefficient of roughness
A = area of conduit
R = hydraulic radius of conduit

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + (H_{j1} \text{ or } H_{j2}) + H_b + H_f$$

Exhibit 1.40 Summary of Energy Losses (Source: Reference 1.1)

11. SANITARY SEWERS

Highway construction, particularly in urban areas, may require realignment of sanitary sewers. State and local (city and county) standards, as appropriate, shall be used for the design and construction of sanitary sewers. All sanitary sewer design and construction will require review and approval from the **Nebraska Department of Environmental Quality** (See Rules and Regulations for the Design, Operation and Maintenance of Wastewater Works, Reference 1.18, (http://www.deq.state.ne.us/NDEQProg.nsf/RuleAndReg.xsp?databaseName=CN=DEQSER6/O=NDEQ!!RuleAndR.nsf&documentId=58E60373A84FB3DB06256BB900545D6C&action=openDocument)).

General guidelines to follow for sanitary sewer design include:

- Any generally accepted material for sanitary sewers should be given consideration, but the material selected should be adapted to local conditions, such as: character of industrial wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, abrasion, corrosion, and similar problems.
- No sanitary sewer main should have an inside pipe diameter of less than 8 in.
- Sanitary sewer laterals should match the existing inside pipe diameter.
- Sewer capacities should be designed for the estimated ultimate tributary population.
- Minimum velocity in the sewer should be 2 ft./sec. Special provisions shall be made to protect against displacement by erosion and impact if velocities exceed 15 ft./sec.
- Sanitary sewers should be sufficiently deep to receive wastewater from basements and to prevent freezing.
- Sanitary sewers shall be installed at least 10 ft. horizontally and 18 in. vertically from water mains (measured edge to edge). In cases where it is not practical to maintain a 10 ft. separation, the **Nebr. Dept. of Health** may allow installation of the sewer closer to the water main, provided that the sewer is laid in a separate trench or on an undisturbed earth shelf located on one side of the water main or at such an elevation that the bottom of the sewer is at least 18 in. above the top of the water main.
- When crossing a water main, the edge of the storm sewer shall be a minimum vertical distance of 18 in. from the outside edge of the water main. This shall be the case whether the sewer is above or below the water main. At crossings, one full length of water pipe shall be located so that both joints will be at least 10 ft. from the sewer, or 20 ft. of the water main shall be enclosed by casing centered on the sewer.
- The **Nebr. Dept. of Health** must specifically approve any variance from the requirements of these instructions when it is impossible to obtain the specified separation distances.
- Where sewers are being installed and these instructions cannot be met, the sewer materials shall be water main pipe or equivalent and shall be pressure treated to insure water tightness.
- For clearance from other utilities, consult the utility owner or operator.

For additional information, see the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 6.D.4, (Reference 1.24).

12. PIPE MATERIAL POLICY

Under this policy designers will select the allowable pipe material options for each installation. The contractor will choose the final pipe material from the list of options provided. For further information see Appendix C, “Pipe Material Policy”.

13. SPECIAL CONSTRUCTION

13.A Pavement Subdrains

Water trapped below pavement surfaces can cause premature or rapid pavement deterioration. Pavement subdrains are designed to remove water from the roadbed and prevent damage.

Pavement subdrain pipes should be designed to provide adequate slope for drainage, usually with grades between 0.2 and 0.5%. Subdrains that discharge into a storm drain or culvert should have sufficient outlet elevation above the flowline so that hydrostatic pressures will not develop and cause backflow problems. The plasticity index is used as a determining factor in when or where to place subdrains. The **Materials and Tests Division** specifies the type of pavement subdrain to be used, (See the Roadway Design Manual, Chapter Eight: Surfacing, Section 3, Reference 1.24).

13.B Inverted Siphons

The term “inverted siphon” refers to an inverted or depressed storm drain section which flows full under pressure. Its purpose is to carry the flow under an obstruction such as a stream, depressed highway, or utility, and to regain as much elevation as necessary after the obstruction has been passed. A primary consideration in the design of a siphon is to provide adequate self-cleaning velocities throughout. Since siphons flow under pressure, velocity varies directly with the quantity of flow. Minimum velocities should be about 3 ft./sec. with the ascending leg decreased in size to accelerate the flow to at least 4 ft./sec. Large siphons perform more satisfactorily and usually require less head for operation than small siphons. For this reason, several small siphons should be joined into one crossing where practicable.

In some cases, an upstream debris rack may be advisable to prevent debris from becoming lodged in the siphon; however, routine maintenance will be required to keep the debris rack clean. If all inlets to a system are grated, a debris rack probably will not be necessary.

Since there is always water in the siphon, freezing may occur resulting in blockage. Inverted siphons should be avoided unless the depressed barrel can be drained or pumped dry.

It is common practice, at least on larger storm drains, to construct multiple barrel siphons. The objective is to provide adequate self-cleaning velocities under widely varying flow condition. However, single barrel siphons have been constructed with diameters ranging in size from about 6 in. to 90 in. Generally, siphons will require more maintenance than a normal storm drain and should be avoided if possible. An access manhole at each end of the siphon and a means to drain it are suggested to facilitate maintenance.

13.C Boring and Jacking

Boring and jacking is a construction method used to install horizontal culvert pipe and utilities, in both soft earth and hard rock faces, below a highway when trench excavation is undesirable. Utilities installed by boring and jacking include water mains, sewers, electrical power, and communication lines. Typical installations include a casing and carrier pipes. Since no trench is required, the roadway can remain open.

Normally, concrete pipe is called for in the plans. It shall be reinforced concrete Class IV or Class V pipe, depending on the final height of the fill on the pipe (See Appendix C, "Pipe Material Policy"). If multiple pipes are to be jacked through stable soil, they should have a minimum of 1 ft. of clearance from outside of pipe to outside of pipe.

Note that if flared end sections are used on multiple pipes, additional clearance between pipes may be necessary to allow the flared ends to fit. Manufactured flared end sections are 12 in. wider than the pipe diameter. Refer to EXHIBIT 1.41 for jacking culvert pipe.

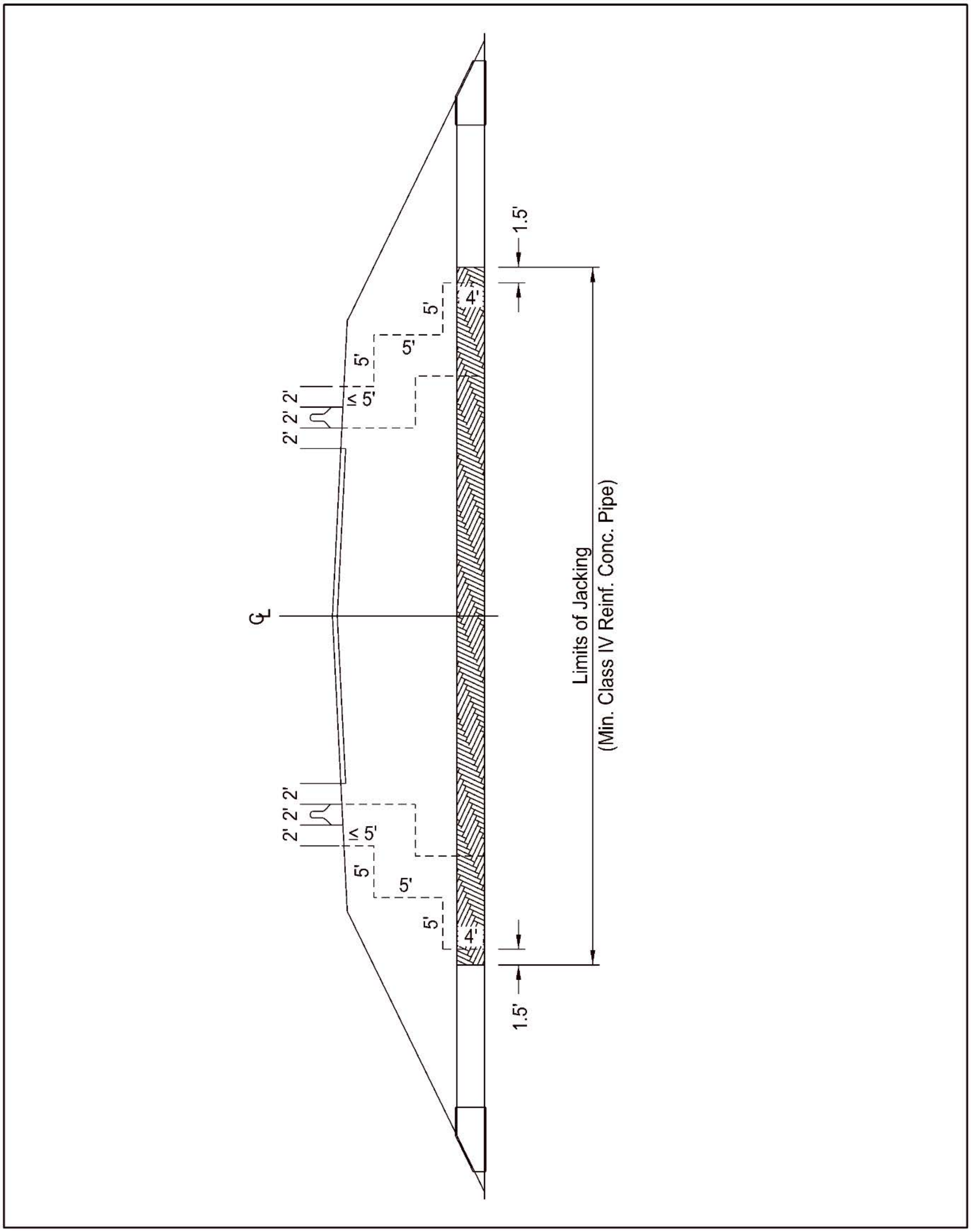


Exhibit 1.41 Jacking Culvert Pipe

13.D Detention, Retention, and Sedimentation Basins

Detention and retention facilities are required when peak stormwater flows cause downstream flooding, or when an existing stormwater system is too small to handle increased loads. Also, storage facilities are used when improvements to the current stormwater system are not possible or within budget constraints. Since new construction often results in increased runoff, many municipalities require retention or detention basins as part of a construction project.

Along with ensuring that the stormwater runoff basins perform as required, there are other considerations that should be evaluated. One area to be looked at is multiple use. For example, detention basins can be designed as soccer fields or volleyball courts and retention basins may provide aesthetically pleasing ponds or small lakes for parks and urban areas. Safety is another area of concern. Lighting and fences might be required if a retention or detention basin is near an area where children may play.

Detention basins are stormwater runoff facilities that usually have a dry bottom except during, and for some time after, the design event. The typical detention basin may be a swale or ditch, a dry pond, concrete basins, rooftops, or parking lots.

Retention basins are stormwater retention facilities that always contain some amount of water and have a capacity to store additional runoff when required. Retention basins often include ponds and small lakes.

Sedimentation basins are stormwater storage facilities that are designed to separate pollutants, suspended solids, and debris from stormwater. A sedimentation basin may be incorporated into the design of a detention or retention basin. More recently biological methods are being used to remove nutrients from the water. For additional information on sedimentation basins, refer to Chapter Two: Erosion and Sediment Control, Section 7.I.2.

14. EXAMPLE PROBLEMS

Example problems are provided in the following sections to illustrate the design procedures discussed earlier in the chapter.

14.A Rational Method

Determine peak discharge rate using the rational method for the following conditions:

Location: Fillmore County (Eastern Nebraska)
Drainage Area: 110 acres
Return Period: 25 year
Surface Type: Cultivated farm land; 2% - 7% slope
Length of Channel: 1,400 ft.
Height of Most Remote Point Above Outlet: 50 ft.

Step 1: Determine the coefficient of runoff (C).

From EXHIBIT 1.5, runoff coefficient (C) is 0.44.

Step 2: Determine the time of concentration (T_C) from EXHIBIT 1.11 using length of channel (maximum length of travel - 1,400 ft.) and height of most remote point above outlet (50 ft.).

Time of concentration from EXHIBIT 1.11 is 7.2 min. Multiply T_C by 1.5 for cultivated farmland, therefore, revised T_C = 7.2 min. X 1.5 = 10.8 min.

Step 3: Determine rainfall intensity (i) from EXHIBIT 1.9 using return period (25 year) and time of concentration (T_C = 10.8 min.).

From EXHIBIT 1.9, rainfall intensity = 6.9 in./hr.

Step 4: Calculate peak discharge rate using Eq. 1.1.

$$\begin{aligned} Q &= CiA \\ &= 0.44 \times 6.9 \times 110 \\ &= 334 \text{ cfs.} \end{aligned}$$

14.B Regression Equations

Determine peak discharge rate using regression equations for the following conditions:

14.B.1

Location: Cheyenne County (Region 1)
Drainage Area: 365 acres
Return Period: 10-year

Step 1: Determine drainage area (A_c) in sq. mi.

$$A_c = 365 \text{ ac} \times \frac{1 \text{ sq. mi.}}{640 \text{ ac}} = 0.570 \text{ sq. mi.}$$

Step 2: Find the mean annual precipitation (P).

From EXHIBIT 1.14, mean annual precipitation = 16 in.

Step 3: Using the appropriate regression equation from EXHIBIT 1.13, calculate peak discharge rate.

$$\begin{aligned} Q_{10} &= 211.7\, A_c^{0.324} P^{0.314} \\ &= 211.7 \times 0.570^{0.324} \times 16^{0.314} \\ &= 421 \text{ cfs} \end{aligned}$$

14.B.2

Location: Blaine County (Region 2)
Drainage Area: 250 acres
Return Period: 50-year
Channel Slopes, S: 75 ft./mi.

Step 1: Determine drainage area (A_c) in sq. mi.

$$A_c = 250 \text{ ac} \times \frac{1 \text{ sq. mi.}}{640\text{ac}} = 0.391 \text{ sq. mi.}$$

Step 2: Find the equivalent moisture content of snow (SN10).

From EXHIBIT 1.16, SN10 = 1.6 in.

Step 3: Calculate peak discharge rate using appropriate regression equation from EXHIBIT 1.13.

$$\begin{aligned} Q_{50} &= 0.0845\, A_c^{1.036}\, S^{2.005}\, SN10^{2.632} \\ &= 0.0845 \times 0.391^{1.036} \times 75^{2.005} \times 1.6^{2.632} \\ &= 632 \text{ cfs} \end{aligned}$$

14.B.3

Location:	Jefferson County (Region 5)
Drainage Area:	500 acres
Return Period:	100 year
Channel Slope, S:	35 ft./mi.

Step 1: Determine drainage area (A_c) in sq. mi.

$$A_c = 500 \text{ ac} \times \frac{1 \text{ sq. mi.}}{640 \text{ ac}} = 0.781 \text{ sq. mi.}$$

Step 2: Determine 2-year, 24 hour rainfall ($I_{24,2}$).

From EXHIBIT 1.15, $I_{24,2}$ = 3.0 in.

Step 3: Calculate peak discharge rate using appropriate regression equation from EXHIBIT 1.13.

$$\begin{aligned} Q_{100} &= 0.00335\, A_c^{0.615}\, S^{0.628}\, I_{24,2}^{10.491} \\ &= 0.00335 \times 0.781^{0.615} \times 35^{0.628} \times 3.0^{10.491} \\ &= 2{,}718 \text{ cfs.} \end{aligned}$$

14.C Concrete Box Culvert Design

Determine the size required for a reinforced cast-in-place concrete box culvert with a conventional entrance for a roadway crossing. From data collected in the field, the following information is known.

50-year Flow Rate, Q = 400 cfs
Culvert Length, L = 275 ft.
Natural Stream Slope, S = 3%
Roadway Elevation = 95 ft.
Inlet Elevation, Eli = 82 ft.
Maximum Allowable Headwater Elevation, Elhd = 92 ft.
Maximum Allowable Headwater = D+1 ft.
Tailwater, TW = 5 ft.
Entrance Configuration = Square Edge with 45º Wingwall Flare
Manning's n = 0.012
Entrance Loss Coefficient, k_e = 0.50

Fall is zero since culvert will be constructed on original streambed.

Step 1: List all given design data as shown on the tabulation form (See EXHIBIT 1.42 and EXHIBIT E.3 of Appendix E, "Design Forms and Checklists").

Step 2: Determine a trial size for the culvert. Although trial sizes can be found using applicable nomographs and equations, arbitrarily selecting a size is also appropriate.

Let the trial size be an 8 ft. x 6 ft. (span x rise) concrete box culvert.

Step 3: Assume inlet control and find the headwater depth (HW). Using the nomograph in EXHIBIT F.8 in Appendix F, "Nomographs and Charts for Culvert Design", compute headwater depth.

$$\begin{aligned} HW &= (HW/D) \times D_{trial} \\ &= 1.14 \times 6 \text{ ft.} \\ &= 6.84 \text{ ft.} \end{aligned}$$

Step 4: Check that the computed headwater elevation (EL_{hi}) is less than design headwater elevation (EL_{hd}).

$$\begin{aligned} El_{hi} &= El_i + HW \\ &= 82 \text{ ft.} + 6.84 \text{ ft.} \\ &= 88.84 \text{ ft.} \end{aligned}$$

$D+1 = 82 + 6 + 1 = 89$

88.84 ft. < 92 ft. OK.

Step 5: Assume outlet control and find the headwater depth (HW); from EXHIBIT F.11 compute critical depth (d_c) in the culvert.

$d_c = 4.27$ ft.

Step 6: Compute h_o, which is the greater of either tailwater depth (TW) or $\frac{d_c + D}{2}$

$$\frac{d_c + D}{2} = \frac{4.27 + 6}{2} = 5.13 \text{ ft.} > TW = 5 \text{ ft.}$$

Therefore, $h_o = 5.13$ ft.

Step 7: Calculate head losses (H) from EXHIBIT F.12.

$H = 2.18$ ft.

Step 8: Calculate outlet control headwater elevation (El_{ho}).

$EL_{ho} = EL_o + H + h_o = 73.75 + 2.18 + 5.13 = 81.06$ ft.

Step 9: The headwater elevation for inlet control (El_{hi}) is greater than the headwater elevation for outlet control (El_{ho}). Therefore, the controlling headwater elevation is El_{hi}. The headwater elevation of 88.84 is less than D + 1 ft., which would be 82 + 6 + 1 = 89, and is below maximum headwater elevation of 92, so this design will work.

Step 10: Try a 6 ft. x 6 ft. (span x rise) reinforced concrete box culvert and evaluate results.

CULVERT DESIGN FORM

PROJECT: ______ STATION: ______ SHEET ____ OF ____

C.N. ______

DESIGNER/DATE: ____ / ____

REVIEWER/DATE: ____ / ____

HYDROLOGICAL DATA

☐ METHOD: ______

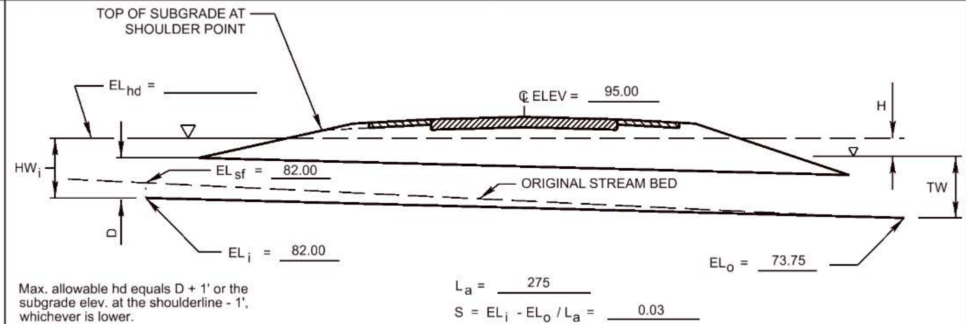
☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____

☐ CHANNEL SHAPE: ______

☐ ROUTING: ____ ☐ OTHER: ____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
50	400	5



CULVERT DESCRIPTION:

MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	Headwater Calculations: Inlet Control HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	Outlet Control: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATIONS	OUTLET VELOCITY	COMMENTS
Conc. Box - 8'x6' - Sq. Edge	400	50	1.14	6.84	0	88.84	5	4.27	5.13	5.13	0.5	2.18	81.06	88.84		Try 6' x 6'
Conc. Box - 6'x6' - Sq. Edge	400	67	1.50	9.00	0	91.00	5	5.17	5.58	5.58	0.5	4.30	83.63	91.00		OK

TECHNICAL FOOTNOTES:

(1) Use Q/NB for box culverts

(2) $HW_i/D = HW/D$ or HW_i/D from design charts

(3) Fall = $EL_{sf} - EL_i$; fall is zero for culverts on grade

(4) $EL_{hi} = HW_i + EL_i$ (Invert of inlet control section)

(5) TW based on downstream control or flow depth in channel.

(6) h_o = TW or $(d_c + D/2)$ (Whichever is greater)

(7) $H = [1+k_e + (29n^2 L)/R^{1.33}] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

COMMENTS/DISCUSSION:

6'x6' Box will work. Controlling headwater elevation (91.00) is below design or maximum allowable headwater elevation (92.00).

CULVERT BARREL SELECTED:

SIZE: 6'x6' (SxR)

SHAPE: Rectangular

MATERIAL: Concrete n 0.012

ENTRANCE: Sq. Edge, 45^ Flare

DEFINITIONS:

A. Cross-Sectional Area of the Barrel
a. Approximate
d_c. Critical Depth
D. Interior Height of Culv. Barrel
f. Culvert Face
g. Acceleration Due to Gravity (32.2 ft/s/s)
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
ke. Entrance Loss Coefficient
L. Length of Culvert Barrel
N. Number of Culvert Barrels
NB. Number of Boxes
o. Outlet
Q. Uniform Discharge, cfs
S. Slope of Culvert
sf. Streambed at Culvert Face
TW. Tailwater Depth Above the Outlet Invert
V. Average Velocity in Culv. Barrel (V=Q/A)

Exhibit 1.42 Example Culvert Design Form

14.D Curb and Gutter Flow

Calculate the width of flow or spread (T) in a low-speed (45 mph or less) curbed four-lane urban section of roadway. Determine if the spread is acceptable and within allowable criteria.

Type of Pavement and Curb:	Concrete pavement with integral concrete barrier curb
Gutter Flow, Q:	5.75 cfs
Longitudinal Slope, S:	0.0300 ft./ft.
Pavement Cross Slope, S_x:	0.0208 ft./ft.

Step 1: Determine Manning's roughness coefficient (n) for concrete pavement (gutter). From Appendix B, "Manning's Coefficient, n", n = 0.016.

Step 2: Calculate width of flow or spread using Eq. 1.7.

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3}$$

$$5.75 = \frac{0.56}{0.016} \times 0.0208^{5/3} \times 0.03^{1/2} \times T^{8/3}$$

$$T = 11.03 \text{ ft.}$$

Step 3: From EXHIBIT 1.37, the maximum allowable spread for a low-speed curbed four-lane urban roadway section is one full lane or 12 ft. The calculated width (from Step 2) is less than 12 ft.; therefore it is acceptable and meets design criteria.

14.E Capacity of Curb Inlet on Continuous Grade

A Standard Plan No. 443 curb inlet with an opening length (L) of 12 ft. will be used to intercept the gutter flow in the example problem in Section 14.D. Determine the:

- Amount of gutter flow intercepted.
- Amount of gutter flow bypassed to the next inlet.

Step 1: The capacity of a curb inlet on a continuous grade can be determined from EXHIBIT G.3 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design". The required information includes:

Length of Opening, L:	12 ft.
Depth of Gutter Depression, a:	5 in.
Gutter Flow, Q_a:	5.75 cfs
Depth of Flow in Gutter, d:	TS_x = 11.03 ft. x 0.0208 ft./ft. = 0.23 ft.

Step 2: From Chart A of EXHIBIT G.3:

$$Q_a/L_a = 0.25$$

Step 3: Determine length of inlet (L_a) which is required to intercept 100% of gutter flow using Eq. 1.9:

$$\begin{aligned} L_a &= Q_a/(Q_a/L_a) \\ &= 5.75/0.25 \\ &= 23 \text{ ft.} \end{aligned}$$

Step 4: Compute L/L_a ratio

$$\begin{aligned} L/L_a &= 12 \text{ ft.}/23 \text{ ft.} \\ &= 0.52 \end{aligned}$$

Step 5: From Chart B of EXHIBIT G.3:

$$Q/Q_a = 0.60.$$

Step 6: Determine flow intercepted and bypassed.

$$\begin{aligned} Q \text{ (intercepted)} &= 0.60 \times 5.75 \text{ cfs} \\ &= 3.45 \text{ cfs} \end{aligned}$$

$$\begin{aligned} Q \text{ (bypassed)} &= 5.75 \text{ cfs} - 3.45 \text{ cfs} \\ &= 2.30 \text{ cfs} \end{aligned}$$

14.F Capacity of Curb Inlet in a Low Point or Sump

A Standard Plan curb inlet with an opening length (L) of 14 ft. is located in the low point, or sump, of the road grade of an urban two-lane road. The following conditions apply at the inlet:

Flow to the Inlet, Q: 11.60 cfs
Pavement Cross Slope, S_x: 0.0208 ft./ft.

Determine if the inlet will sufficiently intercept the runoff within allowable spread criteria.

Step 1: The capacity of a curb inlet in a low point or sump can be determined from EXHIBIT G.4 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design". The required information includes:

Height of Opening, h: 5 in.
Length of Opening, L: 14 ft.
Flow to the Inlet, Q: 11.60 cfs

Step 2: Determine the depth of ponded water (H) at the inlet from EXHIBIT G.4.

Q/L = 11.60 cfs/14 ft.
= 0.83 cfs/ft.

Enter the nomograph EXHIBIT G.4 with h = 5 in. and Q/L = 0.83 cfs/ft. and find H/h. H/h = 1.15.

H = 1.15 x 5 in.
= 5.75 in.
= 0.48 ft.

Step 3: Determine the width of flow or spread (T).

T = (5.75 in. - 5.0 in.) / 0.0208 ft./ft.
= 3 ft.

From EXHIBIT 1.37, the maximum allowable spread width for a two-lane road is 6 ft., therefore, a spread of 3 ft. is acceptable.

14.G Capacity of Grate Inlet on a Continuous Grade

Find the interception capacity (Q_i) for a curved vane grate on a continuous grade with the following conditions:

L = 2 ft.
W = 2 ft.
T = 8 ft.
S_x = 0.025 ft./ft.
S = 0.01 ft./ft.
E_o = 0.69
Q = 3.0 cfs
V = 3.1 ft./sec.
Gutter depression = 2 in.

Step 1: From EXHIBIT G.7 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design", for curved vane grate, R_f = 1.0. From EXHIBIT G.8 R_s = 0.1.

Step 2: Calculate interception capacity (Q_i) using Eq. 1.15.

$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)]$$
$$= 3.0 \text{ cfs } [1.0 \times 0.69 + 0.1(1 - 0.69)]$$
$$= 2.2 \text{ cfs}$$

14.H Capacity of Grate Inlet in a Low Point or Sump

14.H.1 Weir Condition

Find the grate size required for the design storm flow indicated below and determine the spread of flow at the curb for the design and check storm. The check storm is an arbitrary storm greater than the design storm, which verifies the capacity of the grate size selected. Allow for 50% clogging of the grate.

Q_b = 3.6 cfs (bypass flow-design storm)
Q = 8.0 cfs (design storm)
Q_b = 4.4 cfs (bypass flow-check storm)
Q = 11.0 cfs (check storm)
T = 10 ft. (allowable design spread)
S_x = 0.05 ft./ft.
d = TS_x = 0.5 ft.

Step 1: Find the required perimeter (P) from EXHIBIT G.9 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design".

For Q = 8.0 cfs and d = 0.5 ft.
P = 8 ft.

Step 2: Some assumptions must be made regarding the nature of clogging in order to compute the capacity of a partially clogged grate:

- Assume that 50% of a grate is covered by debris and that the debris covered portion does not contribute to its interception.
- The area of the grate opening is reduced 50% but the effective perimeter of the grate is reduced by an amount less than 50%. For example, a 2 ft. x 4 ft. grate with curb has a total effective perimeter of 2 ft. + 4 ft. + 2 ft. or 8 ft. If the grate is clogged so that its effective width is 1 ft., then the effective perimeter is 1 ft. + 4 ft. + 1 ft. or 6 ft.

The area of the opening is reduced by 50% and the perimeter is reduced by 25%. Therefore, assuming 50% clogging along the length of the grate, grates that are 2 ft. x 6 ft., 3 ft. x 5 ft., or 4 ft. x 4 ft. meet the requirements of an 8 ft. perimeter grate that is 50% clogged.

14.H.2 Orifice Condition

Determine grate size required for design flow (Q). Assume there is no curb and:

Q = 16 cfs
T = 10 ft.
S_x = 0.05 ft./ft.

Step 1: Determine required grate size from EXHIBIT G.9 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design".

$$d = TS_x$$
$$= 10 \text{ ft.} \times 0.05 \text{ ft./ft.}$$
$$= 0.50 \text{ ft.}$$

From EXHIBIT G.9:

$$P = 2(W + L)$$
$$= 15 \text{ ft.}$$

The following grate sizes meet the required perimeter (P), therefore, are suitable for intercepting the design flow:

- 4 ft. x 4 ft.
- 2 ft. x 6 ft.
- 3 ft. x 5 ft.

14.I Capacity of a Slotted Pipe Inlet in a Low Point or Sump

14.I.1 Weir Condition

Find the interception capacity (Q_i) of a slotted inlet with the following conditions:

W = 1.5 in
L = 10 ft.
d = 2 in.

Step 1: Determine the interception capacity (Q_i) from EXHIBIT G.12 in Appendix G, "Nomographs and Charts for Gutter Flow & Inlet Design".

$$d = 2 \text{ in.} = 0.17 \text{ ft.}$$

$$\begin{aligned} Q_i &= 1.75 \text{ cfs} \times (1.50 \text{ in.}/1.75 \text{ in.}) \\ &= 1.50 \text{ cfs} \end{aligned}$$

14.I.2 Orifice Condition

Find the interception capacity (Q_i) for a slotted inlet for the following conditions:

W = 1.75 in.
L = 10 ft.
d = 5 in.

Step 1: Calculate interception capacity (Q_i) from Eq. 1.22.

$$\begin{aligned} Q_i &= 0.94Ld^{0.5} \\ &= 0.94 \times 10 \text{ ft.} \times (0.417 \text{ ft.})^{0.5} \\ &= 6.07 \text{ cfs} \end{aligned}$$

14.J Capacity of a Slotted Pipe Inlet on a Continuous Grade

Determine the amount of gutter flow intercepted (Q_i) by a slotted inlet under the following conditions:

L = 10 ft.
S = 0.02 ft./ft.
S_x = 0.0417 ft./ft.
Q = 1.39 cfs
n = 0.015

Step 1: Calculate the length of slotted inlet required to intercept 100% of gutter flow (L_T) from Eq. 1.18.

$$\begin{aligned} L_T &= 0.6\, Q^{0.42}\, S^{0.3}\, (1/nS_x)^{0.6} \\ &= 0.6 \times (1.39)^{0.42} \times (0.02)^{0.3} \times (1/0.015 \times .0417)^{0.6} \\ &= 17.8 \text{ ft.} \end{aligned}$$

Step 2: Calculate the efficiency (E) of the slotted inlet from Eq. 1.19.

$$
\begin{aligned}
E &= 1 - (1 - L/L_T)^{1.8} \\
&= 1 - 1 - (10/17.8)^{1.8} \\
&= 0.77
\end{aligned}
$$

Step 3: Calculate amount of gutter flow intercepted (Q_i) from Eq. 1.20.

$$
\begin{aligned}
Q_i &= EQ \\
&= 0.77 \times 1.39 \\
&= 1.07 \text{ cfs}
\end{aligned}
$$

14.K Capacity of Slotted Vane Inlet

Determine the interception capacity (Q) of a slotted vane inlet with the following conditions:

Depth of Flow in Gutter Upstream of Slotted Vane Drain, d = 0.25 ft.
Transverse Gutter Slope, S_T = 2%
Longitudinal Gutter Slope, S_L = 4%

Step 1: Determine the value of the slotted vane inlet coefficient (K) from EXHIBIT G.13 in Appendix G, “Nomographs and Charts for Gutter Flow & Inlet Design”.

$$
K = 36.5
$$

Step 2: Calculate the interception capacity (Q) using Eq. 1.23.

$$
\begin{aligned}
Q &= Kd^{5/3} \\
&= 36.5 \times 0.25^{5/3} \\
&= 3.62 \text{ cfs}
\end{aligned}
$$

The capacity of the curved vane grate component of the inlet can be determined from hydraulic design procedures presented in Section 10.B.3.

14.L Storm Sewer and Inlet System

The following example problem illustrates the hydraulic design of a storm sewer and inlet system. The storm sewer and inlet system is shown in EXHIBIT 1.43. The Drainage Computation Form used in the example can be found in EXHIBIT E.4 of Appendix E, "Design Forms and Checklists".

Step 1: Using the computation form provided, list all given data for each drainage area such as area, inlet time (Ti), and runoff coefficients (C). Values for runoff coefficients can be found in EXHIBITS 1.4 AND 1.5. Also, list all relevant information provided for the pipe sections in the storm sewer system, such as pipe length and slope.

Step 2: The analysis will utilize the pipe section between structure 1 and structure 2. The runoff (Q) will be calculated to structure 1 in order to determine the pipe size required. For the given inlet time (Ti) for drainage area A, find the corresponding value for rainfall intensity (i) in EXHIBIT 1.9, and record on the computation form.

Given: Ti = 6.9 min.; i = 6.9 in./hr.

Step 3: Multiply the values for the runoff coefficient (C) and the area (A). Record in the column for incremental data. No other drainage areas are tributary to structure 1, therefore, record the same value in the cumulative column.

$$C \times A = 0.30 \times 1.5 \text{ acres} = 0.45$$

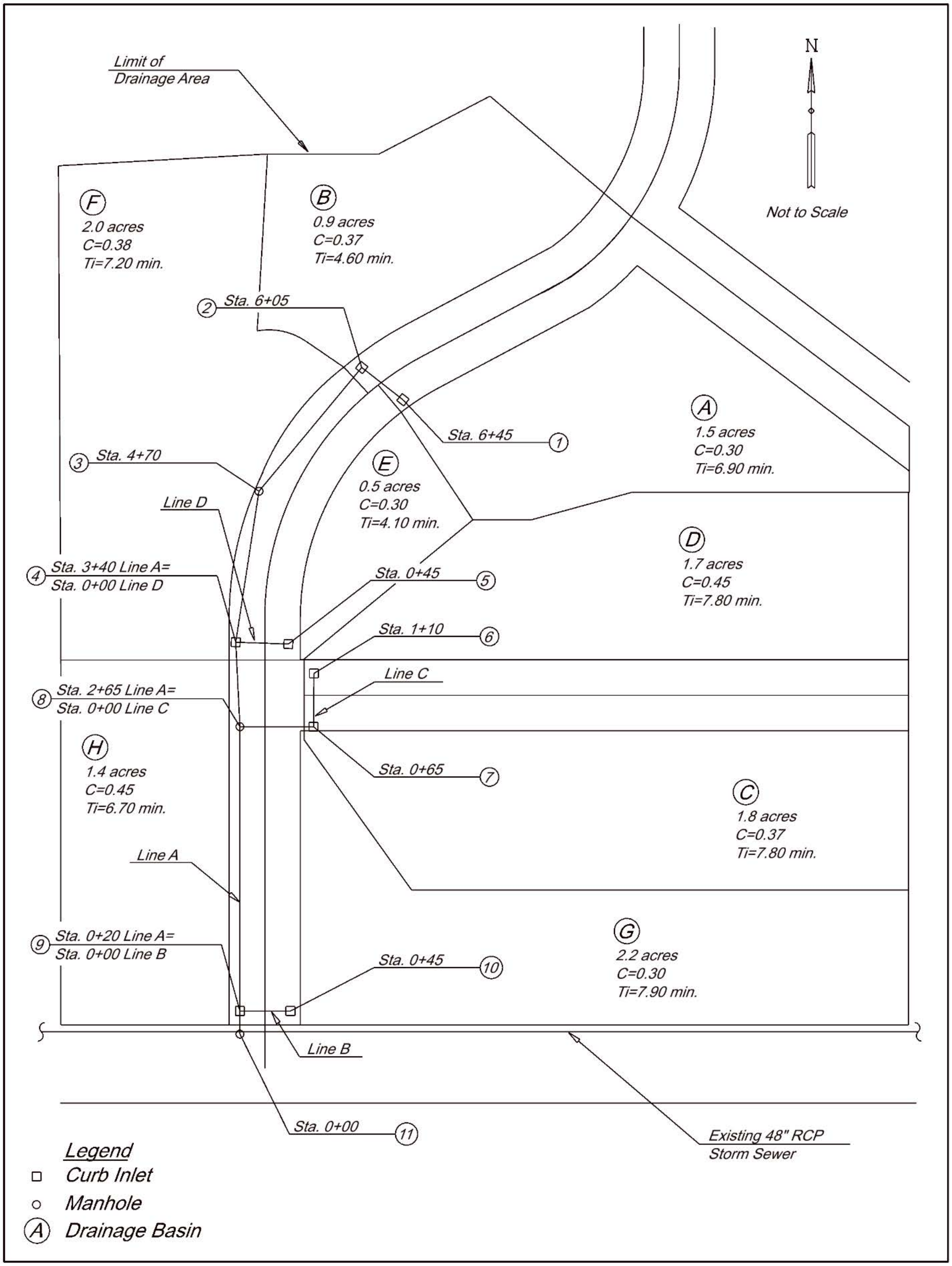


Exhibit 1.43 Drainage Area Map

Step 4: The runoff (Q) is calculated as follows from Eq. 1.1:

$$
\begin{aligned}
Q &= C\ i\ A \\
&= 0.45 \times 6.9 \text{ in./hr.} \\
&= 3.11 \text{ cfs}
\end{aligned}
$$

Step 5: Obtain the corresponding Manning's n value for the selected pipe material from Appendix B, "Manning's Coefficient, n". Using EXHIBIT H.2 of Appendix H, "Nomographs and Charts for Storm Sewer Design", select a pipe diameter.

Given: S = 1.25%; n = 0.012 and Q = 3.11 cfs

The corresponding pipe size selection is a 15 in. diameter pipe. Pipe capacity and velocity flowing full can be calculated using Manning's equation or EXHIBIT H.4.

Step 6: A travel time (Tt) of 0.20 min. is initially assumed for all pipe sections in order to select the pipe sizes required. Calculate the time of concentration (Tc) in subsequent pipe sections within the system as follows:

$$
\begin{aligned}
Tc &= Ti + Tt \\
&= 6.90 + 0.20 \\
&= 7.10 \text{ min.}
\end{aligned}
$$

The maximum time of concentration of all drainage areas tributary to the structure should be utilized in obtaining the runoff (Q) to each structure. If the time of concentration computed is less than 5 min., a time of concentration of 5 min. shall be used.

$Tc_{(A)} = 6.90 + 0.20 = 7.10$ min.

$Ti_{(B)} = 4.60$ min. (Given)

Therefore, use Tc = 7.10 min. for structure 2

Continue with the analysis of the other structures and pipe sections within the system.

Step 7: Once pipe size selections are made for the entire system, Tt must be corrected and the rainfall intensity adjusted accordingly in order to calculate the corresponding Q.

Using the pipe diameter of 15 in. for pipe section 1-2, travel time in the pipe (Tt) can be computed as follows:

$$Tt = L/V = \frac{40 \text{ ft.}/6.38 \text{ fps}}{60 \text{ sec./min.}} = 0.10 \text{ min.}$$

$Tc_{(A)} = 6.90 + 0.10 = 7.00$ min.

$Ti_{(B)} = 4.60$ min. (Given)

Therefore, use Tc = 7.00 min. for structure 2

Step 8: Correct Tt for the remaining pipe sections of the system. Verify that the corrected runoff does not exceed the selected pipe's full flow capacity.

Note: For structures where there is more than one tributary drainage area involved, a weighted runoff coefficient should be calculated as follows:

$$C_w = \frac{C_{(1)}A_{(1)} + C_{(2)}A_{(2)}}{A_{(1)} + A_{(2)}}$$

The following example illustrates the weighted runoff coefficient concept for structure 4 which involves drainage areas E and F:

$$C_w = \frac{(0.3 \times 0.5) + (0.38 \times 2.0)}{(2.0 + 0.5)} = 0.36.$$

The following steps illustrate the calculation of the hydraulic grade line for the storm sewer system illustrated in EXHIBIT 1.44. The Hydraulic Grade Line Computation Form used in the example can be found in EXHIBIT E.5.

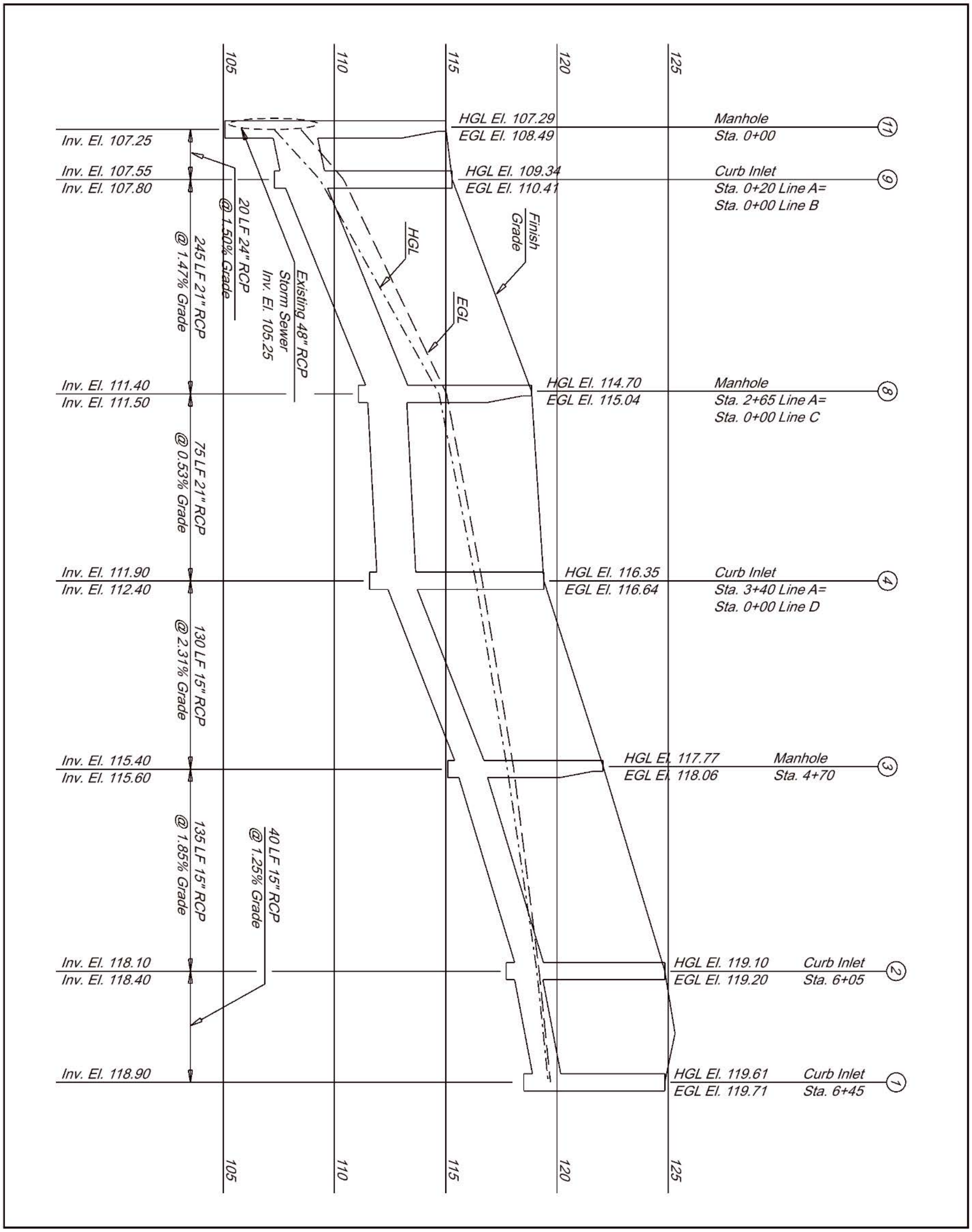


Exhibit 1.44a Storm Sewer Profile

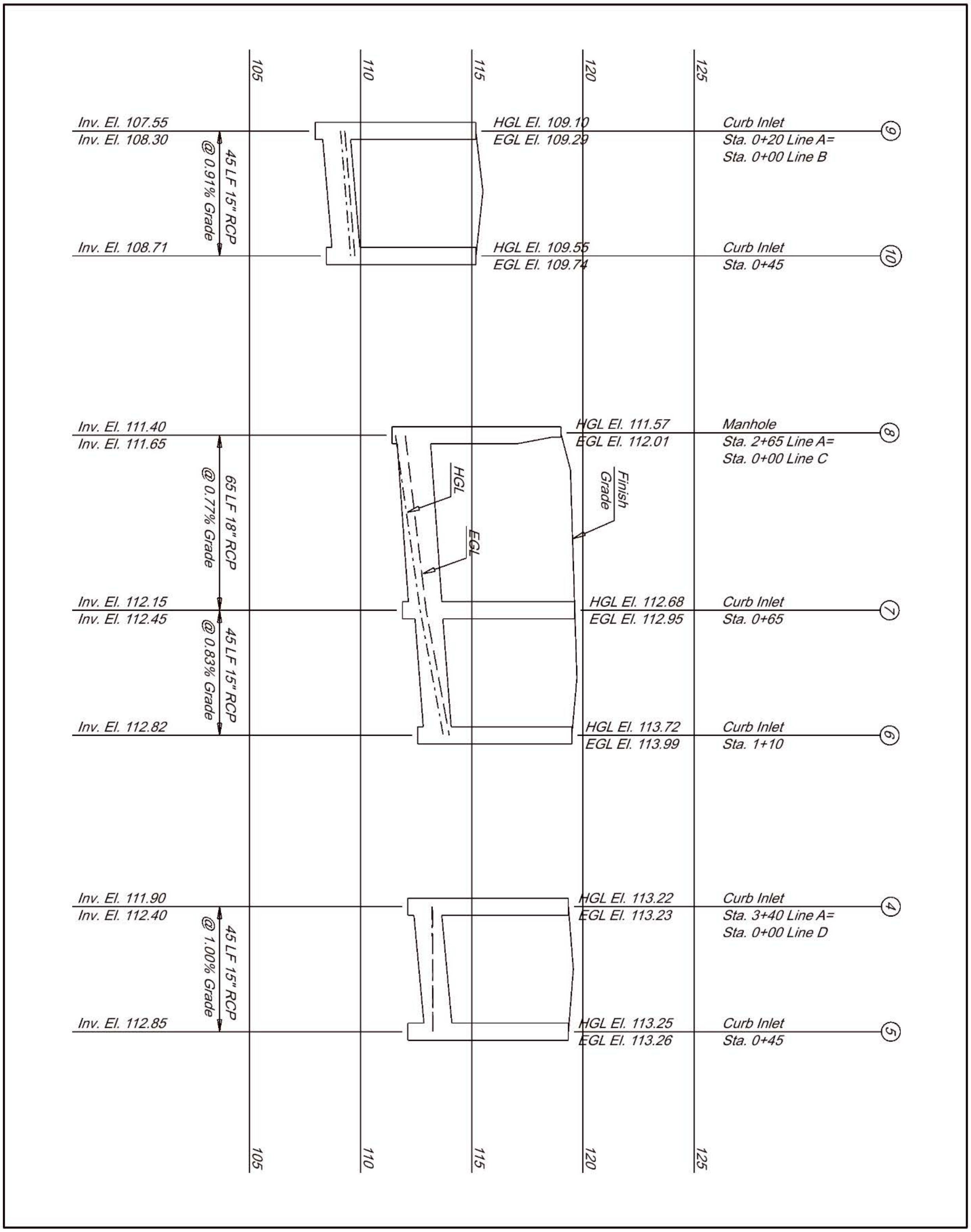


Exhibit 1.44b Storm Sewer Profile

Step 9: Our analysis will compute the hydraulic grade line for the pipe section from structure 1 to structure 2. Enter all known data for the sewer system such as pipe size, pipe length, pipe slope, n value, runoff (q), and pipe capacity (Q_{full}). From this data, proceed to calculate the cross-sectional area (A_{full}) of the pipe.

Step 10: Establish the type of flow within the storm sewer system in order to determine the direction in which calculations are to proceed. Determine the normal depth and the critical depths for the pipe section. Normal depth may be calculated using the hydraulics elements chart in EXHIBIT H.6. For pipe section 1-2, normal depth is computed as follows.

q/ Q_{full} = 3.11 cfs/7.82 cfs = 0.40

Enter the hydraulics element chart on the horizontal axis at 0.40 and follow this value up to the discharge curve (for n, f constant with depth) and read across to the vertical axis of d/D values.

d/D = 0.44

d_n = 0.44 x (15 ft. /12 in.) = 0.55 ft.

Critical depth within the pipe section may be obtained from EXHIBIT H.5 utilizing q= 3.11, D = 15 in., and n = 0.012

d_c = 0.71 ft.

If dc > dn, then the flow is supercritical and calculations for the hydraulic grade line will proceed downstream. If dc < dn, then flow is subcritical and hydraulic grade line calculations will proceed upstream.

d_n = 0.55 ft. < dc = 0.71 ft.

Therefore, the flow is supercritical for the pipe section.

Calculate normal and critical depths for the remaining pipe sections within the sewer system. For all pipe sections in the system, the normal depth is less than the critical depth, therefore, calculations will proceed downstream.

Step 11: Calculate the velocity within the pipe sections as follows:

$V_q = q / A_{full}$

where q = runoff and A_{full} = cross sectional area of the pipe.

For pipe section 1-2, the velocity is computed as follows:

$$V_q = \frac{3.11\text{ cfs}}{1.23\text{ sf}} = 2.53\text{ fps}$$

Step 12: The water surface elevation must be established at the control section, structure 1, and is commonly taken as the critical depth of the pipe section. The hydraulic grade line (HGL) can be calculated by adding the critical depth of pipe section 1-2 to the invert elevation of the pipe at the control section.

$$HGL_{(1)} = 118.90 \text{ ft.} + 0.71 \text{ ft.} = 119.61 \text{ ft.}$$

Step 13: Compute the energy grade line (EGL) at structure 1 as follows:

$$EGL_{(1)} = HGL_{(1)} + (V^2/2g)_{(1\text{-}2)}$$

The velocity within pipe section 1-2 will be used for calculating the velocity head for structure 1

$$\text{Velocity head}_{(1\text{-}2)} = (2.53)^2/(2 \times 32.2) = 0.10 \text{ ft.}$$

$$EGL_{(1)} = 119.61 + 0.10 = 119.71 \text{ ft.}$$

Step 14: The total energy losses between Structure 1 and Structure 2 need to be computed. At structure 1, there is a terminal loss ($H_{tm(1)}$) at the beginning of pipe section 1-2 which is computed using Eq. 1.30:

$$H_{tm\,(1)} = (V^2/2g)_{(1\text{-}2)}$$
$$= 0.10 \text{ ft.}$$

Using Eq. 1.27 and 1.28, friction losses along pipe section 1-2 can be calculated as follows:

$$S_f = [Q_n/1.486AR^{2/3}]^2$$

where R = D/4 and A = cross sectional area of the pipe.

$$S_f = [(3.11 \text{ cfs} \times 0.012)/(1.486 \times 1.23 \text{ sf} \times (15 \text{ in.} \times (1 \text{ ft.}/12 \text{ in.})/4)^{2/3})]^2$$
$$= 0.002$$

$$H_{f\,(1\text{-}2)} = S_{f\,(1\text{-}2)} \times L_{(1\text{-}2)}$$
$$= 0.002 \times 40 \text{ ft.} = 0.08 \text{ ft.}$$

There is an entrance loss ($H_{e\,(2)}$), at structure 2 for the end of pipe section 1-2 which is computed using Eq. 1.31 as follows:

$$H_{e\,(2)} = 0.5\,(V^2/2g)_{(1\text{-}2)}$$
$$= 0.05 \text{ ft.}$$

Structure 2 also has junction losses due to several entering flows:

$H_{j2\ (2)} = [(Q_3V_3^2) - (Q_1V_1^2) - (Q_2V_2^2) + (KQ_1V_1^2)] / (2gQ_3)$

where:

Q_3 = 5.34 cfs
V_3 = 4.35 fps
Q_1 = 3.11 cfs
V_1 = 2.53 fps
K = 0.7 for a 90º or greater bend. (Refer to EXHIBIT 1.39).

thus

$H_{j2\ (2)}$ = 0.28 ft.

Note: V_2 is assumed to be zero, therefore, $Q_2V_2 = 0$. Also, H_{j2} accounts for the bend losses occurring within the junction.

Step 15: The EGL elevation at structure 2 is computed by subtracting the total energy losses computed between structures 1 and 2 from the EGL elevation at structure 1.

$EGL_{(2)} = EGL_{(1)} - H_{tm\ (1)} - Hf_{(1\text{-}2)} - H_{j2\ (2)} - H_{e\ (2)}$
= 119.66 - 0.10 - 0.08 - 0.28 - 0.05 = 119.20 ft.

Step 16: The HGL elevation at structure 2 is computed as follows:

$HGL_{(2)} = EGL_{(2)} - (V^2/2g)_{(1\text{-}2)}$

Note: The hydraulic grade line at a given structure will be computed utilizing the velocity head for the upstream pipe section.

$HGL_{(2)}$ = 119.20 - 0.10
= 119.10 ft.

Continue the analysis of the remaining structures and pipe sections within the system as described in Steps 14 through 16.

15. REFERENCES

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The information contained in Chapter Two: Erosion and Sediment Control, dated August 2006, has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

Chapter Two
Erosion and Sediment Control

Soil erosion is a naturally occurring phenomenon where soil particles are displaced and carried away by water, wind or other agents. The rate at which erosion occurs depends upon the properties of the soil, terrain, climate, rainfall intensity and duration, and the volume and characteristics of the water flow.

Sedimentation is the deposition of eroded soil and may occur in lakes, reservoirs, streams, or other drainage ways. Sedimentation may restrict drainage ways, plug culverts, damage property and adversely impact stream ecological systems.

Erosion and sediment control is accomplished by:

- Absorbing the impact of rainfall.
- Slowing water's velocity, dividing water into smaller quantities.
- Infiltration by soil or vegetation.
- Retention or temporary detention.

Highway construction involves disturbance of large land areas. Erosion and sediment control is a major concern in highway construction and is addressed during all phases of the project from planning and design through construction, and continues into maintenance. An erosion and sediment control program includes the plans of action and provision of documents to achieve an acceptable level of erosion and sediment control.

Roadway designers must keep in mind the need for erosion and sediment control throughout the entire design phase. A preliminary erosion control plan should be developed at the earliest phase of design. This will enable the designer to review the design for effectiveness at the Plan-in-Hand Review or other on site visit. Designers should work closely in the early stages of design with the **Roadside Development & Compliance Unit** (**RDC**) in the **Project Development Division** to achieve erosion and sediment control objectives. EXHIBIT 2.1 shows the erosion control design process designers should follow. Development of an early erosion control plan will help improve cost estimates and will aid the final erosion control design by highlighting areas of concern.

Erosion and sediment control plans must comply with applicable federal, state and local (city and county) rules and regulations, including, but not limited to, the requirements of the National Pollutant Discharge Elimination System Construction Site Permit issued by the **Nebraska Department of Environmental Quality.** The **Project Development Division** requests this permit, (See the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 4, Reference 2.16, (http://www.roads.nebraska.gov/business-center/design-consultant/rd-manuals/)).

Since, in general, the effects of water erosion are more severe than the effects of wind erosion on Nebraska construction projects this chapter primarily focuses on water erosion control measures and techniques to achieve desired on-site erosion and sediment control.

The following References have been used extensively throughout this chapter:

- Highway Drainage Guidelines, (Reference 2.1).
- Model Drainage Manual, (Reference 2.2).
- A Guide For Transportation Landscape and Environmental Design, (Reference 2.3).
- Manual of Erosion and Sediment Control and Stormwater Management Standards, (Reference 2.4).
- Erosion and Sediment Control Manual, (Reference 2.5). (http://www.transportation.wv.gov/highways/engineering/files/erosion/erosion2003.pdf).

1. EROSION AND SEDIMENT CONTROL OBJECTIVES

An effective erosion and sediment control program must accomplish these four objectives:

- Limit both on-site and off-site impacts to acceptable levels both during and after construction.
- Facilitate project construction while minimizing overall costs.
- Aid in the restabilization of the construction site, reducing the long term maintenance requirements.
- Comply with federal, state and local regulations.
- Require minimal maintenance.

Controlling adverse impacts from all construction activities is an important goal of roadway design. Controlling the effects of erosion on-site may facilitate construction activities and reduce the amount of earth re-work required as a result of runoff.

Erosion control measures should:

- Be simple to construct.
- Minimize interruption to normal construction procedures and operations.
- Be effective in their operation.

In addition to controlling any off-site impacts, the erosion and sediment control program is designed to promote revegetation of the construction sites as quickly as possible and to reduce the maintenance requirements of the roadside over the long term.

The fourth objective for an erosion control program is compliance with federal, state and local regulations. Federal controls are administered by several agencies through various permitting requirements. The **Project Development Division** will coordinate the permitting requirements (See the Roadway Design Manual, Chapter Thirteen: Planning and Project Development, Section 4, (Reference 2.16).

The **Federal Highway Administration (FHWA**) requires erosion and sediment control measures be included in the Plans, Specifications and Estimates (PS&E) package for all federal-aid projects. At a minimum, **FHWA** requires the identification of all erosion and sediment-sensitive areas and identification of the methods to be used for minimizing adverse effects. In addition to the **FHWA** requirements, the "National Pollutant Discharge Elimination System" (NPDES) permit requires erosion and sediment control plans for all sites that are 1 acre (0.4 hectare) or larger in size.

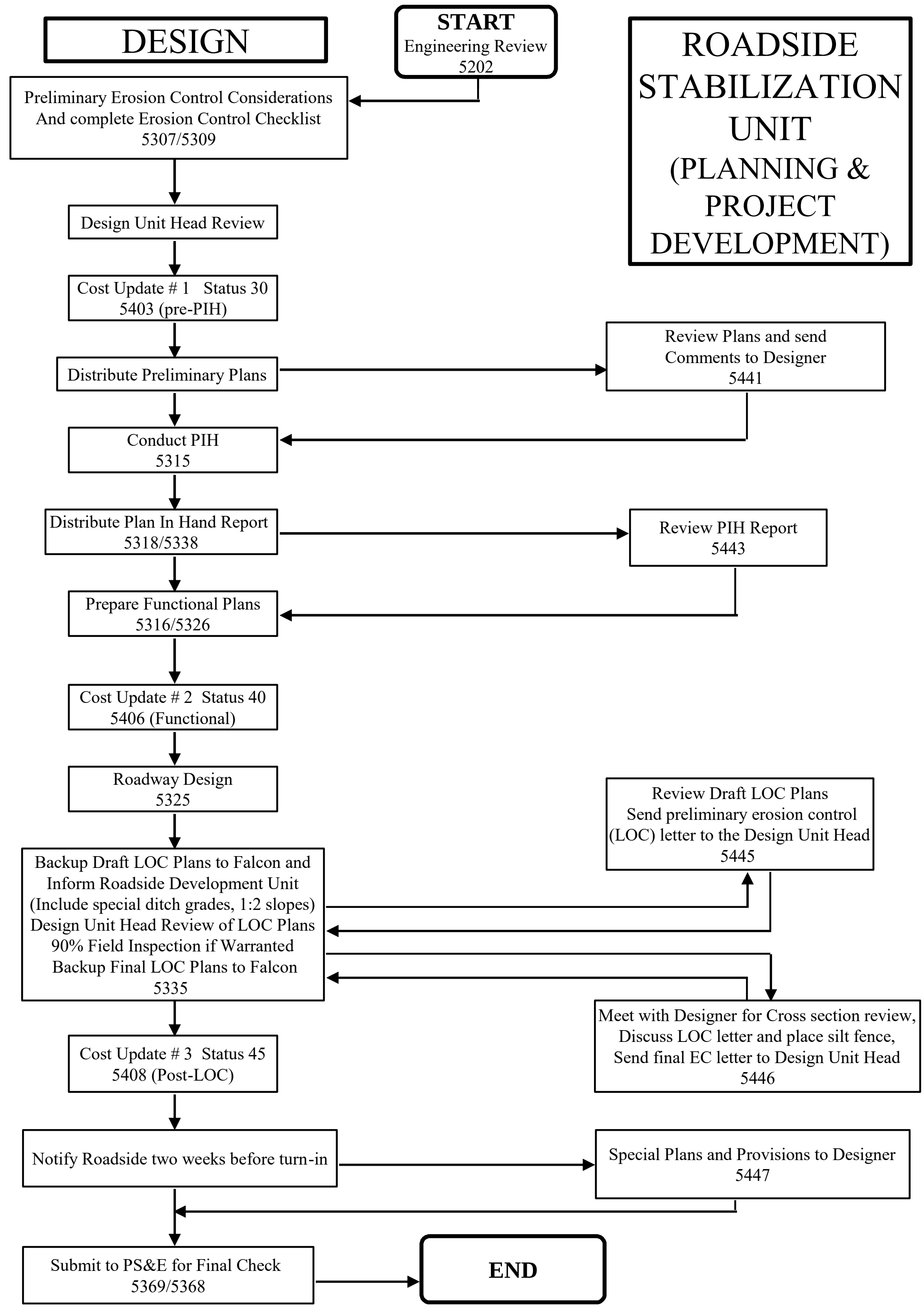


Exhibit 2.1 Erosion Control Design Process

2. SAFETY AND AESTHETICS

Early consideration of the effects of erosion and the specification of proper countermeasures during the design phase can lessen the amount of future maintenance due to erosion and can prevent potential highway hazards. Erosion control can be incorporated directly into the roadway geometric design (especially in cross section design), in the drainage design, and in the landscaping provisions (See the Roadway Design Manual, Chapter Six: The Typical Roadway Cross-Section, Chapter Ten: Miscellaneous Design Issues, Section 3 (Reference 2.16), and Chapter One: Drainage, of this manual). Erosion, safety hazards and maintenance may be minimized by use of properly designed:

- Flat side slopes that gradually transition to the natural terrain.
- Benching of steep slopes (where the slope is 1:3 or greater and 30 ft. or more in length as measured along the slope).
- Drainage channels - width, depth, cross section, slope alignment and protective treatment.
- Inlets, especially with regard to location and spacing.
- Culvert inlets.
- Culvert outlets.
- Groundwater interception facilities.
- Dikes, berms, etc. to protect backslopes.
- Sedimentation devices such as silt fences, silt traps, ditch checks, etc.

Revegetation practices that focus on native and introduced varieties of vegetation are the basis for our erosion control program. The root structure and vegetation supplied by these plants will be responsible for controlling erosion on the site for years after the construction is complete and will provide a more aesthetic roadway environment. Revegetation should focus on:

- Preservation of existing vegetation where possible.
- The transplanting of existing vegetation if necessary and feasible.
- Planting of new native vegetation or of an introduced species that is highly adaptable to the area.
- Selective clearing and thinning.
- Regeneration of existing native and introduced plant species.
- Salvage and reuse topsoil.

3. GENERAL EROSION AND SEDIMENT CONTROL DESIGN CONSIDERATIONS

Erosion is likely to occur at any concentration of flow; however, it occurs most severely in high flow concentrations. Erosion most commonly occurs:

- On longitudinal slopes of more than 1,000 ft. (300 m) (or less depending on the percent slope and soil type.
- On the outer banks of curved channels.
- At a culvert outlet or inlet.
- Where the longitudinal slope of a ditch exceeds 1.5%.
- Where there is sheet flow over a foreslope or backslope.
- At the ends of bridge structures.

The locations of potential erosion throughout the project site should be identified. Consideration should also be given to the general soil type found within the area of the project. Non-cohesive soils (e.g. Loess and sandy soils) are more easily erodable and may require additional erosion control. Soil survey manuals showing soil types with their engineering properties including, susceptibility to erosion, are available in the **NDOT** Library for all Nebraska Counties.

A preliminary erosion and sediment control plan should be completed prior to the Plan-in-Hand site visit. This will give the roadway designer an opportunity to review the plans for effectiveness and to make any necessary design changes. The *Erosion Control Plan-In-Hand Checklist*, **Exhibit F** of the Design Process Outline, (Reference 2.17, http://www.roads.nebraska.gov/media/6761/design-process-outline.pdf), is available to the designer as a tool, used to determine additional items to examine on the Plan-in-Hand. This checklist should be reviewed during the Plan-in-Hand in consultation with the **District Engineer** and **District Construction Engineer**. It should then be sent to the **Roadside Development & Compliance Unit** in the **Project Development Division**, along with the erosion and sediment control plans, for their review and comment prior to the final plan review, (See the Roadway Design Manual, Chapter Two: Roadway Design Process, Section 7, Reference 2.16). The *Erosion Control Plan-In-Hand Checklist* becomes part of the plan-in-hand package and is used for determining appropriate erosion controls as well as for estimating erosion control costs.

4. EROSION AND SEDIMENT CONTROL PLANS

Information on erosion and sediment control may be shown on:

- Erosion and Sediment Control plan sheets.
- Plan and Profile sheets.
- Summary of Quantities sheet.
- Removal and Construction plan sheets.
- Drainage sheets.

Based on the complexity of the project, the **Roadside Development & Compliance Unit**, along with the roadway designer, will determine how to properly show the erosion and sediment control design on the plans. Minor grading projects, such as overlays, may be able to show all of the pertinent information on existing plan sheets. Most projects, however, will require separate Erosion and Sediment Control plans such as:

- Temporary Erosion and Sediment Control Plan: The Temporary Erosion and Sediment Control Plan is a dynamic document, which will be adjusted throughout the life of the construction project. This plan will be modified on an as-needed basis, depending on the contractor's phasing scheme and on the local conditions during construction. Currently this plan is to be developed by the contractor, based on localized conditions that occur during the day to day construction of a project. The roadway designer is responsible for determining which temporary erosion control measures will be required on a project and for providing a quantity of materials for bidding purposes.
- Permanent Erosion and Sediment Control Plan: The Permanent Erosion and Sediment Control Plan is composed of the required elements to permanently re-stabilize the site after construction is completed. The items specified on this plan are designed to work in conjunction with the permanent seeding of the project. These plans are to be dynamic and may be adjusted in the field based on the project conditions.

5. TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES

Temporary erosion and sediment control measures are for use during construction to maintain the site condition and to prevent off-site erosion and sedimentation. Temporary measures may be used until the permanent erosion control and revegetation measures are established or in conjunction with other permanent erosion control measures. It may be necessary for some temporary measures to be applied on the same site several times over the course of the project.

The roadway designer will be responsible for determining which temporary erosion and sediment control measures will initially be required on the project and for determining the approximate quantities for bidding purposes. It is important to note that this is a dynamic plan and it will change throughout the construction of the project based on the contractors phasing scheme, weather conditions, soil types encountered, etc. The quantities computed by the roadway designer for these items are approximations and will be over-run and under-run based on the actual conditions encountered during construction.

5.A Temporary Hydraulic Control Measures

5.A.1 Temporary Slope Drain

A temporary slope drain is a pipe (aboveground or buried) extending from the top to the bottom of a cut or fill slope. It is used to temporarily transport concentrated stormwater runoff safely down the face of a cut or fill slope without causing erosion on or below the slope. It is essential to protect against the potentially high discharge velocity of water at the outlet by using erosion control blankets, riprap or other measures.

Slope drains should be used on cut or fill slopes where there is a potential for flows to go over the face of the slope causing erosion and preventing adequate stabilization. An example of such a case would be the end of a farming terrace removed during phased construction. The slope drain pipe should be sized according to the parameters established in Chapter One: Drainage, and the current pipe material policy (See Appendix C, “Pipe Material Policy”). A culvert cross section will be required. EXHIBIT 2.2 illustrates an aboveground temporary slope drain.

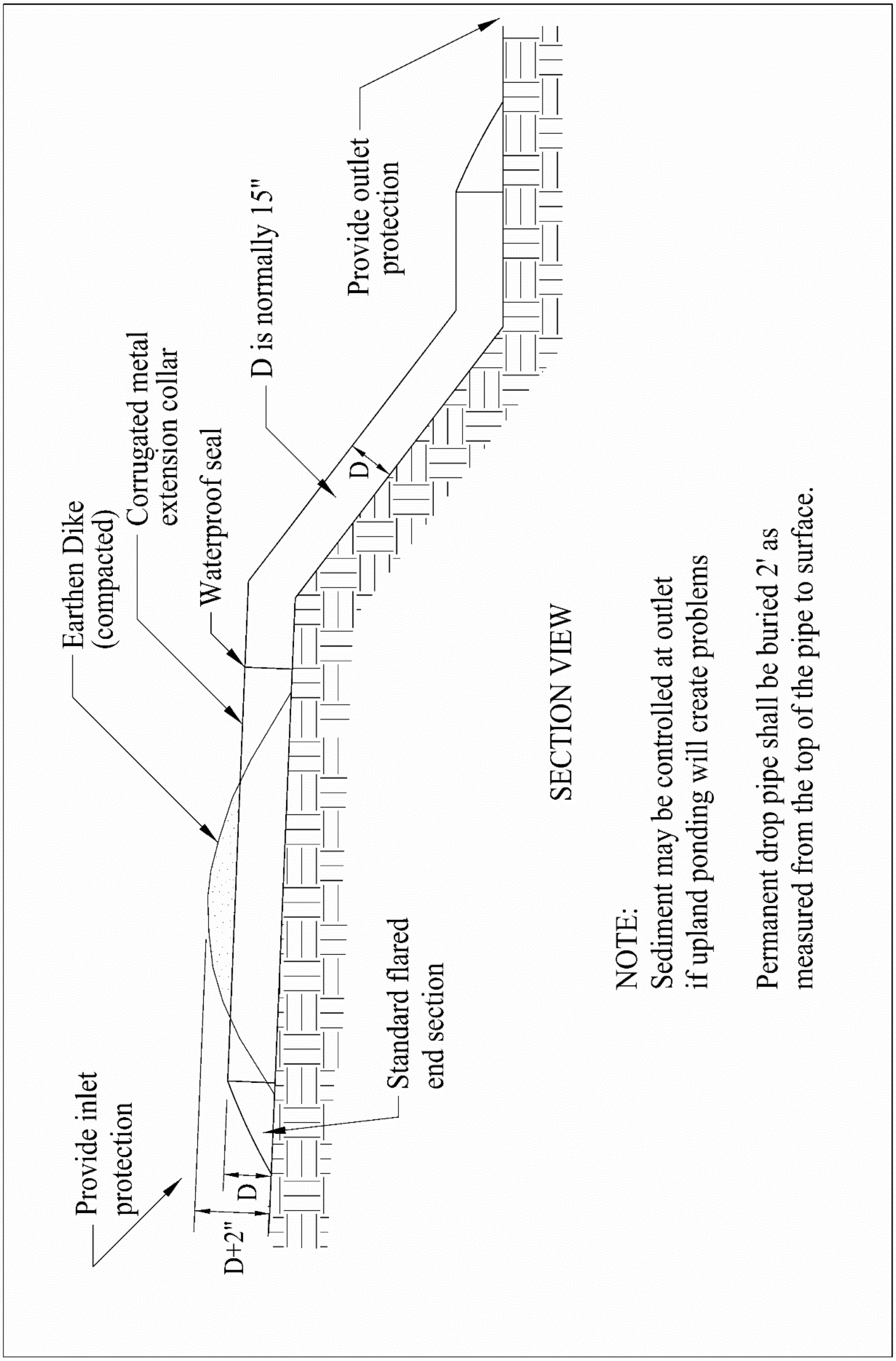


Exhibit 2.2 Temporary Slope Drain

5.B Temporary Erosion Control Measures

5.B.1 Covercrop Seeding

Covercrop Seeding is the establishment of a temporary vegetative cover on disturbed areas with appropriate, rapidly growing annual plants. Covercrops reduce erosion and sedimentation by stabilizing disturbed areas, reduce damage from sedimentation and runoff to downstream or off-site areas, and provide protection to bare soils exposed during construction until permanent vegetation or other erosion control measures can be established.

Covercrop Seeding can be used on surcharge areas, soil stockpiles, dikes, dams, sides of sediment basins, temporary road banks, etc. Covercrop Seeding may also be applied to finish grades as the project progresses to protect the finish grade and reduce erosion. A permanent vegetative cover should be applied as soon as possible upon completion of the finish grading earthwork.

Different species of seed may be used for covercrop during different times of the year. In the spring, for example, oats are planted. Summer seeding may be with foxtail millet or pearl millet. Winter wheat is planted in the fall. Consult the **Roadside Development & Compliance Unit** in the **Project Development Division** for details. Material and application requirements may be found in Section 812 of the Standard Specifications for Highway Construction, (Reference 2.10), (http://roads.nebraska.gov/media/6897/specbook-2007.pdf). Covercrop Seeding should be used in most areas except for where slope protection is required. The area to be covered is measured in acres (hectares) and should be calculated as shown in EXHIBIT 2.4.

Covercrop seeding will be calculated for each phase of the earthwork on projects with phased construction which requires more than one construction season. The 2N sheet will show the quantity of Covercrop Seeding required for each phase, (See the Roadway Design Manual, Chapter Eleven: Plan Preparation, Section 4.G, Reference 2.16).

5.B.2 Temporary Seeding

Temporary Seeding is the establishment of permanent vegetation using perennial grasses for a short duration, usually for more than one growing season, but for periods longer than Covercrop Seeding can protect. Temporary Seeding is generally used in staged construction.

5.B.3 Temporary Mulching

Temporary Mulching is the application of plant residues or other suitable materials to the soil surface. Temporary Mulch is used by itself or in conjunction with Covercrop and Temporary Seeding to provide temporary protection of the soil surface during construction. Temporary Mulching prevents erosion by protecting the soil surface from raindrop impact and by reducing the velocity of overland flow. Temporary Mulching can be used anytime protection of the soil surface is desired.

Temporary Mulching material shall be either dry cured native prairie hay, native grass hay from seed growing operations, native grass hay from planted warm season grass stands, or threshed grain straw (brome hay is not allowed due to its' shallow root structure). Temporary Mulch is applied at the rate of 1.5 tons/acres (3.35 Mg/ha) for hay, 2.0 tons/acre (4.5 Mg/ha) for straw and 2.5 tons/acre (5.6 Mg/ha) for rushes or similar materials. Temporary Mulching material and application requirements may be found in Section 805 of the Standard Specifications for Highway Construction, (Reference 2.10).

5.B.4 Temporary Slope Protection

Temporary Slope Protection is the spreading and crimping of hay on bare soil without seeding. Currently, the use of Temporary Slope Protection is limited to unseeded temporary roads in the Sandhills Region. The hay is used to stabilize the sandy soils and to provide erosion control. The material for Temporary Slope Protection may be hay, straw or rushes and is applied at the rate of 2.0 lbs/sq. yd. (1.1 kg/m^2). Refer to Section 810 of the Standard Specifications for Highway Construction, (Reference 2.10). Slope Protection Netting (See Section 6.B.3) should be included whenever with this item is specified.

The Temporary Slope Protection material may be anchored by whatever methods the contractor deems necessary. Temporary Slope Protection material shall be kept in good repair throughout the life of the construction project. The contractor is responsible for its upkeep, no extra payment will be made for the maintenance and repair of Temporary Slope Protection.

5.B.5 Contour Field Cultivation of Slopes

Contour Field Cultivation of Slopes is a procedure that is used to roughen the foreslope or backslope grade in horizontal strips to reduce the rill erosion common on non-vegetated slopes. The Contour Field Cultivation creates a rough area, 8 to 12 ft. wide (2.5 to 3.7 m), perpendicular to the down hill flow. This cultivated area intercepts the shallow, concentrated rivulets of water and spreads the water over a wider area and back into sheet flow, reducing or eliminating the rill erosion. A field cultivator shall be used to construct the parallel strips, on the contour, at approximately 25 ft. (7.6 m) centers. The initial cultivated strip will be centered no more than 10 ft. (3 m) from the top of the back slope and the final, bottom cultivated strip centered no more than 25 ft. (7.6 m) from the ditch bottom. The cultivation strips will be rough tilled to a depth of 3 to 4 in. deep (75 to 100 mm).The cultivated strips may also be placed on rough graded slopes that will be left exposed for several weeks or on finished grades.

5.C Temporary Sediment Control Measures

5.C.1 Temporary Erosion Checks

Temporary Erosion Checks are barriers placed perpendicular to the flow in ditches with slopes steeper than 3% to slow the velocity of the water, causing silt deposition. Erosion Checks are typically used in ditches where rough grading has been completed but finish grading has yet to begin. The spacing of these products depends on the slope of the ditch; the steeper the slope, the closer together the Temporary Erosion Checks should be spaced (See EXHIBIT 2.9). Temporary Erosion Checks may be used in conjunction with Temporary Silt Traps (See Section 5.C.5) to increase their holding capacity. The products used for Temporary Erosion Checks may occasionally be specified for permanent applications.

Erosion Checks are listed on the Approved Products List, Reference 2.19, (http://www.roads.nebraska.gov/business-center/materials/approved-products/) and any item in this category may be used in a temporary application. The Erosion Checks may be biodegradable or non-biodegradable. At the completion of the project, any non-biodegradable Temporary Erosion Checks must be removed and will remain the property of the Contractor. Any biodegradable Temporary Erosion Checks may be left in place.

5.C.2 Temporary Earth Checks

Temporary Earth Checks are barriers placed perpendicular to the flow in ditches in order to slow the velocity of water, causing silt deposition. Temporary Earth Checks are used in ditches where rough grading has been completed but finish grading has yet to begin. Temporary Earth Checks are constructed of earth and are placed at locations determined by the Contractor or the **District Project Manager**.

Temporary Earth Checks are used on ditches with slopes of 3% or flatter. The steeper the slope, the closer together these checks are placed. Construction consists of building a small earth berm across a ditch to reduce the velocity of water. Temporary Earth Checks can be used in conjunction with Temporary Silt Traps (See Section 5.C.5) on the upstream side of the ditch to increase their sediment holding capacity.

5.C.3 Temporary Rock Checks

Temporary Rock Checks are barriers placed in ditches, perpendicular to the flow, to slow the velocity of the water flow, which causes silt deposition. Temporary Rock Checks are used in ditches where rough grading has been completed but finish grading has not yet begun. Temporary Rock Checks are constructed of rock and are placed at locations determined by the **District Project Manager** or the Contractor.

Temporary Rock Checks are used on steeper ditch slopes, where riprap will be placed as a Permanent Hydraulic Control. The steeper the ditch, the closer the checks are placed to each other. Construction consists of building a small rock berm across the ditch to reduce the velocity of water with rock that is already specified (for riprap construction). Temporary Rock Checks can be used in conjunction with Temporary Silt Traps (See Section 5.C.5) on the upstream side of the ditch to increase the sediment holding capacity.

5.C.4 Temporary Silt Fence

Temporary Silt Fence is a barrier that can be placed at the toe of the slope, across ditches, or in any location where silt may have the opportunity to leave the Right-of-Way. The majority of Silt Fence locations will be specified with the permanent erosion control. Temporary Silt Fence is to be used on locations *not* identified with the permanent erosion control.

Temporary Silt Fence may consist of any product on the Approved Products List (Reference 2.19) for Low Porosity or High Porosity Silt Fence, or from any other commercially available source. The silt fence material shall be a minimum of 36 in. (900 mm) in height. When Temporary Silt Fence is being used across a ditch, High Porosity Silt Fence should be used.

5.C.5 Temporary Silt Trap

A Silt Trap is a temporary ponding area formed by excavating a basin along the path of the water flow. A Silt Trap may be used alone or in conjunction with a Silt Fence, Erosion Control products and/or Erosion Checks. A Silt Trap is normally 1 ft. (300 mm) deep and 6 ft. (1.8 m) wide with slopes into and out of the depression. Silt Traps generally do not include outlet drains; trapped water either evaporates or percolates into the ground.

6. PERMANENT EROSION AND SEDIMENT CONTROL MEASURES

Permanent erosion and sediment control measures are for use during and after construction of the roadway. Permanent measures are designed to limit the amount of erosion that occurs over the life of a project and to maintain the shape of the final embankments, ditches, and channels. Permanent erosion and sediment control measures are selected based on the project design storm and should withstand the majority of storms, (up to and including the design storm), with only minor maintenance. Some sediment control measures, (such as Sediment Basins), will require periodic maintenance to maintain their effectiveness.

The selection of a permanent erosion and sediment control measure is based on the following:

- Location of installation (urban, rural, rest stop, recreation area, etc.).
- Economic analysis of suitable alternatives.
- Principles of agronomy.
- Site-specific requirements.
- Availability of construction materials.
- Future maintenance requirements.
- Wetlands protection.

In addition to the above parameters, the erosion control and sediment measure selected must be able to withstand the expected erosive conditions encountered on the project. One gauge of acceptability is the permissible velocity for water flow over the erosion control measure. EXHIBIT 2.3 shows permissible velocities for various channel linings. It is important to remember that these are maximum velocities developed for flows of short duration. Flows that exceed these values will damage the channel lining and cause erosion.

For additional information, refer to the following **FHWA** publications:

- Hydraulic Design Series No. 4: Design of Roadside Drainage Channels, (Reference 2.7), (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds4.pdf).
- Hydraulic Engineering Circular No. 11: Design of Riprap Revetment, (Reference 2.8), (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec11sI.pdf).
- Hydraulic Design Series No. 3: Design Charts for Open Channel Flow, (Reference 2.9), (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds3.pdf).

STRUCTURALLY-LINED CHANNELS		
Lining Material	**Permissible Velocity * fps (m/s)**	
Cellular Confinement Product ● Soil and Vegetative Fill ● Granular Fill ● Concrete Fill	*Consult Manufacturer*	
Riprap (See Section 7.A and EXHIBIT 2.13)	*10*	*(3.0)*
Gabion Baskets and Revet (Reno) Mattress	*15*	*(4.5)*
Concrete Lining or Interlocking Paver Blocks	*Greater than 15*	*(4.5)*

** Designer should confirm maximum permissible velocity with manufacturer.*

Exhibit 2.3a Permissible Velocity for Structurally-Lined Channels

NON-VEGETATED CHANNELS			
Soil Type	**Channel Slope**	**Permissible Velocity * fps (m/s)**	
Sand	*0-1%*	*2.5*	*(0.8)*
	1-5%	*2.0*	*(0.6)*
	over 5%	*Consider Other Channel Liner Material*	
Silt	*0-3%*	*2.5*	*(0.8)*
	over 3%	*2.0*	*(0.6)*
Loam	*0-1%*	*4.0*	*(1.2)*
	1-3%	*3.5*	*(1.1)*
	Over 3%	*3.0*	*(0.9)*
Clay Loam	*0-2%*	*4.5*	*(1.4)*
	2-5%	*4.0*	*(1.2)*
	>5%	*3.5*	*(1.1)*
Clay	*0-2%*	*5.5*	*(1.7)*
	2-5%	*5.0*	*(1.5)*
	Over 5%	*4.5*	*(1.4)*

** Designer should confirm maximum permissible velocity with manufacturer.*

Exhibit 2.3b Permissible Velocity for Non-Vegetated Channels

6.A Permanent Erosion Control Measures

Vegetation plays an important role in controlling erosion. Vegetation shields the soil surface from the impact of falling rain, reducing one of the primary methods of soil detachment. It holds soil particles in place through its root structure. Decaying vegetation increases the organic matter of the soil, which along with the living vegetation's root structure improves the soil's capacity to absorb water. A good cover of vegetation can reduce the amount and slow the velocity of stormwater runoff.

Vegetation is the preferred choice in erosion control material for the following reasons:

- It is cost effective.
- It will adjust to nearly all changes in the embankment or channel geometry.
- It will filter sediment and other contaminants from the runoff.
- Local damage or loss is self-healing.
- The appearance is natural and generally pleasing.

6.A.1 Seeding

Seeding is the primary method used to provide a permanent vegetative cover to protect against erosion. Permanent seeding is generally initiated at the completion of the project. Types of permanent seeding include:

- Type A Seeding: Permanent placement of seed on the foreslope, ditch, and backslope measured in acres (hectares).
- Type B Seeding: Includes shorter plant varieties that can withstand frequent mowing. Type B Seeding is the permanent placement of seed on the shoulder area and in the median, measured in acres (hectares).
- Type C Seeding: Other seed mixtures as needed for special placement.

See EXHIBIT 2.4 for guidance in the calculation of seeding areas.

6.A.2 Mulching

Mulching is the application of plant residues or other suitable materials to the soil surface. Mulch is always used in conjunction with seeding in order to establish vegetation. Mulch helps foster the growth of vegetation by increasing available moisture and by providing insulation against extreme heat and cold. Mulching helps prevent erosion by protecting the soil surface from raindrop impact and by reducing the velocity of overland flow. Mulching can be used anytime protection of the soil surface is desired.

Mulching material shall be either dry cured native prairie hay, native grass hay from seed growing operations, native grass hay from planted warm season grass stands, or threshed grain straw. Brome hay is not allowed. The mulch is applied at the rate of 2 tons/acre (4.5 Mg/ha) for hay and 2.25 ton/acre (5.0 Mg/ha) for straw. Mulching material and application requirements may be found in Section 805 of the Standard Specifications for Highway Construction, (Reference 2.10).

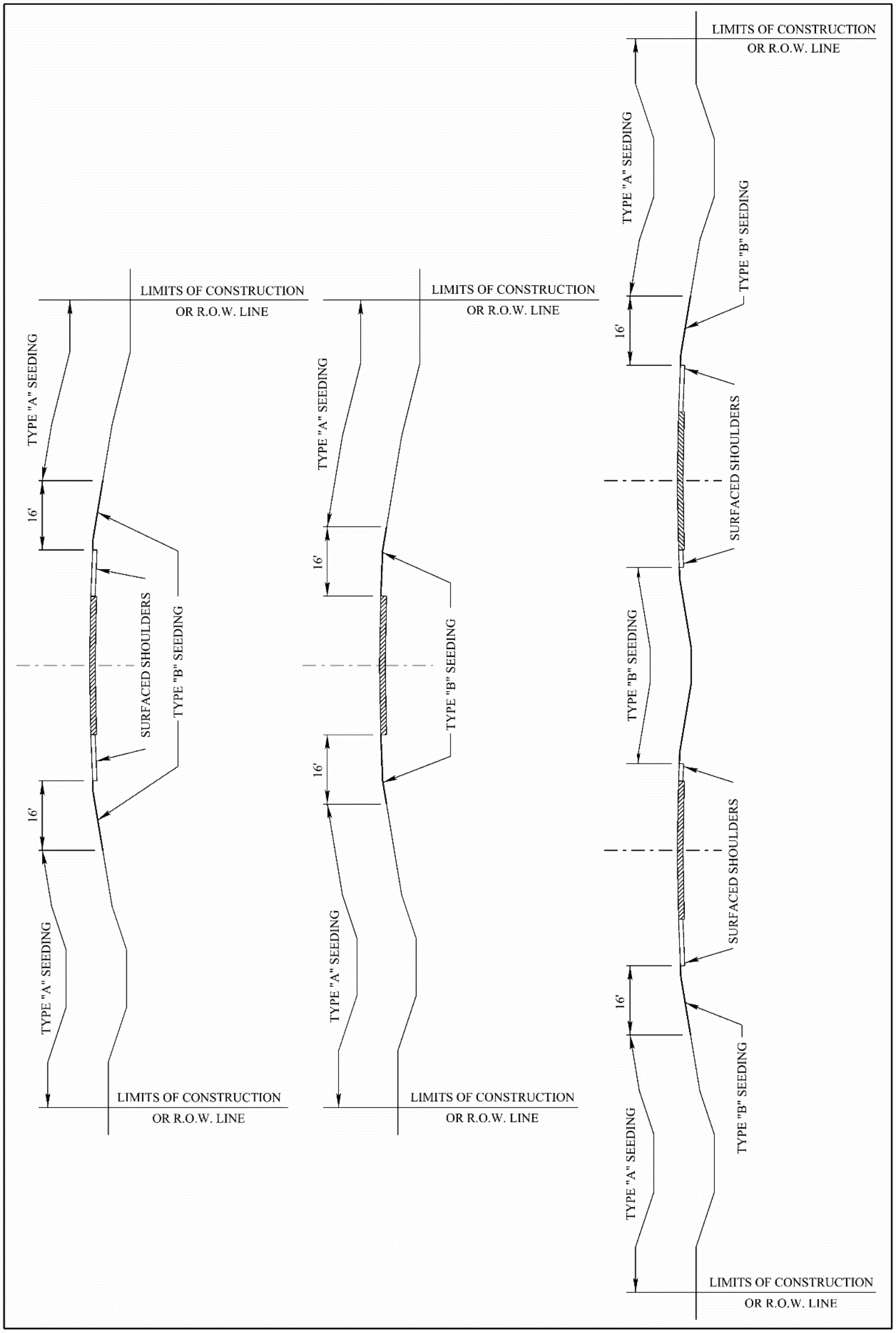


Exhibit 2.4a Seeding Computations

Compute:

Type “B” Seeding

(32 ft. + (Median Width – Inside Surfaced Shoulder Widths)) x Length of Project ÷ 43,560 ft./acre = _____ acres

Or in metric:

(9.8 m + (Median Width – Inside Surfaced Shoulder Widths)) x Length of Project ÷ 10,000 m^2/hectare = _____ hectare

Type “A” Seeding

For Projects on Existing or Shifted Alignment:

(Total area between the L.O.C.’s – Surfacing Area – Type “B” Seeding) x *115% = _____ acres (hectares)

For Projects on New Alignment:

(Total R.O.W. Taking – Surfacing Area – Type “B” Seeding) x **90% = _____ acres (hectares)

* The 115% factor includes the area disturbed by the contractor which lies beyond the limits of construction and will require seeding.
** This assumes that 10% of the total ROW area will not be disturbed during construction and will not require seeding. Use a factor of 100% when calculating seeding through cropland and any other areas which will require seeding to the ROW limits.

Covercrop Seeding:

Type “B” Seeding Area + Type “A” Seeding Area = _____ acres (hectares)

Notes:

1. Seeding should be computed on all projects.
2. Wetland seeding is not included in the above calculations.
3. In the case of a shifted alignment the designer shall ensure that the seeding computations include the area of surfacing removed from the existing alignment which lies outside of the limits of construction.
4. The total R.O.W. taking shown on the R.O.W. Plans may or may not be the correct area to be used in seeding calculations. The designer should check with the R.O.W. designer to determine if the taking area shown on the R.O.W. Plans is to be used.
5. The method used to calculate the Type “A” Seeding (and the appropriate percentage factor) should be noted on the Summary of Quantities Sheet, (See the Roadway Design Manual, Chapter Eleven: Highway Plans Assembly, Section 4.C, Reference 2.16).

Exhibit 2.4b Seeding Computations

6.A.3 Slope Protection

Slope Protection is the spreading and crimping of hay on bare soil in conjunction with seeding. Slope Protection is used in the Sandhills Region and in other areas with non-cohesive soils. The hay is used to stabilize the sandy soils, provide erosion control, and to establish a protective cover that promotes seed germination and the growth of vegetation. Covercrop Seeding is not required in areas with Slope Protection.

Slope Protection covers the area of disturbed soil that is not surfaced and is measured and paid for in sq. yds. (m^2). The material for Slope Protection may be hay, straw, or rushes and is applied at the rate of 2.0 lbs./sq. yd. (1.1 kg/m^2), (Refer to Section 810 of the Standard Specifications for Highway Construction, Reference 2.10). Slope Protection Netting (See Section 6.B.3) should be included whenever this item is specified.

6.A.4 Contour Field Cultivation of Backslopes

Contour Field Cultivation of backslopes is a procedure that is used to roughen the backslope finish grade in parallel strips to reduce the rill erosion common on non-vegetated slopes. The Contour Field Cultivation creates a rough area, 8 to 12 ft. (2.5 to 3.7 m) wide, perpendicular to the down hill flow. This cultivated area intercepts the shallow, concentrated rivulets of water and spreads their water over a wider area and back into sheet flow, reducing or eliminating the rill erosion. A field cultivator shall be used to construct the parallel (on the contour) cultivation strips at approximately 25 ft. (7.6 m) centers. The initial cultivated strip will be centered no more than 10 ft. (3 m) from the top of the backslope and the final, bottom, cultivated strip will be centered no more than 25 ft. (7.6 m) from the ditch bottom. The cultivation strips will be rough tilled to a depth of 3 to 4 in (75 to 100 mm).

The cultivated strips shall be completed as soon as the finish grade for the backslope has been established. Cultivated strips may also be placed on rough graded slopes that will be left exposed for several weeks.

Contour Field Cultivation of backslopes is paid for by the lin. ft. (m).

6.A.5 Sodding

Sodding is the transplanting of grasses. Sodding is done mostly in urban areas and is limited to occupied residential property and business sites. Sod usually comes in rolls, but sometimes slabs and plugs are used. Vacant lots are normally seeded.

Four types of grass are typically used for Sodding:

- Bluegrass: Primarily used on urban projects to match existing lawn conditions where a property owner is available to care for the grass after installation. Occasionally used on rural projects at the request of the property owner. Bluegrass Sodding shall be performed only when weather conditions are favorable. Bluegrass comes in rolls or slabs.
- Fescue: Primarily used on urban projects to match existing lawn conditions where a property owner is available to care for the grass after installation and occasionally used on rural projects at the request of the property owner. Fescue comes in rolls or slabs.
- Buffalograss: Buffalograss is used in medians in urban areas and comes in plugs or slabs.
- Zoysia: Zoysia is planted to match the existing plant material and comes in plugs.

Sodding will not survive without extended care. Sodding is measured and paid for in sq. yd. (m^2) of sod placed.

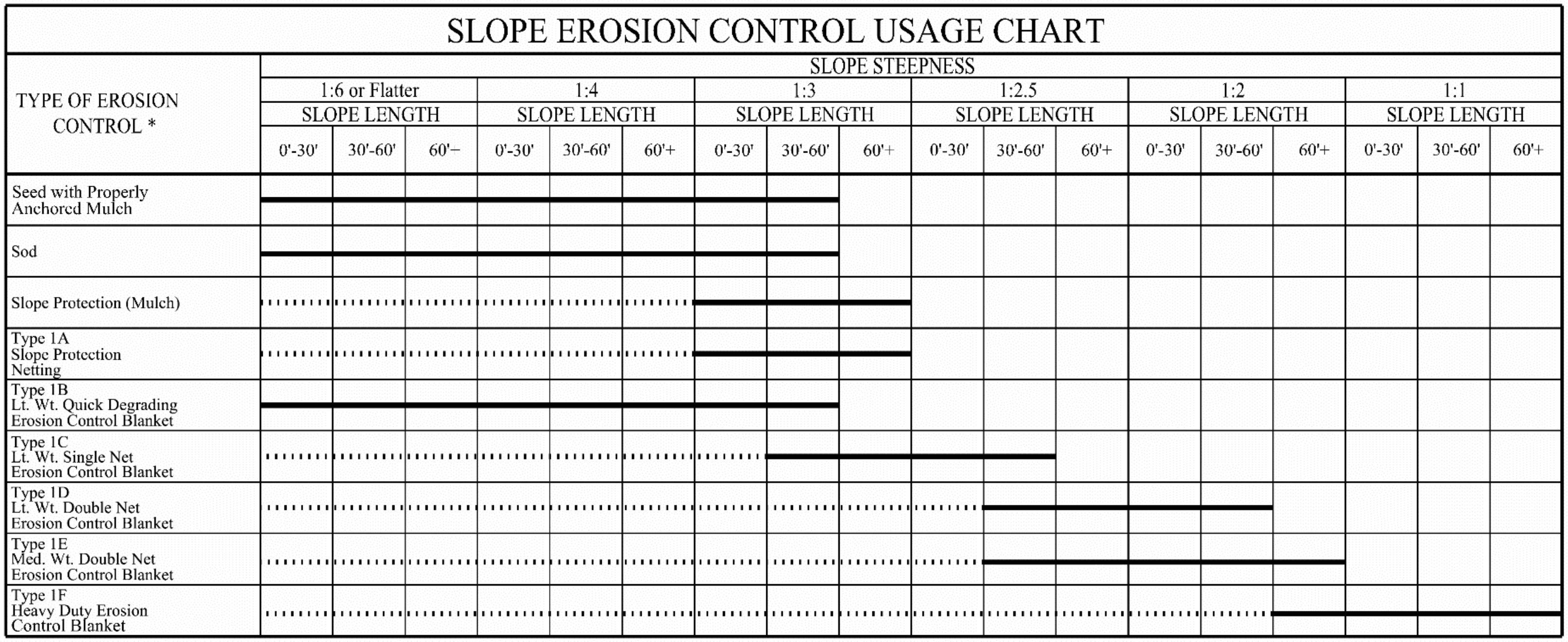
6.A.6 Erosion Control "Type_" Products

The various Erosion Control products are protective blankets or Turf Reinforcement that may be installed on prepared planting areas of steep slopes, channels or shorelines. Erosion Control Blankets and Turf Reinforcement Mats control erosion in critical areas by providing a microclimate that protects young vegetation and promotes its establishment. Some types of Erosion Control Blankets and Turf Control Mats also raise the maximum permissible water velocity on or across turf grass stands in channelized areas by reinforcing the turf to resist the forces of erosion during storm events.

Erosion Control Blankets are typically used on steep slopes (1:3 or steeper) where erosion hazard is high and vegetation growth is likely to be too slow to provide adequate protective cover. Erosion Control Blankets should be used in conjunction with Erosion Checks and may also be used in vegetated ditches with slopes of up to 3%. These blankets are composed primarily of straw or coconut and are selected based on the steepness of the slope and the longevity requirements of the product.

Turf Reinforcement Mats are used around splash basins of flumes or along flumes. They are also used on vegetated ditches and channels where the velocity of design flow exceeds the allowable velocity. Erosion Control Blankets or Turf Reinforcement Mats may be used where moving water is likely to wash out new plantings or in areas where the forces of wind prevent standard mulching practices from remaining in place until vegetation becomes established.

There are numerous erosion control products on the market (See EXHIBITS 2.5 THROUGH 2.9). See the Approved Products List (Reference 2.19) for a complete listing of all Erosion Control Products approved for use on **NDOT's** construction projects. The roadway designer should use special provisions to specify any brand name products recommended by the **Roadside Development & Compliance Unit**.



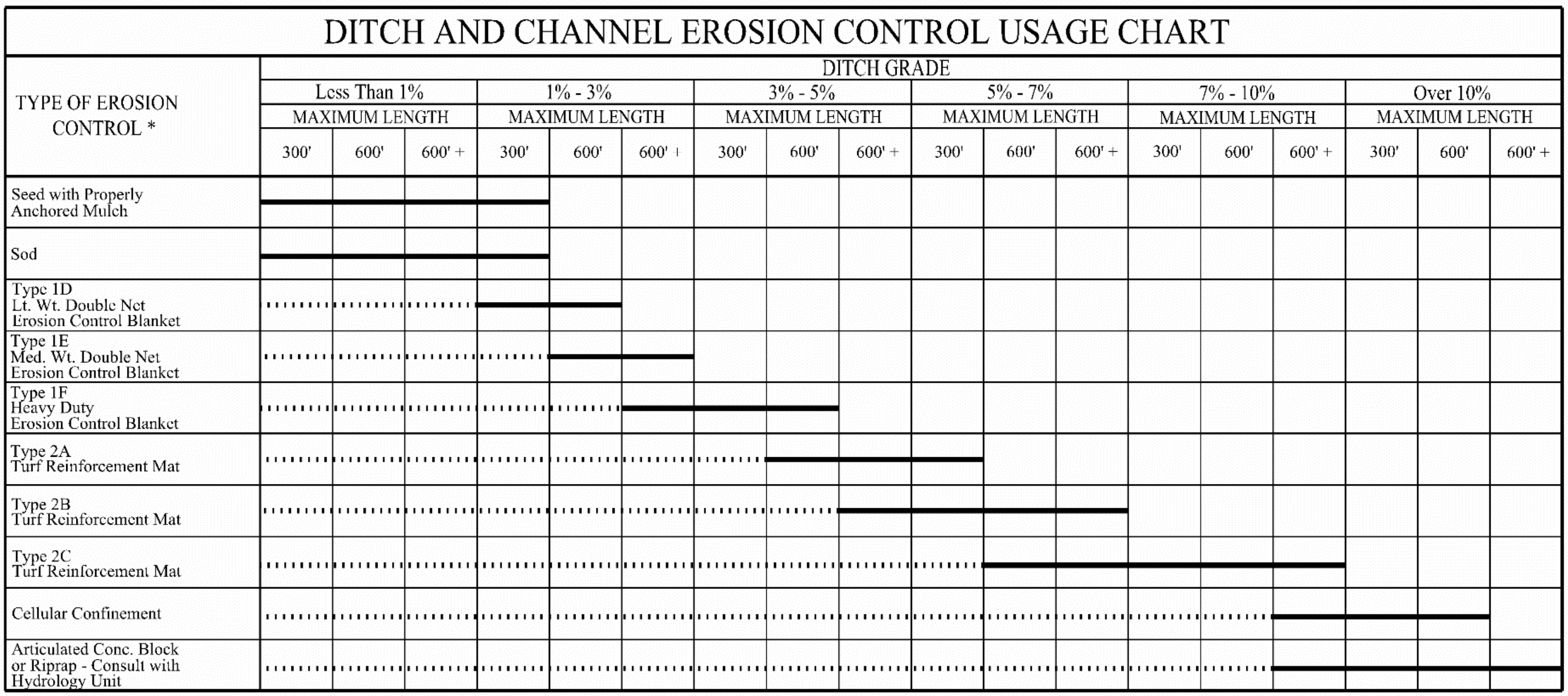
* For a description of the physical properties, see Exhibit 2.7.

Designates instances where a particular Erosion Control Type will be used

Designates instances where a particular Erosion Control Type may be used

Rill and gully erosion on side slopes is the primary concern when designing slope erosion control. When unprotected, the slopes will erode. Rills and gullies provide channels that further concentrate runoff and greatly increase the rate at which sediment is removed from the slopes. Once formed, they can become costly t correct and dangerous for our maintenance crews. Seeding and Mulching is the primary method of slope erosion control. However, Rolled Erosion Control Products (RECP's) are used based on aesthetic considerations, severity of the slopes, and soil types as well as cost.

Exhibit 2.5 Slope Erosion Control Usage Chart



* For a description of the physical properties, see Exhibits 2.7 and 2.8.

Designates instances where a particular Erosion Control Type will be used

Designates instances where a particular Erosion Control Type may be used

Ditches and channels on the Right-of-Way carry water from the roadway, the side slopes, as well as runoff from adjacent properties. The energy produced during times of flow, based upon channel length and grade, soil type, and vegitative cover will affect the channel. The concentrated water can create gullies, some of which may continue up gradient and deepen enough to destabilize the side slope and threaten the roadway. In many instances, grasses, once established, can be sufficient to stabilize the ditch. However, as the lengths and grades become greater, the ditches require synthetic reinforcement of the grass to maintain the ditch. The overall philosophy of ditch and channel erosion control is to find the most economical solution over the long term while minimizing the amount of erosion occuring on the Right-of-Way.

Exhibit 2.6 Ditch and Channel Erosion Control Usage Chart

ROLLED EROSION CONTROL PRODUCT PHYSICAL PROPERTIES SPECIFICATION CHART

PRODUCT TYPE	PRODUCT DESCRIPTION	MATERIAL COMPOSITION	FUNCTIONAL LONGEVITY	BLANKET SIZE		ACCEPTABLE MATRIX FILL MATERIAL	PAY ITEM	STD. PLAN NO.
				Minimum Roll Width	Minimum Thickness ASTM D 6525			
DEGRADABLE BLANKETS								
1A	Slope Protection Netting	A photodegradable synthetic mesh or woven biodegradable natural fiber netting.	12 Months	6.5 ft.	N/A	N/A	SY	5010
1B	Lt. Wt. Quick Degrading Erosion Control Blanket	Processed degradable natural and/or polymer fibers mechanically bound by a single rapidly degrading synthetic or natural fiber netting.	3 Months	4.0 ft.	0.30 in.	Straw or Excelsior	SY	5010
1C	Lt. Wt. Single Net Erosion Control Blanket	Processed degradable natural fibers mechanically bound together by a single degradable synthetic or natural fiber netting.	12 Months	6.5 ft.	0.30 in.	Straw or Excelsior	SY	5010
1D	Lt. Wt. Double Net Erosion Control Blanket	Processed degradable natural fibers mechanically bound together between two degradable synthetic or natural fiber nettings.	12 Months	6.5 ft.	0.30 in.	Straw or Excelsior	SY	5010
1E	Med. Wt. Double Net Erosion Control Blanket	An erosion control blanket composed of degradable natural fibers and/or processed slow degrading natural fibers mechanically bound together between two slow degrading synthetic or natural fiber nettings to form a continuous matrix.	24 Months	6.5 ft.	0.30 in.	Straw, Excelsior, or Coconut Fibers	SY	5010
1F	Heavy Duty Erosion Control Blanket	An erosion control blanket composed of degradable natural fibers and/or processed slow degrading natural fibers mechanically bound together between two slow degrading synthetic or natural fiber nettings to form a continuous matrix.	36 Months	6.5 ft.	0.30 in.	Excelsior or Coconut Fibers	SY	5010

The information in this table has been derived from information obtained from the Erosion Control Technology Council and from the characteristics of products currently on the NDOT Approved Products list.

Exhibit 2.7 Rolled Erosion Control Product Properties Degradable Blankets

ROLLED EROSION CONTROL PRODUCT PHYSICAL PROPERTIES SPECIFICATION CHART

PRODUCT TYPE	PRODUCT DESCRIPTION	MATERIAL COMPOSITION	BLANKET SIZE		ACCEPTABLE MATRIX FILL MATERIAL	PAY ITEM	STD. PLAN NO.
			Minimum Roll Width	Minimum Thickness ASTM D 6525			
LONG TERM NON-DEGRADABLE CHANNEL APPLICATIONS							
2A	Turf Reinforcement Mat	Long term, non-degradable rolled erosion control product composed of UV stabilized, non-degradable synthetic fibers, filaments, nettings and/or wire mesh processed into three dimensional reinforcement matrices designed for permanent and critical hydraulic applications where design discharges exert velocities and shear stresses that exceed the limits of mature, natural vegetation. Turf reinforcement mats provide sufficient thickness, strength and void space to permit soil filling and/or retention and the development of vegetation within the matrix.	6.5 ft.	0.25 in.	Excelsior, Coconut, or Polymer Fibers.	SY	5010
2B	Turf Reinforcement Mat		6.5 ft.	0.50 in.	100% UV Stabilized Polypropylene Fibers	SY	5010
2C	Turf Reinforcement Mat		6.5 ft.	0.50 in.	100% UV Stabilized Polypropylene Fibers	SY	5010

The information in this table has been derived from information obtained from the Erosion Control Technology Council and from the characteristics of products currently on the NDOT Approved Products list.

Exhibit 2.8 Rolled Erosion Control Product Properties
Long Term Non-Degradable Channel Applications

6.B Permanent Sediment Control Measures

Sediment Control Measures are required with the Permanent Erosion Control to aid in:

- Keeping the soil on the site until final stabilization occurs.
- Reduce the amount of erosion on-site.
- Reduce the long-term maintenance costs.

As with the Permanent Erosion Control, there are numerous techniques and products available to aid in controlling sediment after construction is complete.

6.B.1 Erosion Checks

Erosion Checks are hay or straw bales placed in ditches at predetermined intervals to slow the velocity of water and cause silt deposition. Erosion Checks may also be used in conjunction with an Erosion Check-Silt Trap (ST), (See Section 5.C.5). The silt trap versions are used when the expected silt loading will be greater than normal due to sizeable cuts, soil type, etc. The steeper the grade, the closer together the erosion checks should be placed. The designer should refer to EXHIBIT 2.9 for preliminary guidelines regarding placement and spacing of Erosion Checks for the sediment control requirements of their project. The **Roadside Development & Compliance Unit** will review the final Erosion Check locations and intervals for each project.

There are several types of Erosion Checks listed on the Approved Products List (Reference 2.19). Typically, they are used in ditches where the rough grading is complete and before the finish grading is done, however there can be situations where these are used in a permanent capacity.

The roadway designer needs to provide details and calculate the number of bales that will be needed per Erosion Check installation. The Erosion Check design shall extend up the foreslope and backslope of the ditch to effectively contain runoff and to prevent erosion and washout at the edges of the Erosion Check. Erosion Check-ST includes a silt trap on the upstream side of the hay bales. Bale dimensions are typically 14-16 in. H x 18 in. W x 36 in. L (360-400 mm H x 450 mm W x 900 mm L) and are placed in an 6 in. (150 mm) deep trench. See the Erosion Check – Type A, -Type HV and –Type ST Special Plans and the Silt Checks Special Plans in the Standard/Special Plans Book, (Reference 2.12), for further details. EXHIBIT 2.10 illustrates various types of erosion checks.

When used in a ditch, Erosion Checks are typically spaced on 25 ft. (7.6 m), 50 ft. (15.2 m), 75 ft. (22.9 m), or 100 ft. (30.5 m) increments based on the severity of the ditch slope and any local conditions.

EROSION CHECK USAGE CHART					
Bid Item	***Product***	***Usage***	***Ditch Grade***	***Placement Guidelines***	***Required Plans (Standard/Special Plans Book)***
Erosion Check	*Includes: 3 ft., (1 m) Bale, Filter Fabric, and Erosion Control Blanket*	*Used for silt control and to reduce the velocity of water in ditches. Type must match the EC in the ditch.*	*2%*	*100 ft. (30 m) on center*	*Standard Plan 5100 1 E/M*
			3-4%	*50-75ft. (15-23) on center*	
			4-6%	*50-75 ft. (15-23 m) on center*	
			6-9%	*50 ft. (15 m) on center*	
Erosion Check – ST (with silt trap)	*Includes: 3 ft., (1 m) Bale, Filter Fabric, Erosion Control Blanket, and Silt Trap*	*Used for an expected high silt load. Type must match the EC in the ditch.*	*2%*	*100 ft. (30 m) on center*	*Special Plan 5100 1 E/M*
			3-4%	*75-100 ft. (23-30 m) on center*	
			4-6%	*50-75 ft. (15-23 m) on center*	
			6-9%	*50 ft. (15 m) on center*	
Temporary Silt Check	*Includes: 7 ft., (2.1 m) Triangular Foam Dike, and Filter Fabric & Other Products*	*Used as a temporary silt retention device while the project is under construction.*	*2%*	*150 ft. (45 m) on center*	*Special Plan 5108 1 E/M*
			3-4%	*100 ft. (30 m) on center*	
			4-6%	*75 ft. (23 m) on center*	
			6-9%	*50 ft. (15 m) on center*	

Exhibit 2.9 Erosion Check Usage Chart

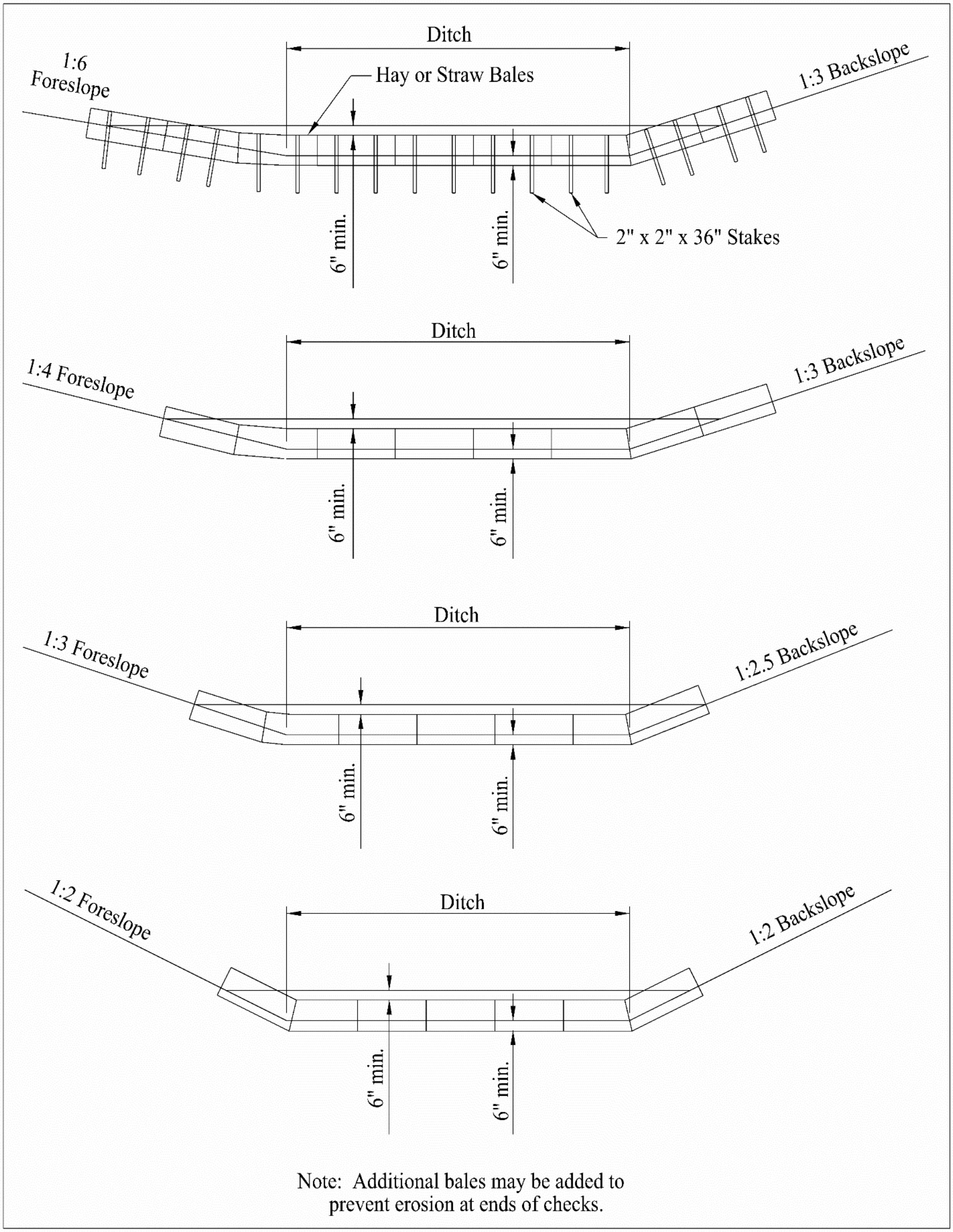


Exhibit 2.10 Erosion Checks

6.B.2 Silt Fence

A Silt Fence is a sediment barrier consisting of synthetic or natural fabric. The fabric is stretched across and attached to supporting posts. The fabric also is entrenched, (See the Fabric Silt Fence Details in the Standard/Special Plans Book, Reference 2.12). Silt Fence is typically used to intercept and retain small amounts of sediment from disturbed areas of limited extent in order to prevent sediment from leaving the construction site. It essentially filters waterborne soil from the water before it leaves the project site. Silt Fence can be used around area inlets, other drainage structures, and on steep and high slopes. High Porosity Silt Fence can be used in ditches to help slow the velocity of the water, allowing soil particles to drop out.

The Silt Fence fabric may be either high porosity or low porosity, and either full height or low profile. Coir Silt Fence is an organic biodegradable fabric made from coconut fibers. Coir Silt Fence, at regular height and high porosity, may be used in wetlands applications or in areas where the functional life of the Silt Fence will need to meet or exceed two years. Silt Fence is sometimes used with a silt trap on the upstream side to prolong the life of the fence, (See Section 5.C.5). EXHIBIT 2.11 lists the different types of silt fences.

Silt Fence is used on most projects, especially on projects with adjoining wetlands, and may be used to control erosion on side slopes. The roadway designer should establish locations where Silt Fence will be required. The **Roadside Development & Compliance Unit** will provide assistance for preliminary and final Silt Fence placement for specific situations and will aid in determining the appropriate Silt Fence type.

Silt Fence is measured and paid for by the lin. ft. (m).

SILT FENCE USAGE CHART

Silt Fence Bid Item	*Product Usage*	*Required Plans*
Low Porosity	*Low flow situations*	*5700 1 E/M*
High Porosity	*High flow areas - mostly rural, some urban*	
Combination of low and high porosity	*Area inlets and urban high flow areas*	
Low profile, low or high porosity	*Low flow structure*	
Combination of low profile, low and high porosity	*Median drains, area inlets*	
Low Profile Coir Fiber	*Wetland protection – low silt load*	
Coir Fiber	*Wetland protection*	

Exhibit 2.11 Silt Fence Usage Chart

6.B.3 Slope Protection Netting

Slope Protection Netting, a photo-degradable lightweight flexible netting, is used over Slope Protection (See Section 6.A.3) to protect the slope protection material from excessive loss due to wind. Slope Protection Netting is typically installed in the sandhills on the east and south sides of high fills, see Standard Plan 5010 in the Standard/Special Plans Book, (Reference 2.12), and Section 811 of the Standard Specifications for Highway Construction, (Reference 2.10).

Slope Protection Netting is measured and paid for in sq. yds. (m^2), (seeding quantities are also calculated for this same area, see Section 6.A.1).

7. PERMANENT HYDRAULIC CONTROL MEASURES

7.A Riprap

Riprap is a layer, facing or protective lining of stones placed to prevent erosion, scour or sloughing. Filter fabric is installed beneath the stone in most applications. Riprap can be used for several different applications including:

- Ditch Lining.
- Lining of channel banks.
- Protection of highway embankments.
- Energy dissipation at culvert outlets.

Riprap, as discussed in this section, is limited to dumped riprap including rock and broken concrete. Rock riprap shall be sandstone, limestone, quartzite or other hard stone, clean and free of earth, clay or refuse (See Section 905 of the Standard Specifications for Highway Construction, Reference 2.10). Broken concrete shall be sized appropriately and placed as specified in Section 906 of Reference 2.10. Each load shall be reasonably well graded from the largest to the smallest size specified. The designer should refer to Hydraulic Engineering Circular No. 11, Design of Riprap Revetment, (Reference 2.8) for discussion of hand-placed riprap, grouted riprap, sacked concrete riprap, and broken concrete riprap.

Dumped riprap is graded rock or stone dumped on a prepared slope in such a manner that segregation will not take place. Dumped riprap has several advantages including:

- The lining is flexible and can adjust to foundation settlement.
- The riprap is free draining, which eliminates hydrostatic pressure problems associated with rigid linings.
- Local damage or loss is easily repaired by the placement of additional rock.
- The appearance is natural and generally pleasing.

Riprap can be an effective erosion resistant lining, however, it is susceptible to damage in the following ways:

- The displacement of individual stones by the forces of water.
- The loss of foundation stability by movement of the underlying soil through the riprap layer when no filter layer is placed or is placed improperly.
- Displacement of the entire mass when the filter fabric acts as a sheer plane on steep slopes.

The resistance of the stone to displacement by moving water depends on:

- The weight, size, and shape of the individual stone.
- The gradation of the stone.
- The depth of water over the stone lining (including buoyancy forces).
- The steepness of the protected slope.
- The stability and efficiency of the filter blanket and the embankment on which the stone is placed.
- The velocity of the flowing water against the stone.

7.A.1 Sizing Riprap

The design method presented in this section is based on the concept of maximum permissible tractive force. The tractive force approach focuses on stresses developed at the boundary between flowing water and the materials forming the channel boundary. The size of rock riprap required can be determined by computing the shear stress on the channel at the design discharge and comparing the calculated shear stress to the permissible value for the size of stone selected.

7.A.2 Tractive Force Theory

The hydrodynamic force of water flowing in a channel is known as the tractive force. The basis for stable channel design with rock riprap is that the flow-induced tractive force should not exceed the permissible or critical shear stress of the lining materials. In a uniform flow, the tractive force is equal to the effective component of the gravitational force acting on the body of water, parallel to the channel bottom. The average tractive force, or shear stress, on the channel is equal to:

$$\tau = \gamma RS \qquad \text{Eq. 2.1}$$

Where: τ = Average tractive force shear stress (lbs./sq. ft.);
γ = Unit weight of water (62.4 lbs./cu. ft.);
R = Hydraulic radius (ft.);
S = Average bed slope or energy slope.

Studies have shown that shear stress in channels is not uniformly distributed along the wetted perimeter, and that the maximum average shear stress, τ_{max}, for a straight channel occurs on the channel bed, where the maximum flow depth occurs.

The tractive force method is applicable over a wide range of channel slopes and channel shapes. However, channels with extremely steep slopes, (S greater than 0.25 ft./ft.), and channels with steep side slopes (steeper than 1:3) should use the modified tractive force method provided in Hydraulic Engineering Circular No. 15: Design of Roadside Channels with Flexible Linings; (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf), (Reference 2.18).

7.A.3 Permissible Shear Stress

The permissible shear stress, τ_p, indicates the force required to initiate movement of the lining material. Prior to movement of the lining, the underlying soil is relatively protected. Therefore permissible shear stress is not significantly affected by the erodibility of the underlying soil.

Values for permissible shear stress for ditch and channel linings are based on research conducted at laboratory facilities and in the field. EXHIBIT 2.12 presents permissible shear stress values for riprap lining type.

PERMISSIBLE SHEAR STRESS FOR RIPRAP		
Riprap Type	*Mean Riprap Stone Size, D_{50} (ft.)*	*Permissible Shear Stress, τ_p (lbs./sq. ft.)*
Rock Riprap Type A	*0.77*	*3.08*
Rock Riprap Type B	*1.02*	*4.08*
Broken Concrete	*1.10*	*4.40*
Rock Riprap Type C	*1.28*	*5.12*

Exhibit 2.12 Permissible Shear Stress for Riprap

The permissible shear stress for non-cohesive soils is a function of mean diameter of the channel material. For other stone sizes, the permissible shear stress is given by the following equation:

$$\tau_p = (4.0 \text{ lbs./cu. ft.}) \times D_{50} \div SF \qquad \text{Eq. 2.2}$$

Where: τ_p = Permissible tractive force shear stress;
4.0 lbs./cu. ft. = A constant provided by **FHWA**;
D_{50} = The mean riprap stone size in ft.;
SF = Safety Factor (usually 1.1).

7.A.4 Riprap Size

The basic comparison required in the design procedure is that of permissible to computed shear stress for the riprap. If the permissible shear stress is greater than the computed shear, the riprap size is considered acceptable. Channels lined with riprap on side slopes steeper than 1:3 must be designed using a steep side slope design procedure provided in Hydraulic Engineering Circular No. 15: Design of Roadside Channels with Flexible Linings, (Reference 2.18).

$\tau_{max} < \tau_p$ - Design Size Acceptable
$\tau_{max} > \tau_p$ - Design Size Not acceptable

Example Problem:

30 cfs is directed down a standard 10' wide, a trapezoidal highway ditch which has 1:3 side slopes and a 10% grade. Find a stable Riprap material to line the ditch.

Step 1. Using Manning's Equation (See Chapter One: Drainage, Section 7.C and Eq. 1.5) determine:

Flow depth, $d = 0.74$ ft.
Hydraulic Radius, $R = 0.62$ ft.

Step 2. Using Equation 2.1, determine the maximum average shear stress, τ_{max}:

$\tau_{max} = \gamma RS$ = (62.4 lbs./cu. ft.) x (0.62 ft.) x (0.1 ft./ft.) = 3.87 lbs./sq. ft.

Step 3. Using Equation 2.2 Determine the Permissible Shear Stress:

Type A Rock Riprap: $D_{50} = 0.77$ ft.

τ_p = (4.0 lbs./cu. ft.) x 0.77 ft. ÷ 1.1 = 2.80 lbs./sq. ft.

Step 4. Determine the Acceptability of riprap material:

τ_{max} = 3.87 lbs./sq. ft. > τ_p = 2.80 lbs./sq. ft.: Design Size Not acceptable.

Repeat Steps 3 and 4 until acceptable riprap material determined:

Type B Rock Riprap: $D_{50} = 1.02$ ft.

τ_p = (4.0 lbs./cu. ft.) x 1.02 ft. ÷ 1.1 = 3.71 lbs./sq. ft.

τ_{max} = 3.87 lbs./sq. ft. > τ_p = 3.71 lbs./sq. ft.: Design Size Not acceptable.

Type C Rock Riprap: $D_{50} = 1.28$ ft.

τ_p = (4.0 lbs./cu. ft.) x 1.28 ft. ÷ 1.1 = 4.65 lbs./sq. ft.

τ_{max} = 3.87 lbs./sq. ft. < τ_p = 4.65 lbs./sq. ft.: Design Size Acceptable.

The answer may also be found using EXHIBIT 2.13:

Acceptable Riprap for 30 cfs in a normal ditch section at a 10% (0.10) slope: = Conc. (33 cfs) and Rock Riprap Type C (43 cfs).

RIPRAP - PERMISSIBLE DITCH FLOW (cfs)				
	Riprap Type			
Slope	*Rock Riprap Type A*	*Rock Riprap Type B*	*Broken Concrete Riprap*	*Rock Riprap Type C*
0.05	*42*	*70*	*81*	*107*
0.06	*33*	*55*	*64*	*84*
0.07	*28*	*45*	*52*	*68*
0.08	*23*	*38*	*43*	*57*
0.09	*20*	*33*	*37*	*49*
0.10	*13*	*28*	*33*	*43*
0.11	*11*	*25*	*29*	*38*
0.12	*10*	*22*	*26*	*34*
*0.13**	*9*	*20*	*23*	*30*
*0.14**	*8*	*14*	*21*	*28*
*0.15**	*8*	*13*	*15*	*26*
*0.16**	*7*	*12*	*13*	*23*
	cfs			

** Consider flattening ditch slope and using drop structure.*

Exhibit 2.13 Permissible Ditch Flow (cfs) for a Normal Ditch Section (Based on the Mean Stone Size and the Design Depth Thickness, See Exhibit 2.15)

7.A.5 Channel Bends

On channel bends, the flow around the bend creates secondary currents, which impose higher shear stresses on both the channel sides and bottom compared to the straight channel. The location of this increased shear stress changes across the length of the bend. The shear begins to increase on the inside of the curve and progresses to the outside of the curve as the bend is completed. The maximum shear stress in the bend is a function of the ratio of channel curvature to bottom width, R_c/B. As R_c/B decreases, (i.e. the bend becomes sharper), the maximum shear stress increases. The bend shear stress, is determined by multiplying the straight channel shear stress by a constant, K_b.

$$\tau_b = (K_b) \times (\tau_{max}) \qquad \text{Eq. 2.3}$$

Where: τ_b = Bend shear stress;
K_b = A factor for maximum shear stress on channel bends, (See EXHIBIT 2.14);
τ_{max} = The maximum straight channel shear stress (See Eq. 2.1).

This increased shear stress continues downstream of the bend for a distance, L_p.

$$L_p = (0.604) \times (R^{7/6} \div n_b) \qquad \text{Eq. 2.4}$$

Where: L_p = Distance shear stress continues downstream of bend;
R = Hydraulic radius;
n_b = Manning's roughness in the bend.

Example Problem:

A roadside ditch carries 40 cfs from a culvert through a 20' radius bend adjacent to a county road before emptying into a stream. The 10 ft. wide ditch is at a grade of 0.05 ft./ft. and has 1:3 side slopes. Determine the necessary riprap size.

Step 1. Using Manning's Equation (See Chapter One: Drainage, Section 7.C and Eq. 1.5) determine:

Flow depth, d = 0.90 ft.
Hydraulic Radius, R = 0.73 ft.

Step 2. Using Equation 2.1, determine the maximum average shear stress, τ_{max}:

$\tau_{max} = \gamma RS$ = (62.4 lbs./cu. ft.) x (0.73 ft.) x (0.05 ft./ft.) = 2.28 lbs./sq. ft.

Step 3. Using EXHIBIT 2.14, determine the maximum bend shear stress factor, K_b:

Radius of Curve ÷ Bed Width, (R_c ÷ B) = 20 ft. ÷ 10 ft. = 2.0

From EXHIBIT 2.14: For (R_c ÷ B) = 2.0, K_b = 2.0

Step 4. Using Equation 2.3, determine the maximum bend shear stress, τ_b:

$\tau_b = (K_b) \times (\tau_{max})$ = (2.0) x (2.28 lbs./sq. ft.) = 4.56 lbs./sq. ft.

Step 5. Using Equation 2.2, determine the Permissible Shear Stress, τ_p:

τ_p = (4.0 lbs./cu. ft.) x D_{50} ÷ SF

Type A Rock Riprap: D_{50} = 0.77 ft.; SF = 1.1

τ_p = (4.0 lbs./cu. ft.) x 0.77 ft. ÷ 1.1 = 2.80 lbs./sq. ft.

Type C Rock Riprap: D_{50} = 1.28 ft.; SF = 1.1

τ_p = (4.0 lbs./cu. ft.) x 1.28 ft. ÷ 1.1 = 4.65 lbs./sq. ft.

Step 6. Determine the Acceptability of riprap material:

τ_{max} = 2.28 lbs./sq. ft. < τ_p = 2.80 lbs./sq. ft.: Design Size Acceptable for Straight.

τ_b = 4.56 lbs./sq. ft. < τ_p = 4.65 lbs./sq. ft.: Design Size Acceptable for Bend.

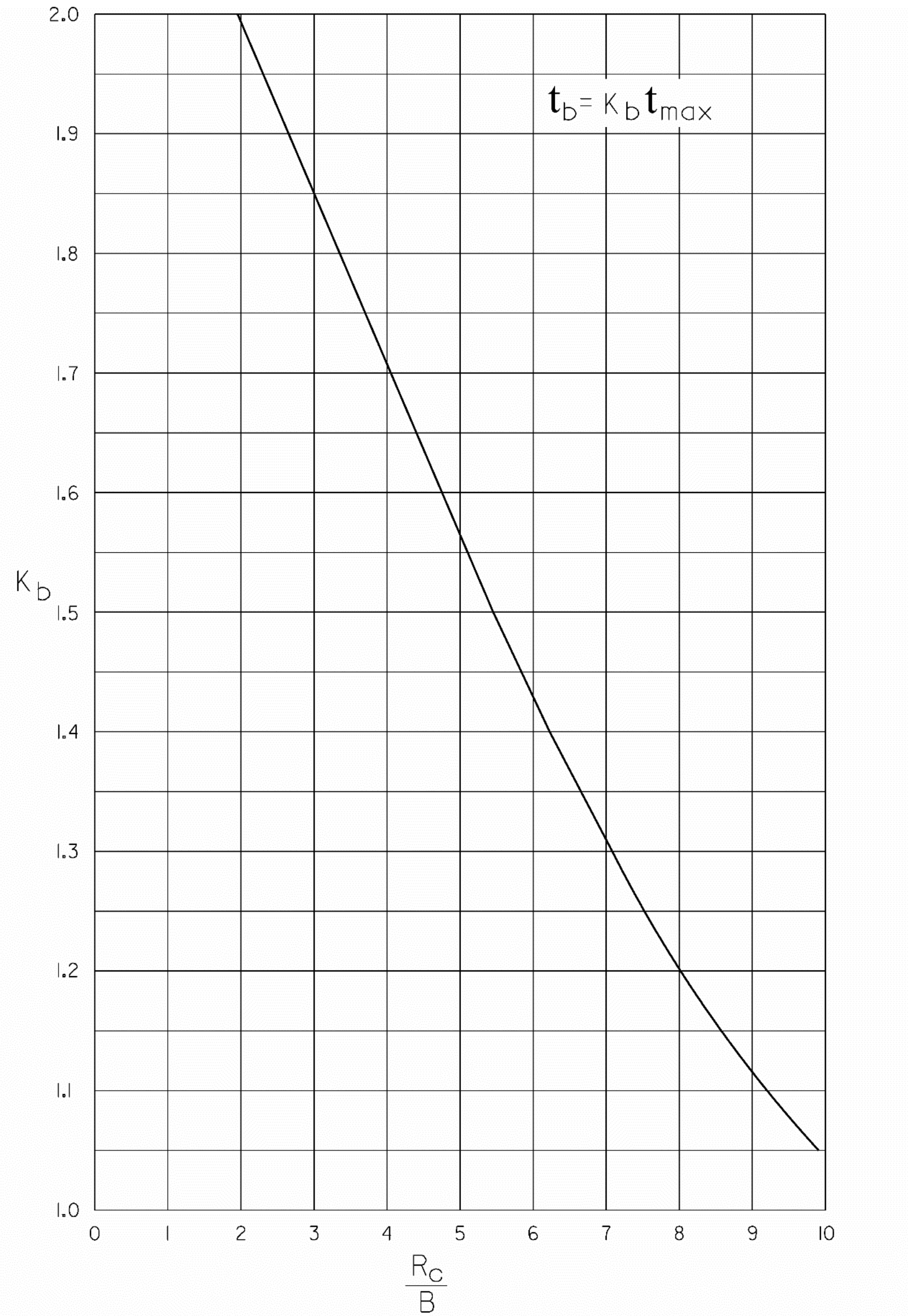


Exhibit 2.14 K_b Factor for Maximum Shear Stress on Channel Bends
(Source: Reference 2.18, Chart 10)

7.A.6 Other Considerations

Two additional design considerations are required for riprap channel linings: Riprap Gradation and Filter Material under rock riprap.

7.A.6.a Riprap Gradation and Thickness

Riprap gradation should follow a smooth size distribution curve. Most riprap gradations should fall within the acceptable range where the ratios D_{100}/D_{50} and D_{50}/D_{20} are between 1.5 and 3.0. The most important criterion is a proper distribution of sizes in the gradation so that interstices formed by larger stones are filled with smaller sizes in an interlocking fashion, preventing the formation of open pockets. Flat slab-like stones should be avoided at all times since they are easily dislodged by the flow, and tend to redirect flow into unprotected banks. These gradation requirements apply regardless of the type of filter design used.

7.A.6.b Filter Design

When rock riprap is used the need for an underlying filter material must be evaluated. The filter material may be either a granular filter layer or an engineering fabric.

Granular Filter Blanket

For a granular filter blanket, the following criteria must be met:

$$(D_{15} \text{ filter} \div D_{85} \text{ base}) < 5 < (D_{15} \text{ filter} \div D_{15} \text{ base}) < 40$$

$$(D_{50} \text{ filter}) < (D_{50} \text{ base}) < 40$$

This relationship must hold between the filter blanket and base material and between the riprap and filter blanket. The filter blanket criteria may require more than one layer of filtering material to be placed. The thickness of the granular filter blanket should approximate the maximum size in the filter gradation. The minimum thickness for a filter blanket should be 6 in.

Engineering Filter Fabric

The following properties of an engineering filter fabric are required to assure that their performance is adequate as a filter under riprap.

- The fabric must be able to transmit water faster than the soil.
- The following criteria for the apparent opening size (AOS) must be met:
  1. For soil with less than 50 percent of the particles by weight passing a US No 200 sieve, AOS < 0.024 in. (0.6 mm) (greater than a US No 30 sieve).
  2. For soil with more than 50 percent of the particles by weight passing a US No 200 sieve, AOS < 0.012 in. (0.297 mm) (greater than a US No 50 sieve).

7.A.7 Placing Riprap

The minimum thickness of the riprap lining placed in a channel is equal to the maximum size stone. EXHIBIT 2.15 provides the various **NDOT** riprap stone sizes and design thicknesses.

NDOT RIPRAP PROPERTIES				
Riprap Type	*Median Diameter D_{50} (ft.)*	*Maximum Diameter D_{100} (ft.)*	*Minimum Design Depth / Thickness (ft.)*	*Basis of Payment (Ton)*
Rock Riprap Type A	*0.77*	*1.28*	*1.50*	*1.35 Ton/cu. yd.*
Rock Riprap Type B	*1.02*	*1.61*	*1.75*	*1.35 Ton/cu. yd.*
Broken Concrete Riprap	*1.10*	*1.88*	*2.00*	*1.35 Ton/cu. yd.*
Rock Riprap Type C	*1.28*	*2.12*	*2.25*	*1.35 Ton/cu. yd.*

Exhibit 2.15 NDOT Riprap Properties

7.A.7.a Channel Bank Riprap

Excavation of the channel bottom and sides for the placement of the riprap material is necessary for the stability of the lining. When riprap is placed in the ditch the flowline of the ditch should be the top of the riprap and the earth side slopes above the riprap should match the top of the riprap, (See EXHIBIT 2.16).

On a straight channel, riprap bank protection should begin and end at a stable feature in the bank. If a stable feature does not exist, cutoffs should be provided as shown in EXHIBIT 2.17. Where the protective cover is long, intermediate cutoffs should be provided to reduce the hazard of complete failure of the stone blanket.

For stream channels composed of sand or silt, bank protection should extend a minimum vertical distance of 3 ft. (1 m) below the streambed on a continuous slope with the embankment (See EXHIBIT 2.18). On the outside of curves or sharp bends, scour is particularly severe, and the toe of the bank protection should be placed deeper than in straight reaches. Where a toe trench cannot be excavated, the riprap blanket should terminate in a stone toe at the level of the streambed (See EXHIBIT 2.19). The toe provides material that will fall into a scour hole and thus extend the blanket.

The purpose of the toe protection is to prevent undermining, not to support the riprap blanket. Unless the protection has sufficient stability to support itself on the embankment slope, the protection cannot be considered adequate.

The upper vertical limit of the riprap blanket should extend at least 1 ft. (300 mm) above design high water. This allowance for freeboard depends upon the velocities near the riprap cover and at some locations, on the height of waves that might be generated on the water surface. Established sod above the stone protection will provide considerable protection from floods that overtop the riprap cover.

Details of riprap used for lining channel banks are shown in EXHIBITS 2.16 THROUGH 2.19.

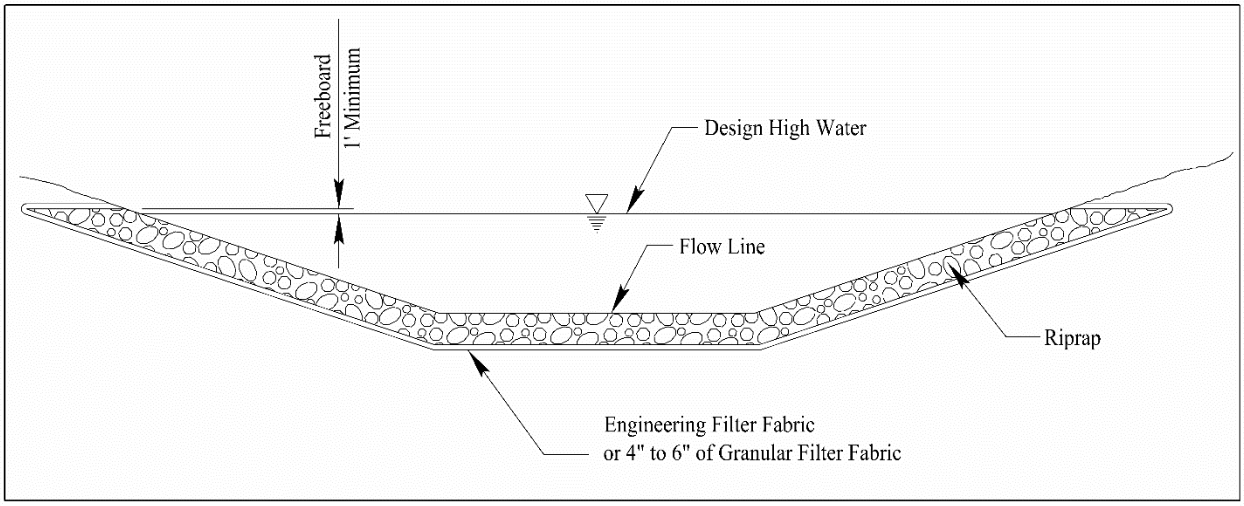


Exhibit 2.16 Channel Perimeter Riprap

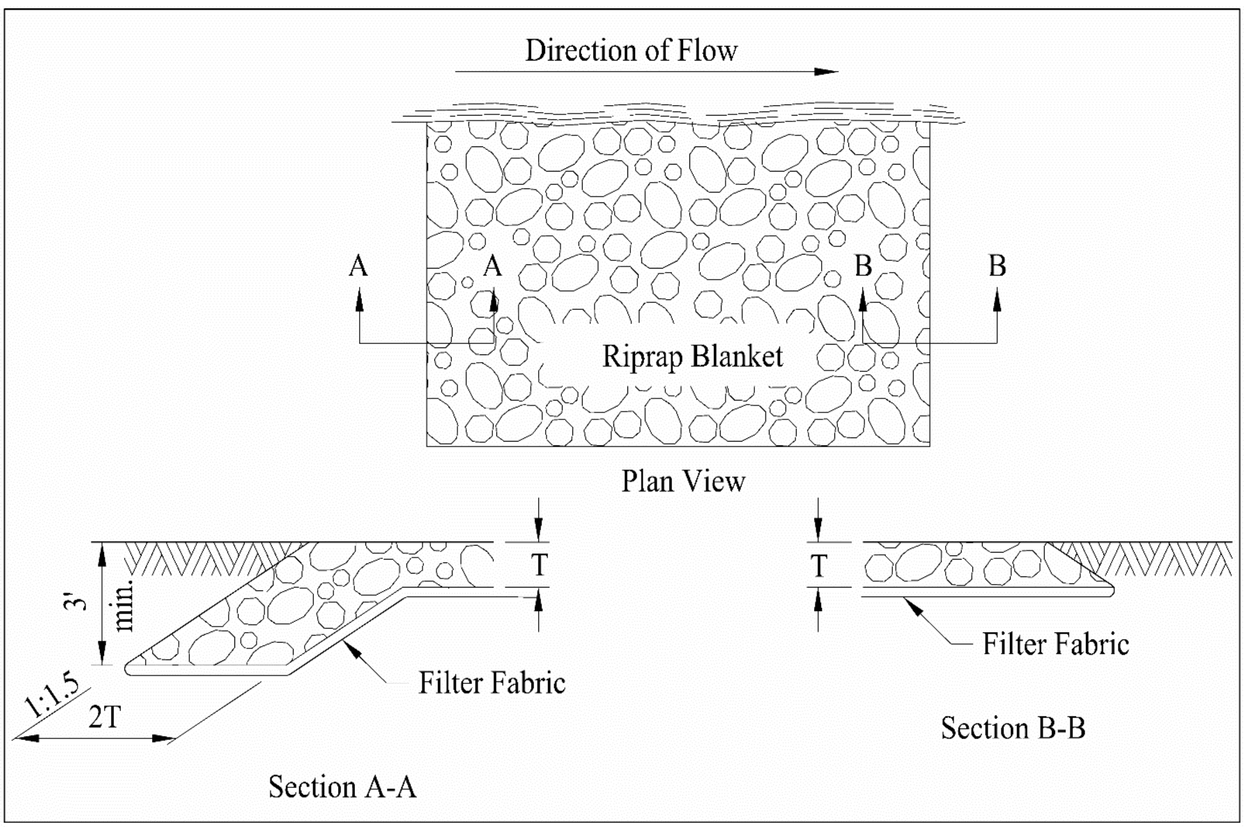


Exhibit 2.17 Riprap Cutoff Detail

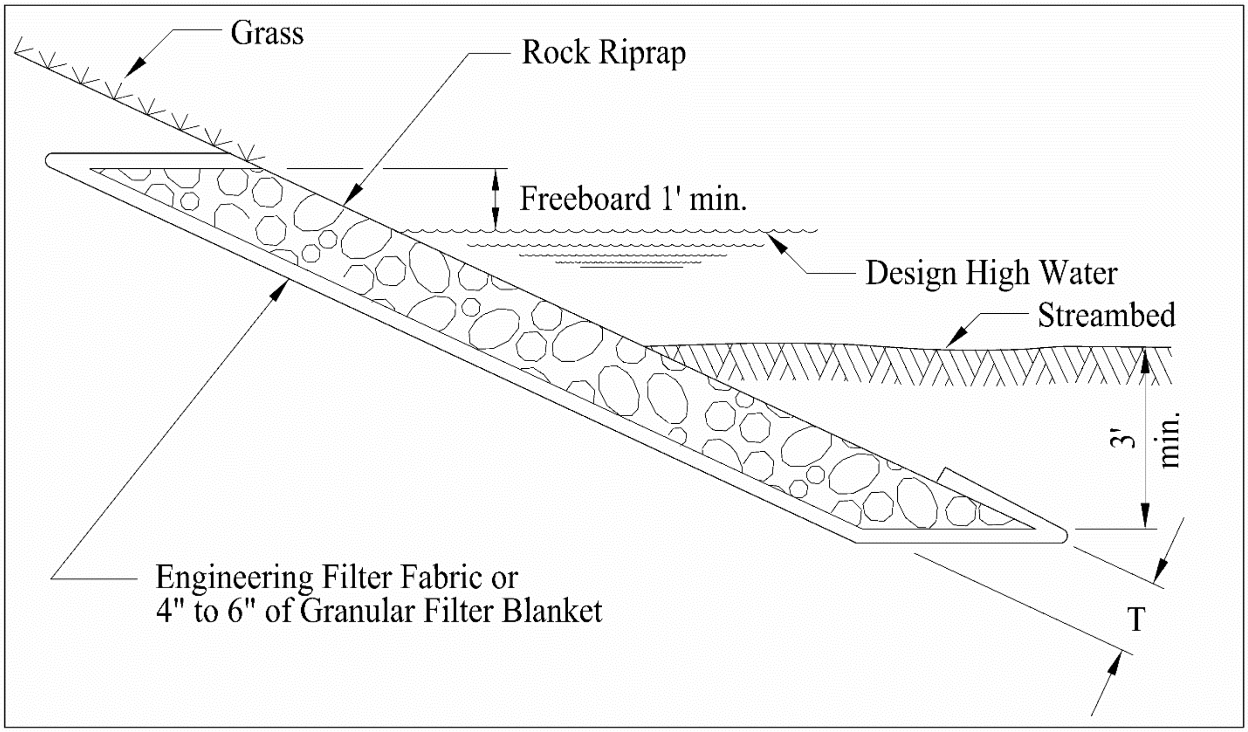


Exhibit 2.18 Riprap Blanket and Toe Trench Detail

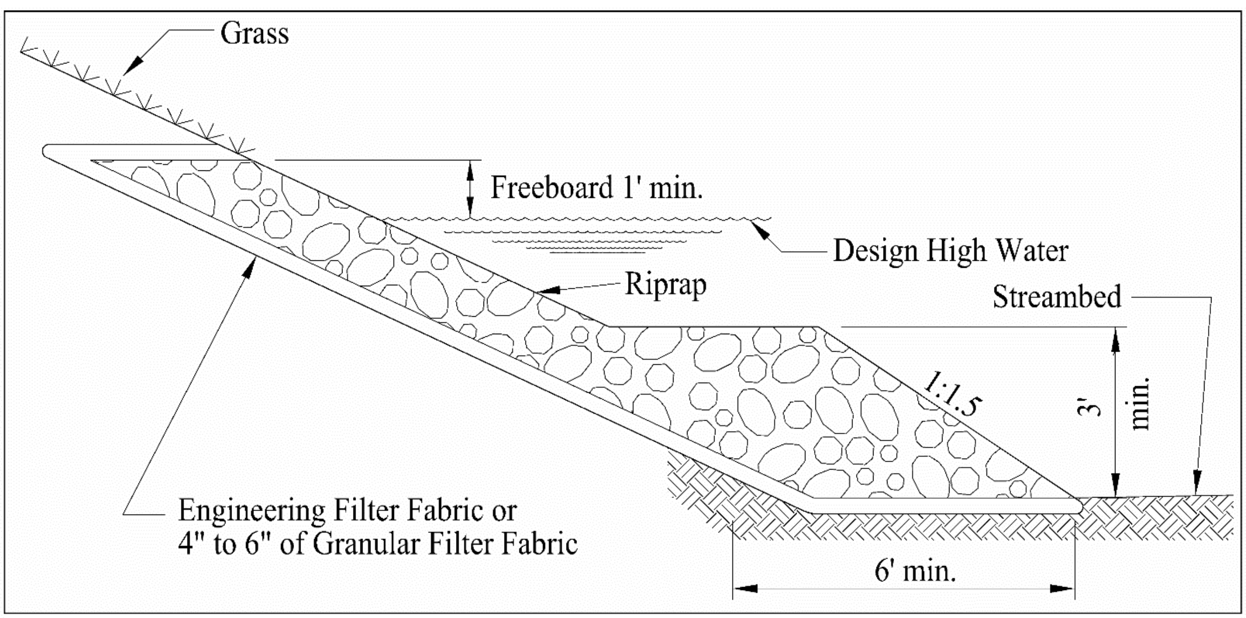


Exhibit 2.19 Riprap Blanket and Toe Detail

7.B Gabions

Gabions are rectangular, rock-filled wire baskets. These wire baskets can be placed side-by-side and stacked vertically to form an unbroken lining or wall. Gabion baskets are shown in EXHIBIT 2.20. Gabions are suitable for several applications including:

- Lining of high, steep channel banks.
- Channel drop structures.
- Energy dissipation at the outlet of culverts.
- Low-height retaining walls.

Gabion baskets are available from manufacturers in different sizes. The 6 ft. and longer Gabion baskets are usually divided into cells with interior diaphragms that reinforce the basket and facilitate basket assembly in the field. Wire used to fabricate the baskets is normally zinc coated. If the baskets will be installed in a corrosive or polluted environment, a continuous, polyvinyl chloride (PVC) sheath can be applied to the zinc coated wire.

When gabions are used for lining of channel banks, the baskets are laid directly on the banks to be protected and function only as a facing or lining. Accordingly, the slope or bank must itself be stable, and must not be so steep as to cause the basket to slide. The **Materials and Research Division** must be consulted to determine the safe, stable slope for the channel bank.

The stability of gabion structures depends on the following:

- Strength of the basket's wire mesh.
- Thickness of the lining.
- Grading of the stone fill.
- Resistance of the wire baskets to corrosion.
- Resistance of the stone fill to freeze thaw and abrasion.

The thickness of the lining and grading of stone fill is dependent upon the velocity of water in the channel. The “critical velocity” is the velocity at which the gabion basket will remain stable without movement of the stone fill. The “limit velocity” is that which is still acceptable although there is some deformation of the basket due to movement of the stones within the compartment.

Research has indicated that the size of rocks needed for gabion baskets is half that required for riprap for a given hydraulic condition. For the same size stone or rock, the acceptable velocity for gabion baskets is approximately 3-4 times that for riprap. The designer should consult manufacturer's literature that provides required lining thickness and stone sizes for a range of velocities.

If gabions baskets are to function as a retaining wall, the Bridge Division must be contacted to design the retaining wall system. Consideration must be given to scour and potential for undermining the foundation.

Construction specifications for gabions are found in Section 907 of the Standard Specifications for Highway Construction, (Reference 2.10). Gabions are measured according to the number of baskets placed and are paid for by each.

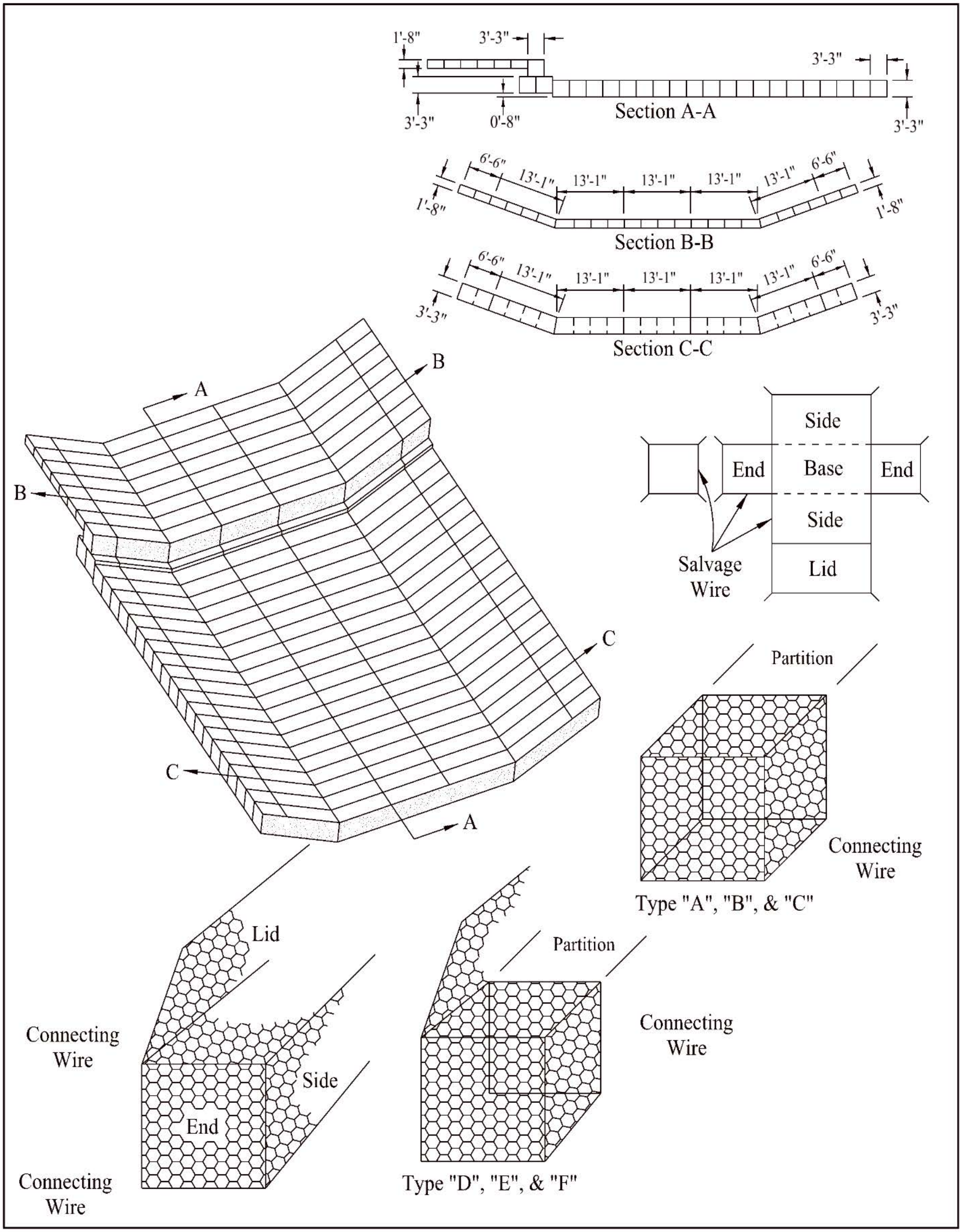


Exhibit 2.20 Gabions

7.C Revet Mattress

A Revet mattress is a special type of gabion (See Section 7.B) with a large surface area-to-thickness ratio. Revet mattresses are multicell containers with internal diaphragms spaced at 3 ft. (1 m) intervals. Wire used for fabricating revet mattresses is zinc coated, and a PVC coating is also available for corrosive environments. Filter fabric is required under revet mattresses.

Revet mattresses are suitable for several applications including:

- Lining of low, flat channel banks.
- Energy dissipation at the outlet of culverts.
- Protection of highway embankments.

Revet mattresses have requirements for placement, stability, lining, rock size, etc. similar to those for gabions, (See Section 7.B for discussion).

Refer to manufacturer's publications for construction specifications and plans for revet mattresses. Revet mattresses are measured and paid by each, (Refer to Section 907 of the Standard Specifications for Highway Construction, Reference 2.10).

7.D Cellular Confinement System

A cellular confinement system consists of a network of three-dimensional cells constructed of High Density Polyethylene (HDPE) plastic. The HDPE sections are shipped as collapsed bundles that are expanded at the job site into thick sheets of varying size and installed. The cellular confinement system provides root reinforcement and increased shear resistance when the cells are filled with native soils and vegetation, but the cells can also act as a ditch lining when filled with various materials including granular materials and concrete. EXHIBIT 2.21 shows a cellular confinement system.

A cellular confinement system can be used for several types of applications including:

- Stabilizing ditch bottoms.
- Lining of stream channel banks.
- Construction of retaining walls.
- Protection of highway embankments.

The cellular system is flexible and can conform readily to changes in subgrade profile. When filled with concrete, the system eliminates the need for forms and expansion joints. The flexible nature of the concrete filled cellular confinement system allows it to respond more favorably to changes in ditch or channel geometry caused by erosion or subsidence than cast-in-place pavements.

The designer should contact the manufacturer to obtain hydraulic design parameters including allowable velocities and Manning's "n" values.

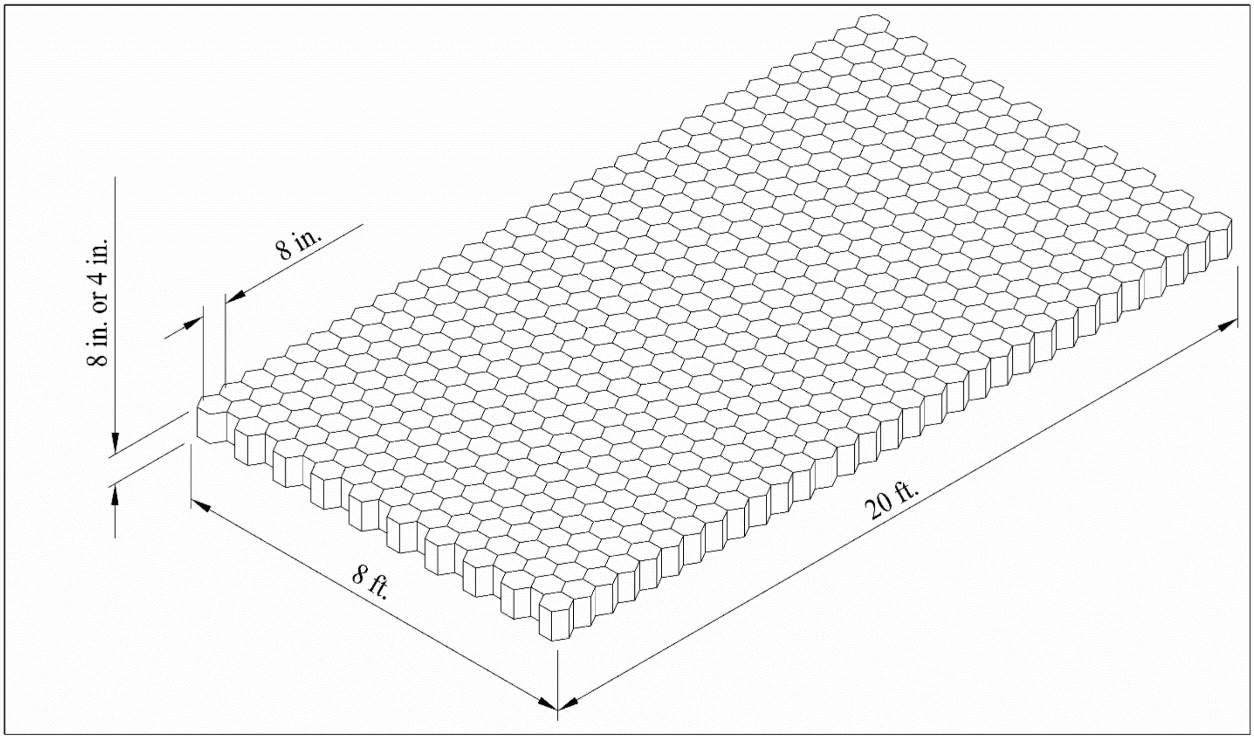


Exhibit 2.21 Typical Cellular Confinement System

7.E <u>Curb and Flume</u>

7.E.1 3 in. (75 mm) Curb

Curb and flumes may be used as a permanent erosion control measure. They may also be used as temporary measures during phasing similar to the ditch lining concept. Use 3 in. (75 mm) curb on:

- Both shoulders of roadways with profile grades steeper than 3%.
- Both shoulders of roadways with profile grades 2.5% and steeper, which are longer than 1500 ft. (450 m).
- Low side of superelevated sections.
- Highly erodible soils.
- Multilane facilities in section when approved by the **Assistant Design Engineer**.
- Large and steep embankments.

A 3 in. (75 mm) curb shall have a minimum 2 ft. (600 mm) turf shoulder behind the curb. The turf shoulder should preferably drain *away* from the curb, desirably at a 4% grade but a flat shoulder slope is acceptable Water flowing in the gutter shall be removed from the roadway before reaching the 3 in. (75 mm) depth using Type I through VIII Concrete Flumes.

7.E.2 Concrete Flumes

Concrete flumes are used to collect stormwater runoff from the curbed shoulder and convey it down the foreslope of the highway embankment to the toe of the slope and into a receiving ditch or drainage way. Two types of flumes are used by **NDOT**:

- Above ground spillways - Drop Curb and Concrete Flume Types I and II.
- Buried flume pipe structures - Concrete Flume Types IV through VIII.

The above ground spillways convey the water collected from the gutter down the top of the foreslope in a curbed concrete spillway (See Special Plans 4341 through 4346 in the Standard/Special Plans Book, Reference 2.12). These spillways have a tendency to break and separate on fill depths greater than 4 ft. (1.2 m), as measured from the shoulder, and at breaks in the foreslope where it changes from a 1:6 to a 1:4 or 1:3 slope. The use of Concrete Flume Types I and II should be limited to shallow embankments, 4 ft. (1.2 m) or less, with uniform foreslopes.

The buried flume pipe structures convey the water collected by a grate or area inlet down a 15 in. (375 mm) corrugated metal pipe buried in the foreslope embankment, (Refer to Standard Plans 5470 and 5480 in the Standard/Special Plans Manual, Reference 2.12, http://www.roads.nebraska.gov/business-center/design-consultant/stand-spec-manual/). The buried flume pipe is the preferred method for conveying water down high embankments, greater than 4 ft. (1.2 m), or where breaks occur in the foreslope.

Prior to the plan-in-hand field inspection the designer will create a list of all roadway grades between 2% and 3½% and a list of all roadway grades greater than 3½% for a comparative analysis of erosion control techniques (i.e. curb and flume vs. other erosion control methods).

Spacing of the concrete flumes is based on the 2-Year design storm, Q_2, the spread distance of the water on the shoulder, and the 3 in. (75 mm) height of the asphalt curb, (Refer to EXHIBIT 2.22 for a guideline on Concrete Flume Spacing). Not all roadway design situations will fall within the parameters used to establish the above guideline; therefore the roadway designer should check the effect of a 2-Year design storm on the gutter flow for the chosen concrete flume spacing, (See Chapter One: Drainage, Section 10.A).

CONCRETE FLUME SPACING		
Concrete Flumes, Types I & II (For Embankment Heights of 4 ft. (1.2 m) or Less)		
Slope of Roadway Centerline	*Distance between Flumes*	*Superelevated*
2% - < 2.5% (if longer than 1500 ft. (450m))	*500 ft. (150 m)*	*400 ft. (120 m)*
2.5% - < 4%	*500 ft. (150 m)*	*400 ft. (120 m)*
4% - < 5%	*400 ft. (120 m)*	*300 ft. (90 m)*
5% - < 6%	*350 ft. (110 m)*	*250 ft. (80 m)*
Concrete Flumes, Type IV – VIII (For Embankment Heights Greater than 4 ft. (1.2 m)		
Slope of Roadway Centerline	*Distance between Flumes*	*Superelevated*
2% - < 2.5% (if longer than 1500 ft. (450m))	*400 ft. (120 m)*	*350 ft. (110 m)*
2.5% - < 4%	*400 ft. (120 m)*	*350 ft. (110 m)*
4% - < 5%	*350 ft. (110 m)*	*300 ft. (90 m)*
5% - < 6%	*300 ft. (90 m)*	*250 ft. (80 m)*

Exhibit 2.22 Concrete Flume Spacing Guidelines

7.F Runoff Intercepting Methods

7.F.1 Intercepting Earth Dike

An intercepting earth dike is a ridge of soil constructed to divert stormwater runoff at non-erosive velocities to a stabilized outlet or pond. Intercepting earth dikes are commonly located at the uphill side of a disturbed backslope or borrow area. Intercepting earth dikes may also be used to divert sediment-laden drainage runoff from a disturbed area to a sediment trapping facility, (See Section 7.I). Intercepting earth dikes should be considered when:

- Runoff from higher areas may damage property, cause erosion, or interfere with the establishment of vegetation on lower areas.
- Runoff with high sediment loads threatens sensitive wetlands or water bodies.
- It is necessary to maintain a separation between different types of flow such as irrigation and stormwater runoff.

Intercepting earth dikes are frequently used to concentrate and direct water to slope drains (See Section 7.F.3), grade control structures (See Section 7.G) and other outlet structures. When they are used for this purpose, it is important to place the earth dike outlets in a manner that will maintain the existing drainage pattern. Refer to EXHIBIT 2.23 for typical intercepting earth dike designs.

When designing an intercepting earth dike it is important to keep in mind that water may be diverted on both sides of the dike, not just the water on the upslope side.

Temporary intercepting earth dikes may be used whenever stormwater runoff must be temporarily diverted to protect disturbed areas and slopes, or to retain sediment on-site during construction.

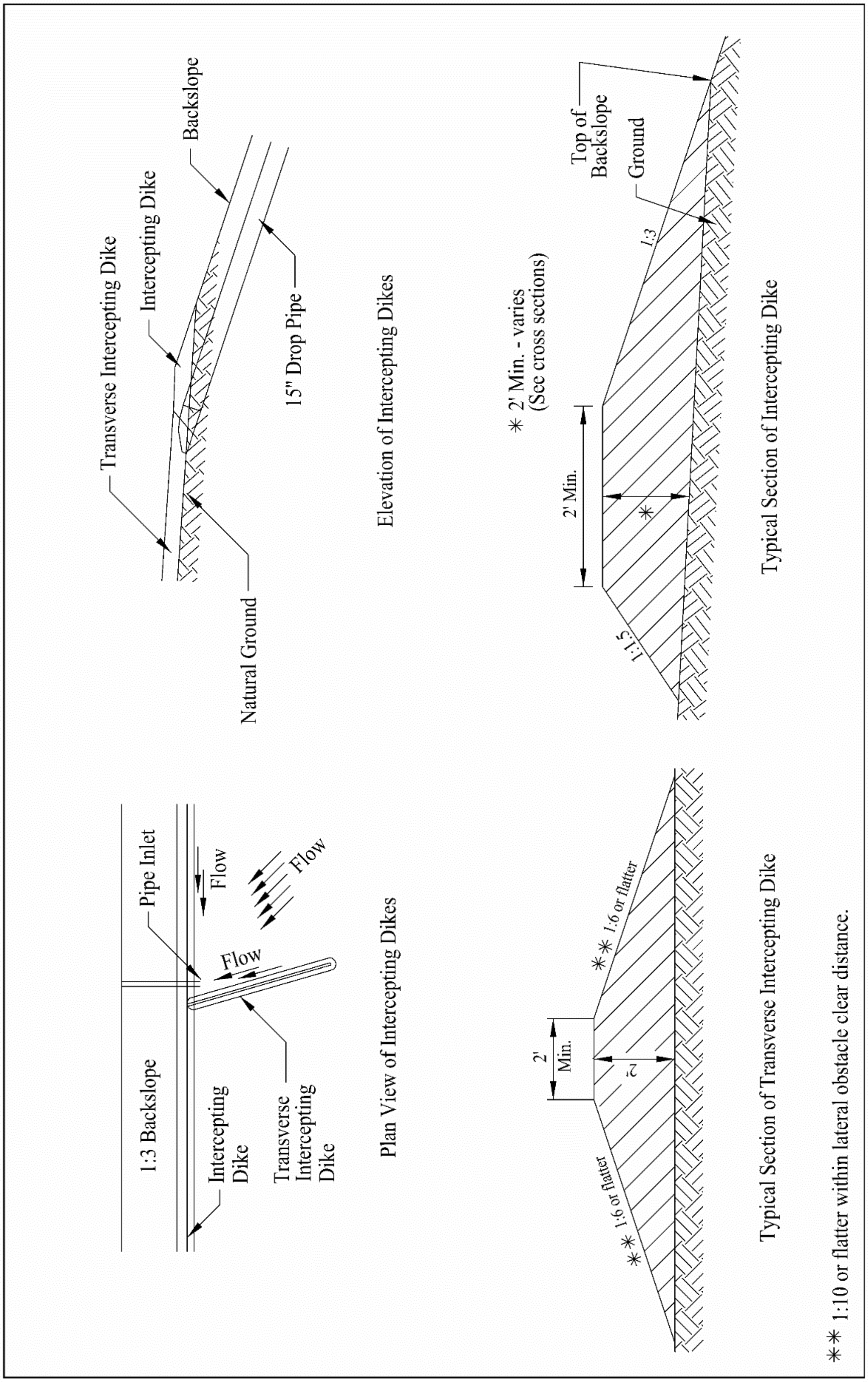


Exhibit 2.23 Intercepting Earth Dikes

7.F.2 Intercepting Ditch

An intercepting ditch is a channel constructed at the midpoint of deep backslopes. It is used to intercept and convey water at non-erosive velocities to an adequate and stable outlet.

On backslopes where the slope width is in excess of 100 ft. (30 m) measured along the slope and transverse to the top edge of the backslope, an intercepting ditch will be required halfway down the slope and at a maximum of 100 ft. (30 m) intervals. Intercepting ditches can only be utilized on cut slopes. They are not allowed on foreslopes. Temporary intercepting ditches may be used on the lower side of cleared areas that will be excavated. The **Roadside Development & Compliance Unit** in the **Project Development Division** can be consulted and will provide recommendations on the location of intercepting ditches.

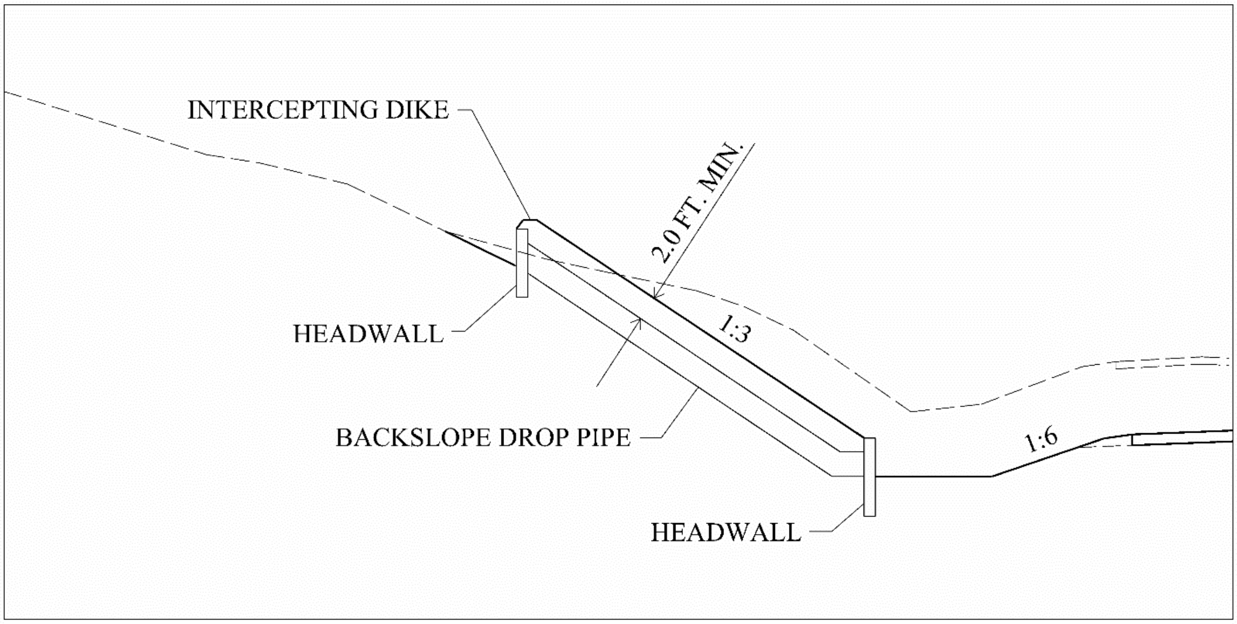
Care must be taken to outlet intercepting ditches into adequately stabilized areas. All intercepting ditches must be seeded in accordance with the guidelines for temporary and permanent vegetation (See Sections 5.B and 6.A, respectively). Intercepting ditches should be protected with the proper erosion control, when necessary.

7.F.3 Backslope Drop Pipe

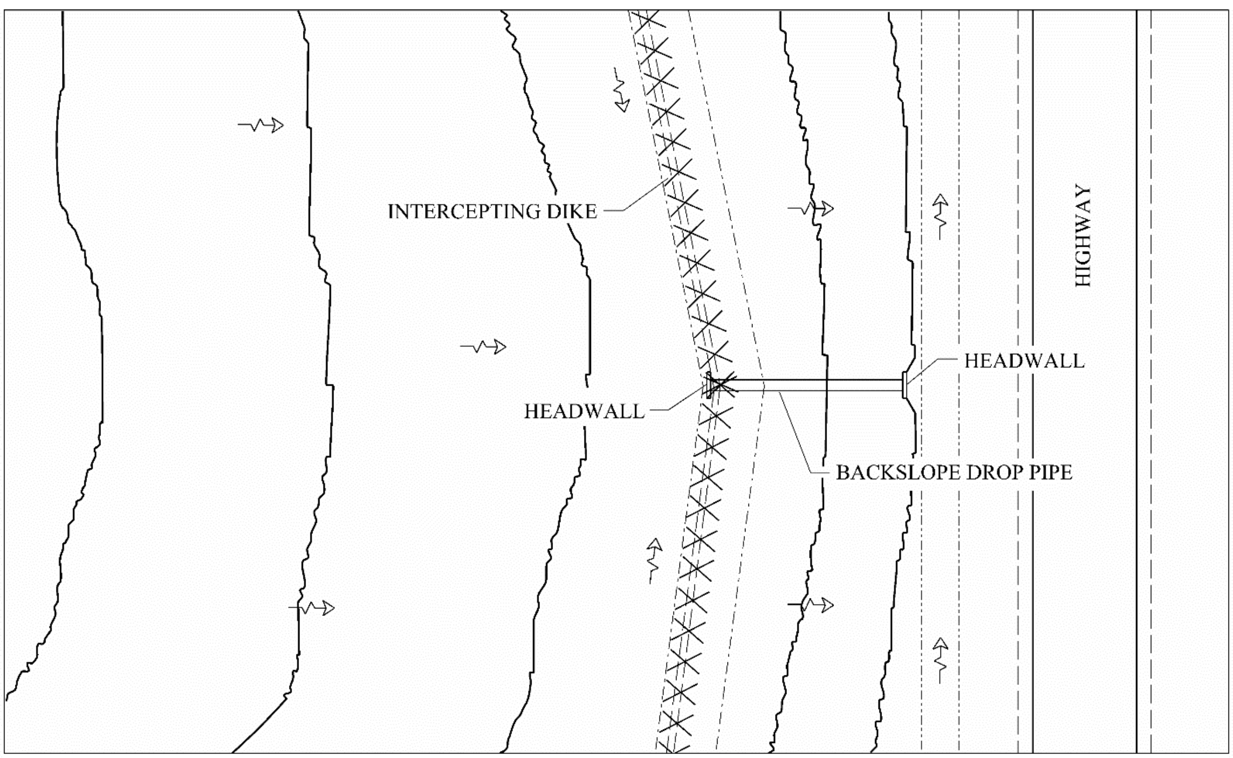
Backslope drop pipes are frequently used to convey water diverted and concentrated by intercepting dikes and ditches without causing erosion on or below the slope. It is essential to protect against the potentially high discharge velocity of water at the outlet by using erosion control blankets, riprap or other measures.

Backslope drop pipes should be used on cut or fill slopes where there is a potential for upstream flows to go over the face of the slope causing erosion and preventing adequate stabilization. An example of such a case would be where a hill slopes down to a highway with a newly lowered profile, causing a 1:3 backslope to be excavated which is 200 ft. (60 m) long and 10 ft. (3 m) deep. An intercepting earth dike is placed to collect runoff before it flows over the backslope. The water concentrated by the dike is diverted to two drop pipes spaced 100 ft. (30 m) apart, which lower the collected water to the road ditch.

Backslope drop pipe should be sized according to the parameters established in Chapter One: Drainage and Appendix C, “Pipe Material Policy”. A culvert cross section will be required. EXHIBIT 2.24 illustrates a backslope drop pipe.



Cross-Section View



Plan View

Exhibit 2.24 Backslope Drop Pipe with Intercepting Dike

7.G Ditch Grade Control Structures

Ditch grade control structures are used to prevent degradation of the roadside ditch by scour or head-cutting. The grade control structures consist of various permanent erosion control measures, such as sills or check structures and drop structures, which reduce the flow velocity in the ditch to within acceptable limits.

Sills or check structures consist of earth or stone mounds placed across the ditch with a rise of 1 to 2 ft. (300 to 600 mm) above the flow line of the ditch. The check structures control the scour in the ditch by ponding water behind the structure and slowing the water. To be effective the check structures need to be placed close enough together to control the energy grade line of the water flowing in the ditch.

Drop structures consist of various measures used to convey water down large vertical distances over short lengths. The placement of drop structures controls scour in the ditch by permitting the construction of milder, less erosive, longitudinal slopes, and controls head-cutting of the ditch by conveying the water to the lowered flow line of the downstream body of water in a non-erodible channel, culvert, or other structure. Drop structures can consist of a series of short vertical drops spaced periodically along the length of the ditch or a single large vertical drop at some point along the ditch, usually at the beginning or end of the ditch.

The design of any type of ditch grade control structure must consider the safety of the traveling public. Grade control structures placed within the lateral obstacle clearance zone shall be designed to be traversable, (See the Roadway Design Manual, Chapter Six: The Typical Roadway Cross-Section and Chapter Nine: Guardrail and Roadside Barriers, Reference 2.16).

7.G.1 Drop Pipe

Drop pipes are one type of ditch grade control structure, and are commonly found adjacent to bridges where the roadside ditch outlets to a stream that flows at a significantly lower elevation. (Backslope drop pipes are discussed in Section 7.F.3). Where ditches enter into a receiving stream above its base flow elevation, a drop structure should be designed to convey the water from the ditch to the stream.

The typical drop pipe will be a broken-back (BB) culvert that passes through an earthen dike, which closes the ditch, and outlets into a lower ditch or body of water. Headwalls are preferred at the inlet and outlet of the culvert, though flared end sections are acceptable. Headwalls are preferred because they provide a cutoff wall at the inlet that will reduce water movement along the outside of the culvert pipe and they provide physical support at the inlet and outlet to protect against movement of the culvert.

Drop pipe will be designed according to the parameters established in Chapter One: Drainage and Appendix C, “Pipe Material Policy”. A maximum headwater elevation of 1.5 ft. (450 mm) below the shoulder point of the roadway shall be maintained. Where the drop pipe exists in a critical location it may be advisable to design the culvert to a larger storm event than is indicated in Chapter One: Drainage, Section 6. The outlet section of the culvert pipe should be designed to keep the outlet velocity to a minimum. When necessary, an energy dissipator shall be constructed at the outlet.

The earthen dike used to close the ditch should be built to an elevation that provides a 1 ft. (300 mm) freeboard for the design headwater. The foreslope and backslope of the dike shall be designed to meet safety requirements.

EXHIBIT 2.25 shows a typical design for a drop pipe flowing from a ditch into a stream.

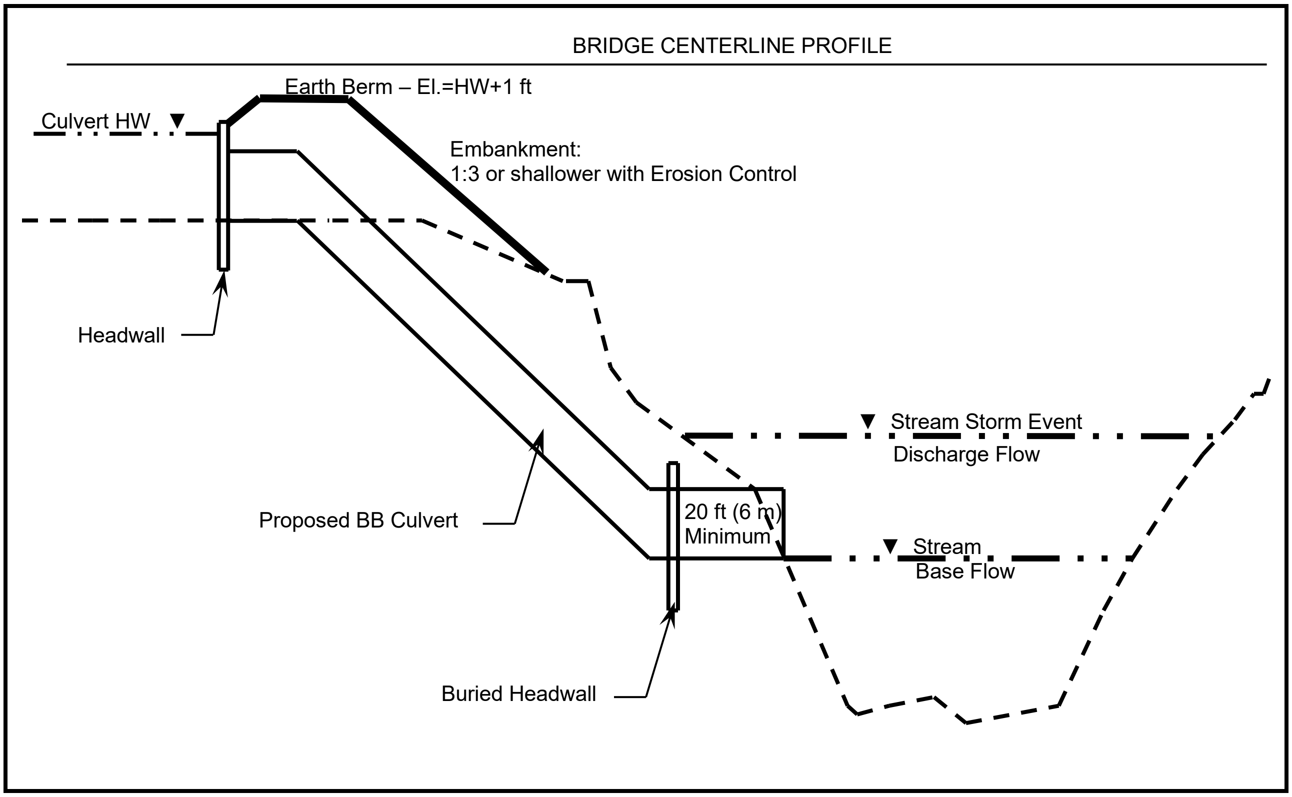


Exhibit 2.25 Typical Drop Pipe Design from Ditch to Stream

7.H Ditch Lining

Ditch lining, (concrete or bituminous), represents the highest level of erosion control that can be provided for an embankment, ditch or channel. The ditch lining consists of paving an embankment or floor and banks of a ditch or channel with either pre-cast concrete blocks, cast-in-place reinforced concrete pavement, or bituminous pavement. Placement of ditch lining is applicable to locations where the anticipated erosive forces are too great to maintain a grade for vegetation and other erosion control methods prove to be unacceptable. Consideration should be given for concrete ditch linings where slopes are greater than 9%, and in locations where continuous water flow occurs.

7.H.1 Articulated Concrete Block Lining

Articulated concrete block linings consist of pre-formed sections that interlock with each other, are attached to each other with wire cable or synthetic fiber rope, or butt together in order to form a continuous blanket or mat. The concrete blocks that make up the various manufacturers' mats differ in shape and method of articulation, but share certain common features. These features include:

- Rapid installation with some flexibility in placement geometry.
- Flexibility to respond to minor changes in ditch or channel geometry without failure.
- Permeable in nature to allow free draining of back slopes and ponded water.
- Provisions for establishment of vegetation within the lining.

Although the design procedure for the articulated concrete block linings is similar to that of riprap linings, the individual manufacture's design procedures should be followed. In any case, filter fabric shall be placed under all installations.

7.H.2 Cast-In-Place Concrete Ditch Lining

Cast-in-place concrete ditch linings consist of reinforced concrete pavement placed on a prepared base along the bottom and sides of the ditch. The concrete pavement forms a continuous rigid structure that will handle high velocity flows with little or no deterioration of the lining.

Cast-in-place concrete ditch linings should be considered where the hydraulic efficiency of a smooth channel surface is important, or the width of the ditch must be kept at a minimum. A concrete channel will carry nearly the same amount of water as a grass ditch twice its width, and can be built with vertical walls.

Cast-in place concrete ditch lining can be very maintenance intensive. These linings are rigid structures that do not normally respond favorably to changes in ditch or channel geometry. The designer should be aware that rigid concrete channel lining is susceptible to damage by the following:

- Undercutting.
- Hydrostatic uplift.
- Erosion and scour along the interface between the lining and the natural channel surface.

The loss of even small sections of the supporting base can cause complete failure of the cast-in-place concrete ditch lining.

Cast-in-place concrete ditch lining details are provided in Standard Plan 4550 in the Standard/Special Plans Manual (Reference 2.12) and in Section 908 of the Standard Specifications for Highway Construction, (Reference 2.10).

7.H.3 Concrete Slope Protection

Concrete slope protection includes the placement of concrete slabs on bridge embankments to protect the slope from erosion. Concrete slope protection is usually placed on the embankments of bridges over highways. Bridges over rivers generally have riprap placed on the embankments (See Section 7.A). Refer to Section 908 of the Standard Specifications for Highway Construction, (Reference 2.10) and to the project Bridge Plans for details.

7.I Sediment Control

7.I.1 Sediment Trap

A sediment trap (See EXHIBIT 2.27) is a temporary structure that is used to detain runoff from small drainage areas so the sediment can settle out. These devices are constructed by excavation and by the construction of an embankment that will provide a determined storage volume. The release, or flow from the structure, is controlled by either a rock spillway or pipe outlet. Sediment traps are generally limited to a contributing drainage area of 5 acres (2 hectares).

When properly designed, located, and constructed, sediment traps can remove non-colloidal sediment at efficiencies of up to 80%. Sediment traps are excellent perimeter controls, provided that runoff from the disturbed area drains to one location and that sufficient right-of-way and storage volume are available. Temporary sediment traps may also be constructed upstream of inlets during grading operations, but only if sufficient storage volume can be created.

7.I.1.a Design

The design of sediment traps involves determining the required storage volume, the dimensions of the spillway, and the necessary elevations. In most cases a simple approach is used to determine the storage volume. The length of the spillway can be computed as a function of the drainage area. This design approach is acceptable for small drainage areas, however, a more precisely designed and efficient designed trap may be sized using the procedures for sediment basins (See Section 7.I.2).

7.I.1.b Location

The location of sediment traps is critical in their design and should be determined based on the existing and proposed topography of the site. As a perimeter control the sediment trap should be placed where 2 to 5 acres (0.8 to 2 hectares) drain to one location. The designer should attempt to choose a location where maximum storage can be obtained using the natural topography, reducing the required excavation. The location of the sediment trap should also be at a location which will minimize interference with construction activities and will allow the trap to remain in service until the site is stabilized. The site must be accessible for future clean-out of the sediment. The designer should also consider the consequences should the structure fail.

7.I.1.c Storage Volume

In order for a sediment trap to function properly the required storage volume must be provided. This volume is created by excavation of the site and/or the construction of an embankment to detain runoff. The required storage volume equates to 134 cu. yd. per acre (253 m^3 per hectare) of drainage area. The drainage area should not exceed 5 acres (2 hectares). This storage volume provides enough space to trap the first 1.0 in. (25 mm) of runoff from the drainage area. If this storage volume cannot be obtained due to site constraints, the maximum available storage volume should be provided. The available storage volume should not fall below a minimum value of 67 cu. yd. per acre (126 m^3 per hectare). The storage volume should preferably be designed at a minimum length to width ratio of 2:1.

The total storage volume of the sediment trap should be divided equally between wet and dry storage. The wet storage should consist of a permanent ponding area based on a volume of 67 cu. yd. per acre (126 m^3 per hectare), which is either excavated earthwork or placed behind a solid earth berm. The dry storage should consist of a temporary ponding area based on a volume of 67 cu. yd. per acre (126 m^3 per hectare), which is behind and below the crest of a self-draining embankment.

When the storage volume falls below the desired 134 cu. yd. per acre (253 m^3 per hectare), the dry storage volume of 67 cu. yd. per acre (126 m^3 per hectare) shall be maintained. Any loss to the sediment trap volume shall be from the wet storage volume.

When a natural depression is used as the sediment trap, and no other methods of calculating the storage volume is available, the volume can be approximated by the following equation:

Volume = 0.4 x Ponding Surface Area at Crest x Maximum Depth Below Crest

7.I.1.d Embankment

The embankment of the structure should be constructed to a maximum height of 5 ft. (1.5 m). The desirable top width of the embankment should be 4 ft. (1.2 m) and the side slopes should be 1:3 or flatter. The embankment berm, once constructed, should be immediately seeded with Temporary Seed (See Section 5.B.2).

7.I.1.e Outlet

The outlet for the sediment trap is a self draining embankment, constructed of Rock Riprap Type A. The crest of the self draining embankment shall be constructed 1 ft. (305 mm) below the top of the impermeable soil embankment. The weir length of the spillway is based on the upstream contributing drainage (See EXHIBIT 2.26).

Contributing Drainage Area	***Weir Length***
1 Acre (0.4 hectare)	*4 ft. (1.2 m)*
2 Acre (0.8 hectare)	*5 ft. (1.5 m)*
3 Acre (1.2 hectare)	*6 ft. (1.8 m)*
4 Acre (1.6 hectare)	*10 ft. (3.0 m)*
5 Acre (2.0 hectare)	*12 ft. (3.6 m)*

Exhibit 2.26 Weir Lengths for Sediment Trap's Self Draining Embankment

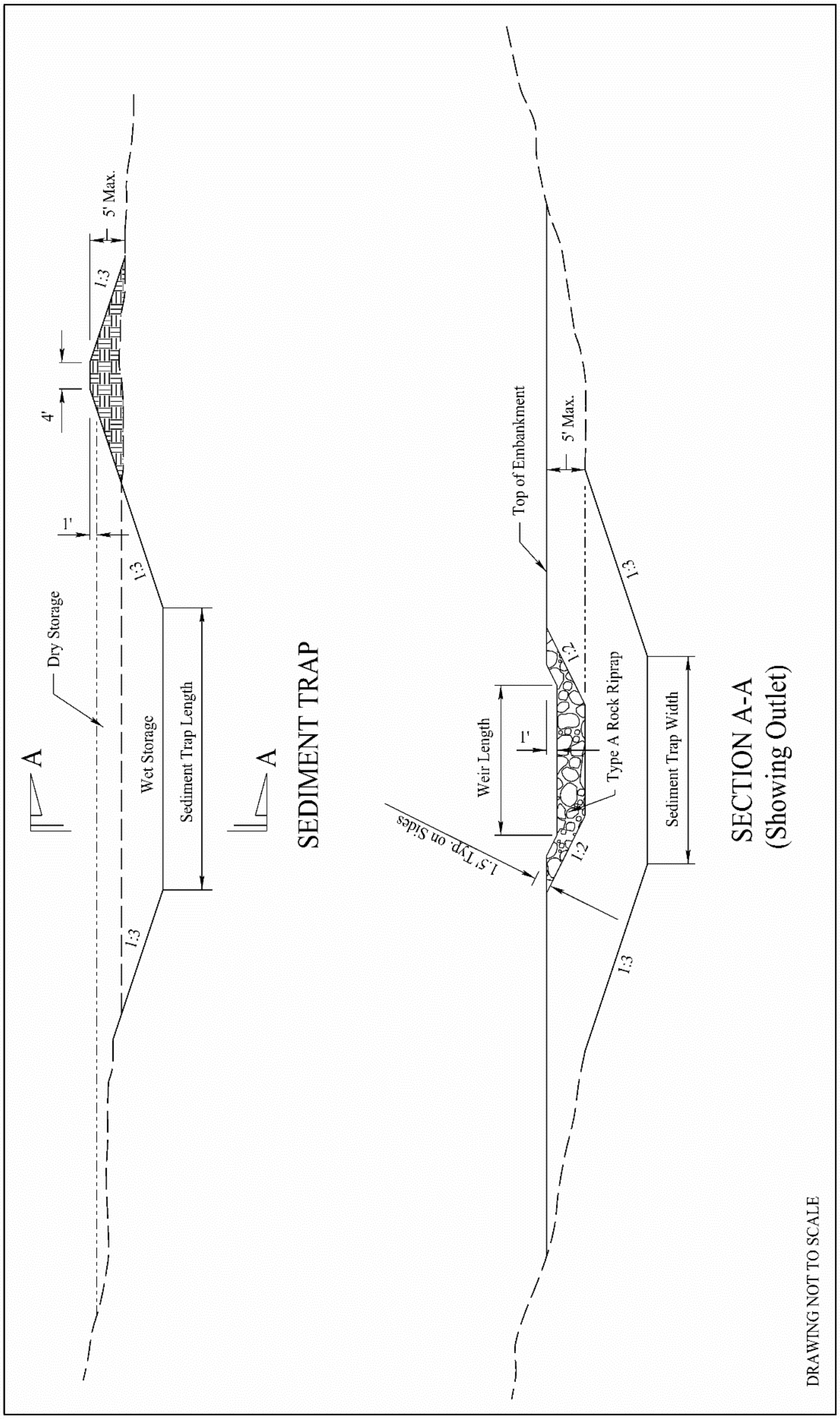


Exhibit 2.27 Sediment Trap

7.I.2 Sediment Basin

A sediment basin is a structure used to detain sediment-laden runoff from disturbed areas long enough to allow the majority of the sediment to settle out. The structure is created by excavating a basin or by constructing an embankment of compacted soil across a drainage way as a temporary barrier or a permanent dam. Ponded water is released through a controlled stormwater release structure. Sediment basins are generally located downslope from the construction site. The **Roadside Development & Compliance Unit** in the **Project Development Division** recommends sediment basin locations. Sediment basins should be constructed prior to clear running streams and where downstream land use is sensitive to sedimentation.

Sediment basins are used for disturbed areas where the total contributing area is equal to or greater than 5 acres (2 hectares). Sufficient space and appropriate topography must be available for the construction of a temporary sediment basin. These structures should be considered to have a limited useful life of 18 months unless the basins are designed as permanent impoundments.

Large sediment basins require extensive design. There are three general areas of consideration in the design of these sediment basins:

1. Adequate storage volume for expected sediment.
2. Adequate retention of runoff to allow settlement of suspended particles.
3. A dam and spillway to accommodate expected flows.

Storage volume requirements can best be determined from past experience at similar sites. Contact the **Roadway Design Division Hydraulic Engineer** for further information. It is generally not cost effective to provide a volume sufficient to contain the total expected sediment runoff from an area during the entire construction period. Therefore, a reasonable length of time between cleanouts should be established and a volume chosen to accommodate this period.

Required retention time for runoff in a basin is dependent on sediment particle size and the desired percent of sediment removal. It is generally acceptable and practicable to remove 70% to 90% of particles larger than the very fine sands having diameters greater than 0.002 in. (0.051 mm). Silt and clay-sized particles require excessive retention time so it is generally not feasible to design a basin to remove them. Widely used methods of determining suitable size for retention basins are based on particle settling times or a set runoff volume. National Cooperative Highway Research Program (NCHRP), Report 70, Design of Sediment Basins, (Reference 2.13), provides details for sediment basin design and selection.

While retention determinations are based on small inflows in the range of a 10-year return frequency event, the emergency spillway must be designed to accommodate a much larger event. Since failure could result in release of considerable quantities of stored sediment, spillway design should be based on an economic assessment of potential damages.

7.J Energy Dissipators

When the outlet velocity from a culvert cannot be reduced to acceptable levels by other means, the flow energy should be dissipated before the discharge is returned to the downstream channel. Prior to designing a scour hole, or other energy dissipator, the designer should try to reduce outlet velocity of the culvert by:

- Choosing gentler slopes if possible.
- Installing a soil saver end section at the inlet and lowering the slope of the culvert.
- Designing a broken back culvert with a flat outlet section (See Chapter One: Drainage, Section 8 and Appendix C, "Pipe Material Policy", for more information on culvert design).

An energy dissipator should be constructed when the outlet velocity of a culvert exceeds the values shown in EXHIBIT 2.28. For further information on the types and design of energy dissipation systems, consult Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, (Reference 2.15), (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf).

REQUIREMENTS FOR ENERGY DISSIPATORS	
Design Flow Outlet Velocity in fps (m/s)	***Is an Energy Dissipator Required?***
Less than 8 (2.5)	*No*
8-15 (2.5-4.5)	*Evaluate on a case-by-case basis*
Greater than 15 (4.5)	*Yes*

Exhibit 2.28 Requirements for Energy Dissipators

7.J.1 Preformed Scour Hole (Riprap Basin)

A preformed scour hole is an excavated hole or depression that is lined with riprap of a stable size and designed to prevent scouring at a culvert outlet, (See EXHIBIT 2.29). The depression provides for both a vertical and lateral expansion of the flow and a temporary stilling pool at the culvert outlet. The turbulence caused by the flow expansion and stilling pool dissipates the excessive energy in the culvert discharge within the protected depression. The creation of a scour hole can result in a significant reduction in the size of the riprap stone required compared to a flat riprap apron.

The design of preformed scour holes is based upon research conducted by Colorado State University and discussed in detail in Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, (Reference 2.15). The depth of the depression for the preformed scour hole is based on the flow velocity and depth at the culvert outlet, and the size of the riprap used to line the depression.

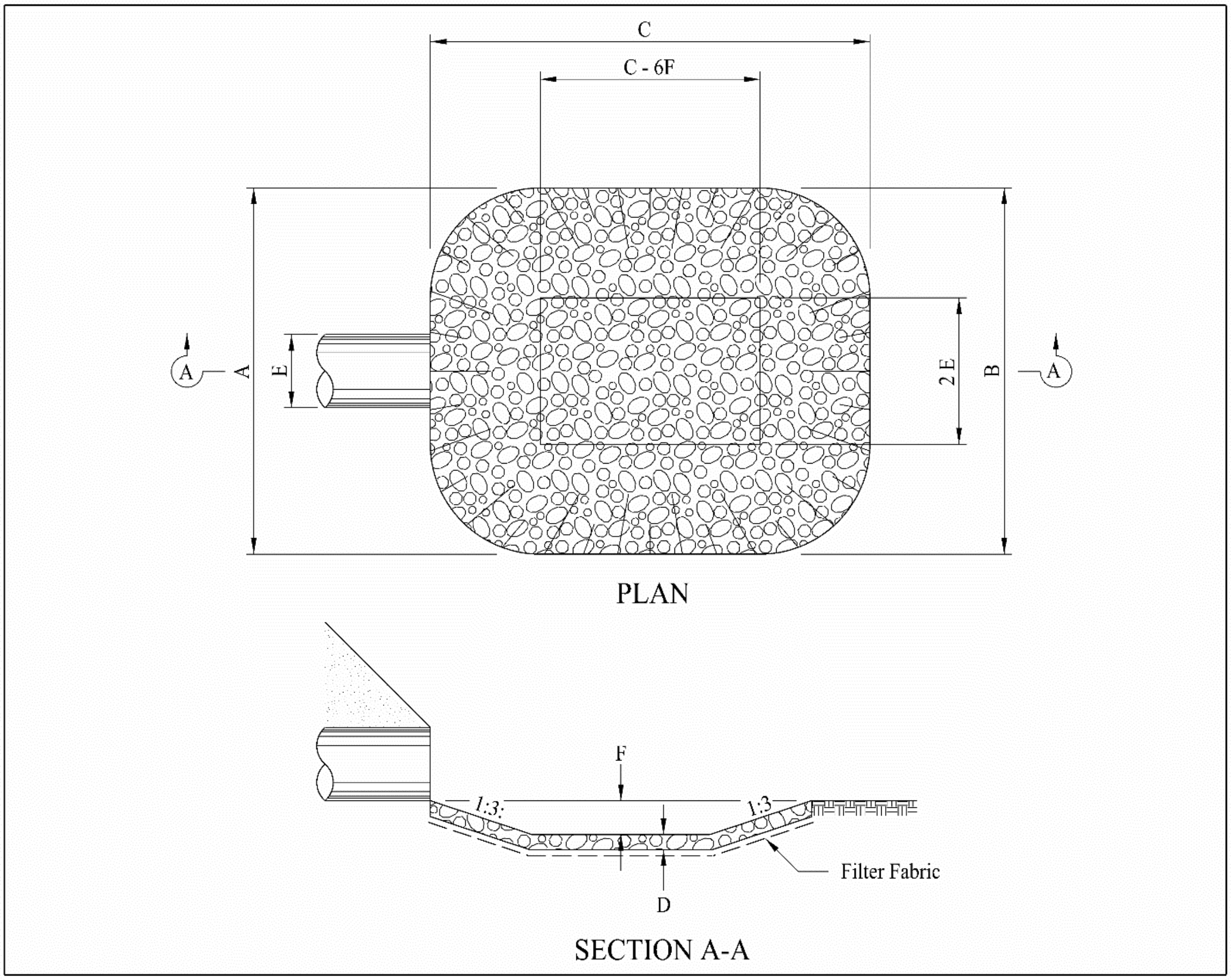


Exhibit 2.29 Preformed Scour Hole

Dimensions for a preformed scour hole basin are shown in EXHIBIT 2.29 and can be determined from the following equations:

A =	Basin inlet width, ft. (m)	=	2E + 6F
B =	Basin outlet width, ft. (m)	=	2E + 6F
C =	Basin length, ft. (m)	=	Greater of 10F or 3E
D =	Thickness of riprap lining, ft. (m)	=	Greater of $2D_{50}$ or $1.5D_{max}$
E =	Culvert diameter or span, ft. (m)	=	E
F =	Basin depression, ft. (m)	=	As described in Eq. 2.8

Empirical equations developed for the hydraulic design of preformed scour holes are presented on the next page. The equations are applicable to both circular and rectangular culverts flowing full or partly full, and where the ratio of the downstream tailwater depth to the culvert outlet depth is less than 0.75 ($TW/Y_0 < 0.75$). For design considerations where $TW/Y_0 > 0.75$, refer to Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, (Reference 2.15).

Determine the following culvert parameters:

TW	=	Tail water depth above culvert invert, ft. (m)	
Y_0	=	Depth of flow at culvert outlet, ft. (m)	
Y_e	=	Brink depth at culvert outlet, ft. (m)	
	=	Y_0 for box culverts, or	
	=	$(A \div 2)^{0.5}$ for pipe culverts,	Eq. 2.5
V_0	=	Velocity of flow at culvert outlet, ft./s (m/s)	
Fr	=	Froude Number = $V_0 \div (g\ Y_e)^{0.5}$	Eq. 2.6

Where: A = Flow Area at outlet, (See EXHIBIT 2.30)
g = Acceleration of Gravity = 32.2 ft./s^2 (9.81 m/s^2)

Select the desired riprap size:

d_{50}	=	Median stone diameter desired for scour hole, ft. (m)
	=	0.77 ft. (230 mm) – Type A Rock Riprap
	=	1.02 ft. (310 mm) – Type B Rock Riprap
	=	1.10 ft. (340 mm) – Broken Concrete Riprap
	=	1.28 ft. (390 mm) – Type C Rock Riprap

To calculate the Basin Depression, F:

Step 1. Using the d_{50}/Y_e and Fr values determined above, obtain the value F/Y_e either from the chart in EXHIBIT 2.31 or from Equations 2.7a through 2.7f given below, (which were used to derive the chart).

$F/Y_e = (3.0864 \times Fr) - 2.0062$,	for $0.10 \le (d_{50}/Y_e) \le 0.20$	Eq. 2.7a
$F/Y_e = (1.8519 \times Fr) - 1.5370$,	for $0.21 \le (d_{50}/Y_e) \le 0.30$	Eq. 2.7b
$F/Y_e = (1.5432 \times Fr) - 1.4198$,	for $0.31 \le (d_{50}/Y_e) \le 0.40$	Eq. 2.7c
$F/Y_e = (1.3514 \times Fr) - 1.4189$,	for $0.41 \le (d_{50}/Y_e) \le 0.50$	Eq. 2.7d
$F/Y_e = (1.3053 \times Fr) - 1.5534$,	for $0.51 \le (d_{50}/Y_e) \le 0.60$	Eq. 2.7e
$F/Y_e = (1.1719 \times Fr) - 1.5352$,	for $0.61 \le (d_{50}/Y_e) \le 0.70$	Eq. 2.7f

Step 2. To determine basin depth, ft. (m):

$F = (F/Y_e) \times Y_e$ Eq. 2.8

Step 3. Check to determine if the basin depth to riprap size meets $2 < (F/d_{50}) < 4$; if not, select new riprap size and repeat steps 1 through 3 until riprap size meets the F/d_{50} requirement.

If the necessary riprap size exceeds the d_{50} for Class C Rock Riprap, consider other energy dissipation methods, (See Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, Reference 2.15.

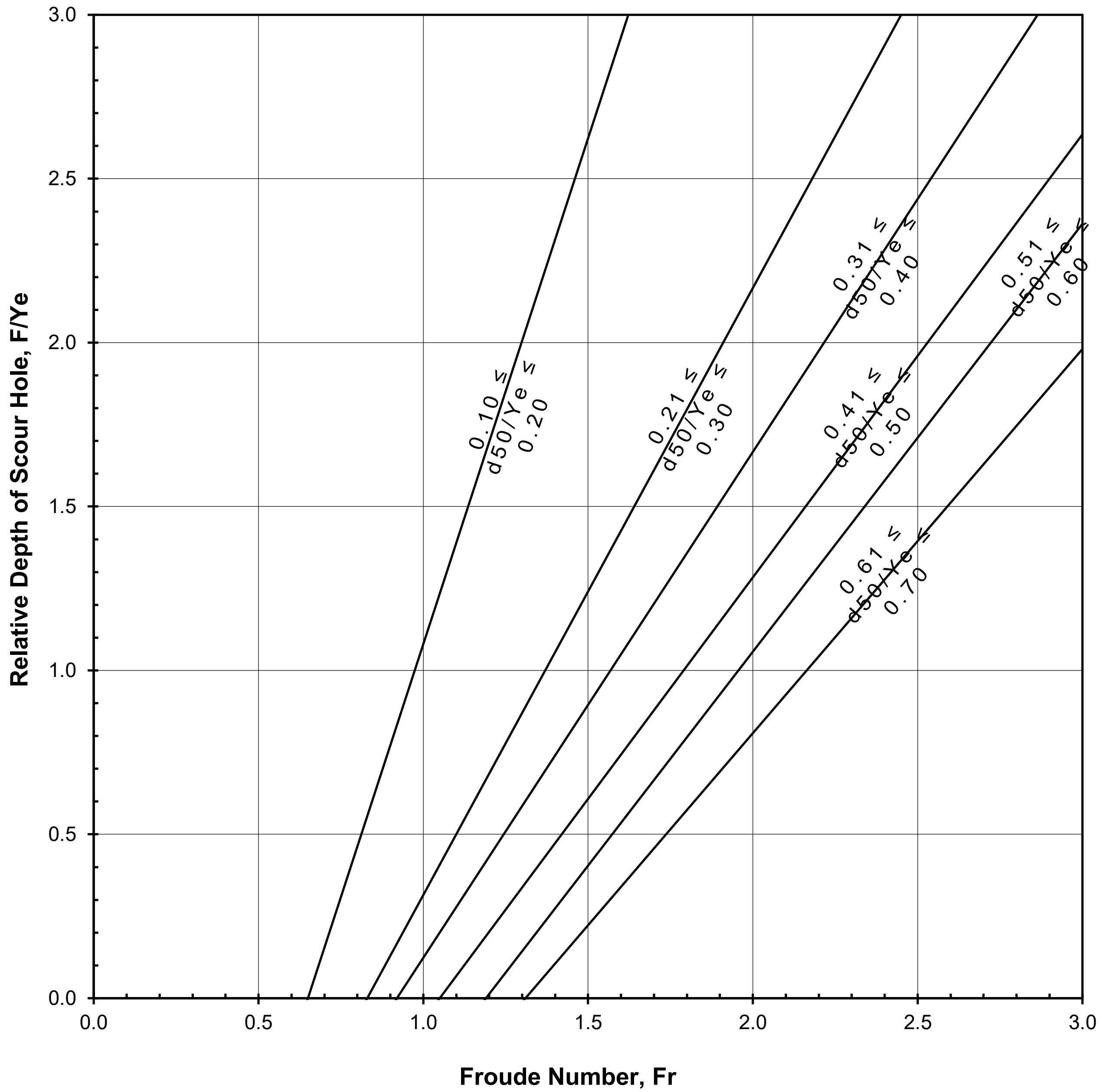


Exhibit 2.30 Relative Depth of Scour Hole vs Froude Number of Culvert Outlet Flow (Source: Modified from Figure X1-2 of Reference 2.15)

d/D	A/D^2	Flow Area (sq.ft.) for Given Pipe Diameter														
		15-inch	18-inch	24-inch	30-inch	36-inch	42-inch	48-inch	54-inch	60-inch	66-inch	72-inch	78-inch	84-inch	90-inch	96-inch
0.01	0.0013	0.0020	0.003	0.005	0.008	0.012	0.016	0.021	0.026	0.033	0.039	0.047	0.055	0.064	0.073	0.083
0.02	0.0037	0.006	0.008	0.015	0.023	0.033	0.045	0.059	0.075	0.093	0.112	0.133	0.156	0.181	0.208	0.237
0.03	0.0069	0.011	0.016	0.028	0.043	0.062	0.085	0.110	0.140	0.173	0.209	0.248	0.292	0.338	0.388	0.442
0.04	0.0105	0.016	0.024	0.042	0.066	0.095	0.129	0.168	0.213	0.263	0.318	0.378	0.444	0.515	0.591	0.672
0.05	0.0147	0.023	0.033	0.059	0.092	0.132	0.180	0.235	0.298	0.368	0.445	0.529	0.621	0.720	0.827	0.941
0.06	0.0192	0.030	0.043	0.077	0.120	0.173	0.235	0.307	0.389	0.480	0.581	0.691	0.811	0.941	1.080	1.229
0.07	0.0242	0.038	0.054	0.097	0.151	0.218	0.296	0.387	0.490	0.605	0.732	0.871	1.022	1.186	1.361	1.549
0.08	0.0294	0.046	0.066	0.118	0.184	0.265	0.360	0.470	0.595	0.735	0.889	1.058	1.242	1.441	1.654	1.882
0.09	0.0350	0.055	0.079	0.140	0.219	0.315	0.429	0.560	0.709	0.875	1.059	1.260	1.479	1.715	1.969	2.240
0.10	0.0409	0.064	0.092	0.164	0.256	0.368	0.501	0.654	0.828	1.023	1.237	1.472	1.728	2.004	2.301	2.618
0.11	0.0470	0.073	0.106	0.188	0.294	0.423	0.576	0.752	0.952	1.175	1.422	1.692	1.986	2.303	2.644	3.008
0.12	0.0534	0.083	0.120	0.214	0.334	0.481	0.654	0.854	1.081	1.335	1.615	1.922	2.256	2.617	3.004	3.418
0.13	0.0600	0.094	0.135	0.240	0.375	0.540	0.735	0.960	1.215	1.500	1.815	2.160	2.535	2.940	3.375	3.840
0.14	0.0668	0.104	0.150	0.267	0.418	0.601	0.818	1.069	1.353	1.670	2.021	2.405	2.822	3.273	3.758	4.275
0.15	0.0739	0.115	0.166	0.296	0.462	0.665	0.905	1.182	1.496	1.848	2.235	2.660	3.122	3.621	4.157	4.730
0.16	0.0811	0.127	0.182	0.324	0.507	0.730	0.993	1.298	1.642	2.028	2.453	2.920	3.426	3.974	4.562	5.190
0.17	0.0885	0.138	0.199	0.354	0.553	0.797	1.084	1.416	1.792	2.213	2.677	3.186	3.739	4.337	4.978	5.664
0.18	0.0961	0.150	0.216	0.384	0.601	0.865	1.177	1.538	1.946	2.403	2.907	3.460	4.060	4.709	5.406	6.150
0.19	0.1039	0.162	0.234	0.416	0.649	0.935	1.273	1.662	2.104	2.598	3.143	3.740	4.390	5.091	5.844	6.650
0.20	0.1118	0.175	0.252	0.447	0.699	1.006	1.370	1.789	2.264	2.795	3.382	4.025	4.724	5.478	6.289	7.155
0.21	0.1199	0.187	0.270	0.480	0.749	1.079	1.469	1.918	2.428	2.998	3.627	4.316	5.066	5.875	6.744	7.674
0.22	0.1281	0.200	0.288	0.512	0.801	1.153	1.569	2.050	2.594	3.203	3.875	4.612	5.412	6.277	7.206	8.198
0.23	0.1365	0.213	0.307	0.546	0.853	1.229	1.672	2.184	2.764	3.413	4.129	4.914	5.767	6.689	7.678	8.736
0.24	0.1449	0.226	0.326	0.580	0.906	1.304	1.775	2.318	2.934	3.623	4.383	5.216	6.122	7.100	8.151	9.274
0.25	0.1535	0.240	0.345	0.614	0.959	1.382	1.880	2.456	3.108	3.838	4.643	5.526	6.485	7.522	8.634	9.824
0.26	0.1623	0.254	0.365	0.649	1.014	1.461	1.988	2.597	3.287	4.058	4.910	5.843	6.857	7.953	9.129	10.387
0.27	0.1711	0.267	0.385	0.684	1.069	1.540	2.096	2.738	3.465	4.278	5.176	6.160	7.229	8.384	9.624	10.950
0.28	0.1800	0.281	0.405	0.720	1.125	1.620	2.205	2.880	3.645	4.500	5.445	6.480	7.605	8.820	10.125	11.520
0.29	0.1890	0.295	0.425	0.756	1.181	1.701	2.315	3.024	3.827	4.725	5.717	6.804	7.985	9.261	10.631	12.096
0.30	0.1982	0.310	0.446	0.793	1.239	1.784	2.428	3.171	4.014	4.955	5.996	7.135	8.374	9.712	11.149	12.685
0.31	0.2074	0.324	0.467	0.830	1.296	1.867	2.541	3.318	4.200	5.185	6.274	7.466	8.763	10.163	11.666	13.274
0.32	0.2167	0.339	0.488	0.867	1.354	1.950	2.655	3.467	4.388	5.418	6.555	7.801	9.156	10.618	12.189	13.869
0.33	0.2260	0.353	0.509	0.904	1.413	2.034	2.769	3.616	4.577	5.650	6.837	8.136	9.549	11.074	12.713	14.464
0.34	0.2355	0.368	0.530	0.942	1.472	2.120	2.885	3.768	4.769	5.888	7.124	8.478	9.950	11.540	13.247	15.072
0.35	0.2450	0.383	0.551	0.980	1.531	2.205	3.001	3.920	4.961	6.125	7.411	8.820	10.351	12.005	13.781	15.680
0.36	0.2546	0.398	0.573	1.018	1.591	2.291	3.119	4.074	5.156	6.365	7.702	9.166	10.757	12.475	14.321	16.294
0.37	0.2642	0.413	0.594	1.057	1.651	2.378	3.236	4.227	5.350	6.605	7.992	9.511	11.162	12.946	14.861	16.909
0.38	0.2739	0.428	0.616	1.096	1.712	2.465	3.355	4.382	5.546	6.848	8.285	9.860	11.572	13.421	15.407	17.530
0.39	0.2836	0.443	0.638	1.134	1.773	2.552	3.474	4.538	5.743	7.090	8.579	10.210	11.982	13.896	15.953	18.150
0.40	0.2934	0.458	0.660	1.174	1.834	2.641	3.594	4.694	5.941	7.335	8.875	10.562	12.396	14.377	16.504	18.778
0.41	0.3032	0.474	0.682	1.213	1.895	2.729	3.714	4.851	6.140	7.580	9.172	10.915	12.810	14.857	17.055	19.405
0.42	0.3130	0.489	0.704	1.252	1.956	2.817	3.834	5.008	6.338	7.825	9.468	11.268	13.224	15.337	17.606	20.032
0.43	0.3229	0.505	0.727	1.292	2.018	2.906	3.956	5.166	6.539	8.073	9.768	11.624	13.643	15.822	18.163	20.666
0.44	0.3328	0.520	0.749	1.331	2.080	2.995	4.077	5.325	6.739	8.320	10.067	11.981	14.061	16.307	18.720	21.299
0.45	0.3428	0.536	0.771	1.371	2.143	3.085	4.199	5.485	6.942	8.570	10.370	12.341	14.483	16.797	19.283	21.939
0.46	0.3527	0.551	0.794	1.411	2.204	3.174	4.321	5.643	7.142	8.818	10.669	12.697	14.902	17.282	19.839	22.573
0.47	0.3627	0.567	0.816	1.451	2.267	3.264	4.443	5.803	7.345	9.068	10.972	13.057	15.324	17.772	20.402	23.213
0.48	0.3727	0.582	0.839	1.491	2.329	3.354	4.566	5.963	7.547	9.318	11.274	13.417	15.747	18.262	20.964	23.853
0.49	0.3827	0.598	0.861	1.531	2.392	3.444	4.688	6.123	7.750	9.568	11.577	13.777	16.169	18.752	21.527	24.493
0.50	0.3927	0.614	0.884	1.571	2.454	3.534	4.811	6.283	7.952	9.818	11.879	14.137	16.592	19.242	22.089	25.133

Exhibit 2.31a Flow Area at Culvert Outlet for Relative Depth of Flow and Pipe Diameter

d/D	A/D^2	Flow Area (sq.ft.) for Given Pipe Diameter														
		15-inch	18-inch	24-inch	30-inch	36-inch	42-inch	48-inch	54-inch	60-inch	66-inch	72-inch	78-inch	84-inch	90-inch	96-inch
0.51	0.4027	0.629	0.906	1.611	2.517	3.624	4.933	6.443	8.155	10.068	12.182	14.497	17.014	19.732	22.652	25.773
0.52	0.4127	0.645	0.929	1.651	2.579	3.714	5.056	6.603	8.357	10.318	12.484	14.857	17.437	20.222	23.214	26.413
0.53	0.4227	0.660	0.951	1.691	2.642	3.804	5.178	6.763	8.560	10.568	12.787	15.217	17.859	20.712	23.777	27.053
0.54	0.4327	0.676	0.974	1.731	2.704	3.894	5.301	6.923	8.762	10.818	13.089	15.577	18.282	21.202	24.339	27.693
0.55	0.4426	0.692	0.996	1.770	2.766	3.983	5.422	7.082	8.963	11.065	13.389	15.934	18.700	21.687	24.896	28.326
0.56	0.4526	0.707	1.018	1.810	2.829	4.073	5.544	7.242	9.165	11.315	13.691	16.294	19.122	22.177	25.459	28.966
0.57	0.4625	0.723	1.041	1.850	2.891	4.163	5.666	7.400	9.366	11.563	13.991	16.650	19.541	22.663	26.016	29.600
0.58	0.4724	0.738	1.063	1.890	2.953	4.252	5.787	7.558	9.566	11.810	14.290	17.006	19.959	23.148	26.573	30.234
0.59	0.4822	0.753	1.085	1.929	3.014	4.340	5.907	7.715	9.765	12.055	14.587	17.359	20.373	23.628	27.124	30.861
0.60	0.4920	0.769	1.107	1.968	3.075	4.428	6.027	7.872	9.963	12.300	14.883	17.712	20.787	24.108	27.675	31.488
0.61	0.5018	0.784	1.129	2.007	3.136	4.516	6.147	8.029	10.161	12.545	15.179	18.065	21.201	24.588	28.226	32.115
0.62	0.5115	0.799	1.151	2.046	3.197	4.604	6.266	8.184	10.358	12.788	15.473	18.414	21.611	25.064	28.772	32.736
0.63	0.5212	0.814	1.173	2.085	3.258	4.691	6.385	8.339	10.554	13.030	15.766	18.763	22.021	25.539	29.318	33.357
0.64	0.5308	0.829	1.194	2.123	3.318	4.777	6.502	8.493	10.749	13.270	16.057	19.109	22.426	26.009	29.858	33.971
0.65	0.5404	0.844	1.216	2.162	3.378	4.864	6.620	8.646	10.943	13.510	16.347	19.454	22.832	26.480	30.398	34.586
0.66	0.5499	0.859	1.237	2.200	3.437	4.949	6.736	8.798	11.135	13.748	16.634	19.796	23.233	26.945	30.932	35.194
0.67	0.5594	0.874	1.259	2.238	3.496	5.035	6.853	8.950	11.328	13.985	16.922	20.138	23.635	27.411	31.466	35.802
0.68	0.5687	0.889	1.280	2.275	3.554	5.118	6.967	9.099	11.516	14.218	17.203	20.473	24.028	27.866	31.989	36.397
0.69	0.5780	0.903	1.301	2.312	3.613	5.202	7.081	9.248	11.705	14.450	17.485	20.808	24.421	28.322	32.513	36.992
0.70	0.5872	0.918	1.321	2.349	3.670	5.285	7.193	9.395	11.891	14.680	17.763	21.139	24.809	28.773	33.030	37.581
0.71	0.5964	0.932	1.342	2.386	3.728	5.368	7.306	9.542	12.077	14.910	18.041	21.470	25.198	29.224	33.548	38.170
0.72	0.6054	0.946	1.362	2.422	3.784	5.449	7.416	9.686	12.259	15.135	18.313	21.794	25.578	29.665	34.054	38.746
0.73	0.6143	0.960	1.382	2.457	3.839	5.529	7.525	9.829	12.440	15.358	18.583	22.115	25.954	30.101	34.554	39.315
0.74	0.6231	0.974	1.402	2.492	3.894	5.608	7.633	9.970	12.618	15.578	18.849	22.432	26.326	30.532	35.049	39.878
0.75	0.6319	0.987	1.422	2.528	3.949	5.687	7.741	10.110	12.796	15.798	19.115	22.748	26.698	30.963	35.544	40.442
0.76	0.6405	1.001	1.441	2.562	4.003	5.765	7.846	10.248	12.970	16.013	19.375	23.058	27.061	31.385	36.028	40.992
0.77	0.6489	1.014	1.460	2.596	4.056	5.840	7.949	10.382	13.140	16.223	19.629	23.360	27.416	31.796	36.501	41.530
0.78	0.6573	1.027	1.479	2.629	4.108	5.916	8.052	10.517	13.310	16.433	19.883	23.663	27.771	32.208	36.973	42.067
0.79	0.6655	1.040	1.497	2.662	4.159	5.990	8.152	10.648	13.476	16.638	20.131	23.958	28.117	32.610	37.434	42.592
0.80	0.6736	1.053	1.516	2.694	4.210	6.062	8.252	10.778	13.640	16.840	20.376	24.250	28.460	33.006	37.890	43.110
0.81	0.6815	1.065	1.533	2.726	4.259	6.134	8.348	10.904	13.800	17.038	20.615	24.534	28.793	33.394	38.334	43.616
0.82	0.6893	1.077	1.551	2.757	4.308	6.204	8.444	11.029	13.958	17.233	20.851	24.815	29.123	33.776	38.773	44.115
0.83	0.6969	1.089	1.568	2.788	4.356	6.272	8.537	11.150	14.112	17.423	21.081	25.088	29.444	34.148	39.201	44.602
0.84	0.7043	1.100	1.585	2.817	4.402	6.339	8.628	11.269	14.262	17.608	21.305	25.355	29.757	34.511	39.617	45.075
0.85	0.7115	1.112	1.601	2.846	4.447	6.404	8.716	11.384	14.408	17.788	21.523	25.614	30.061	34.864	40.022	45.536
0.86	0.7186	1.123	1.617	2.874	4.491	6.467	8.803	11.498	14.552	17.965	21.738	25.870	30.361	35.211	40.421	45.990
0.87	0.7254	1.133	1.632	2.902	4.534	6.529	8.886	11.606	14.689	18.135	21.943	26.114	30.648	35.545	40.804	46.426
0.88	0.7320	1.144	1.647	2.928	4.575	6.588	8.967	11.712	14.823	18.300	22.143	26.352	30.927	35.868	41.175	46.848
0.89	0.7384	1.154	1.661	2.954	4.615	6.646	9.045	11.814	14.953	18.460	22.337	26.582	31.197	36.182	41.535	47.258
0.90	0.7445	1.163	1.675	2.978	4.653	6.701	9.120	11.912	15.076	18.613	22.521	26.802	31.455	36.481	41.878	47.648
0.91	0.7504	1.173	1.688	3.002	4.690	6.754	9.192	12.006	15.196	18.760	22.700	27.014	31.704	36.770	42.210	48.026
0.92	0.7560	1.181	1.701	3.024	4.725	6.804	9.261	12.096	15.309	18.900	22.869	27.216	31.941	37.044	42.525	48.384
0.93	0.7612	1.189	1.713	3.045	4.758	6.851	9.325	12.179	15.414	19.030	23.026	27.403	32.161	37.299	42.818	48.717
0.94	0.7662	1.197	1.724	3.065	4.789	6.896	9.386	12.259	15.516	19.155	23.178	27.583	32.372	37.544	43.099	49.037
0.95	0.7707	1.204	1.734	3.083	4.817	6.936	9.441	12.331	15.607	19.268	23.314	27.745	32.562	37.764	43.352	49.325
0.96	0.7749	1.211	1.744	3.100	4.843	6.974	9.493	12.398	15.692	19.373	23.441	27.896	32.740	37.970	43.588	49.594
0.97	0.7785	1.216	1.752	3.114	4.866	7.007	9.537	12.456	15.765	19.463	23.550	28.026	32.892	38.147	43.791	49.824
0.98	0.7817	1.221	1.759	3.127	4.886	7.035	9.576	12.507	15.829	19.543	23.646	28.141	33.027	38.303	43.971	50.029
0.99	0.7841	1.225	1.764	3.136	4.901	7.057	9.605	12.546	15.878	19.603	23.719	28.228	33.128	38.421	44.106	50.182
1.00	0.7854	1.227	1.767	3.142	4.909	7.069	9.621	12.566	15.904	19.635	23.758	28.274	33.183	38.485	44.179	50.266

Exhibit 2.31b Flow Area at Culvert Outlet for Relative Depth of Flow and Pipe Diameter

8. REFERENCES

2.1 American Association of State Highway and Transportation Officials, Highway Drainage Guidelines, Washington, DC, 1992.

2.2 American Association of State Highway and Transportation Officials, Model Drainage Manual, Washington, DC, 1991.

2.3 American Association of State Highway and Transportation Officials, A Guide For Transportation Landscape and Environmental Design, 1991.

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DEPARTMENT OF TRANSPORTATION

The Nebraska Department of Transportation Drainage Design and Erosion Control Manual Chapter Three, "Stormwater Treatment", April 2018, has been approved for use.

Approved by: ______________________________ / 4-9-2018

Mike Owen, Roadway Design Engineer, P.E. Date

Approved by: ______________________________ / 4/20/2018

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The information contained in Chapter Three: Stormwater Treatment, dated April 20, 2018 has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Manual chapters and other reference material citations occurring since the latest publication of this chapter.

This chapter replaces Chapter Three: Stormwater Treatment, dated September 2013. The Nebraska Division of the FHWA approved this chapter for use on the National Highway System and other federal projects on April 20, 2018.

CHAPTER THREE
STORMWATER TREATMENT
WITHIN MUNICIPAL SEPARATE STORM SEWER SYSTEM (MS4) COMMUNITIES

This Chapter provides designers with the necessary guidance for incorporating Water Quality Stormwater Treatment Facilities (STF) into Nebraska Department of Transportation (**NDOT**) projects. Specifically, this chapter details the selection, placement and design of STFs.

Stormwater Treatment policies, procedures and guidelines are subject to amendment as conditions warrant. They are not intended to be nor do they establish legal standards. Special situations may call for variations from these requirements, subject to approval from the **Roadway Design Unit Head** (**Unit Head**) or **Assistant Design Engineer** (**ADE**). The proper documentation of drainage decisions is important for the purposes of project records and archiving.

SELECTED DEFINITIONS
(See the Glossary for additional information)

De minimis - De minimis means “of minimum importance”. It refers to something that is so small or trivial that law does not consider it and is often used to describe exemptions in government rules and regulations.

Ephemeral Stream - A stream that flows only during and immediately after precipitation events.

Impervious Surface - A hard surface area that prevents or retards the entry of water into the soil.

Intermittent Stream - An intermittent or seasonal stream is one that has a consistent base flow, but only for part of the year.

Land Disturbance - Areas of exposed, erodible soil, including stockpiles, that are within the limits of construction and that result from construction activities.

Linear Facility – A roadway.

MS4 Community - An Urbanized Area with a population of 10,000 or greater and a population density of at least 1,000 people/square mile (See Appendix O).

MS4 Permit - An MS4 Permit (NPDES Permit) is EPA’s program to control the discharge of pollutants to waters of the United States.

New Pavement - New Pavement is defined as an impervious surface which is placed in an area currently devoid of such surfacing, or the complete removal and replacement of existing surfacing with modification of the base and/or subgrade.

Non-Linear Facilities - Rest Areas, Maintenance Yard, Offices, etc.

Perennial Stream – A stream or river (channel) that has continuous flow in parts of its bed all year round during years of normal rainfall.

Receiving Water - Creeks, streams, rivers, lakes, estuaries, or other surface water bodies into which stormwater is discharged.

Stormwater Treatment Facility (STF) - A STF is a measure that is implemented to protect water quality and reduce potential for pollution associated with stormwater runoff.

Total Suspended Solids (TSS) - TSS is the weight of particles that are suspended in water.

Wetland - Areas that are inundated or saturated by surface or ground water at a frequency or duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs and similar areas.

ACRONYMS AND ABBREVIATIONS

3R	Resurfacing, Restoration, and Rehabilitation
ADE	Assistant Design Engineer in the Roadway Design Division
CN	Curve Number
DPO	Design Process Outline
EPA	Environmental Protection Agency
Form A	Stormwater Treatment within MS4 Communities / Form A – Project Evaluation (See Appendix L)
Form B	Stormwater Treatment within MS4 Communities / Form B – STFs (See Appendix M)
Form C	Stormwater Treatment within MS4 Communities / Form C – Maintenance (See Appendix Q)
LPA	Local Public Agency
MS4	Municipal Separate Storm Sewer System
NDEQ	Nebraska Department of Environmental Quality
NDOT	Nebraska Department of Transportation
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
PDD	Project Development Division
ROW	Right-of-Way
RDC	Roadside Development & Compliance Unit in PDD
STF	Stormwater Treatment Facility
T_C	Time of Concentration
TMDL	Total Maximum Daily Loads
TSS	Total Suspended Solids
Unit Head	Roadway Design Unit Head
WQV	Water Quality Volume

1. STORMWATER TREATMENT OBJECTIVE

Stormwater Treatment is a condition of **NDOT's** Municipal Separate Storm Sewer System (MS4) permit. The objective of stormwater treatment is to minimize the discharge of potential pollutants in the highway's post project stormwater runoff to waters of the state. The **NDOT** will accomplish this objective by evaluating projects and implementing stormwater STFs where appropriate.

STFs are a combination of permanent structural and/or non-structural best management practices (STFs) used to improve stormwater quality throughout the functional life of the roadway. Structural STFs include practices that remove pollutants from stormwater runoff by the settling of particulate matter, filtration, biological uptake, and soil adsorption. Examples include:

- Storage practices such as extended-detention ponds
- Filtration practices such as filter strips
- Grassed swales
- Bio-retention
- Sand filters and
- Infiltration practices such as infiltration basins and infiltration trenches

Non-structural measures are typically source control measures designed to reduce the level of contaminants before they are carried away in stormwater runoff. Examples include:

- Policies and ordinances that provide requirements and standards to direct growth to identified areas
- Protection of sensitive areas such as wetlands and riparian areas
- Maintaining and/or increasing open spaces
- Providing buffers along sensitive water bodies
- Minimize impervious surfaces, and
- Minimizing disturbance of soils and vegetation

2. LEGAL AND REGULATORY

2.A Municipal Separate Storm Sewer System (MS4) Permit

The **NDOT** is a regulated MS4 and is required to meet the conditions of its National Pollutant Discharge Elimination System (NPDES) MS4 permit. This permit is administered by the **Nebraska Department of Environmental Quality** (**NDEQ**) and requires the **NDOT** to:

- Develop and implement strategies which include a combination of structural and/or non-structural STFs
- Have a policy requiring the implementation of STFs to the extent allowable under State, Tribal or local law
- Ensure adequate long-term operation and maintenance of STFs

2.A.1 Local Public Agencies

A **Local Public Agency** (**LPA**) permitted as a MS4 operates under its own NPDES permit. Therefore the requirement to establish stormwater treatment controls is guided by that specific permit and is subject to review only by the **NDEQ** and the **Environment Protection Agency** (**EPA**). The **NDOT's** stormwater treatment program does not supersede a **LPA's** stormwater treatment program or act as a minimum standard, except when a **LPA** project is being constructed on a State or Federal Highway located within a MS4 community. In those instances, the **LPA** may utilize its own program as long as it meets the minimum requirements established in this Chapter. For additional information on **LPA** projects, contact the **Local Projects Section** of the **Materials and Research Division** of the **NDOT**.

2.B Total Maximum Daily Loads (TMDLs)

Under section 303(d) of the Clean Water Act (http://www.thecre.com/fedlaw/legal14water/cwa.htm), the **NDEQ** is required to compile a list of impaired waters that fail to meet the applicable water quality standards or cannot support their designated or existing uses. This list, known as the "303(d) list," is submitted to the **EPA** every two years. The **NDEQ** may then develop a TMDL for pollutants causing impairment of a water body on the list. Included within the TMDL is a treatment standard that is designed to reduce the level of the restricted pollutants to which all entities discharging into the stream must adhere.

Highways that have stormwater outfalls discharging into receiving waters for which TMDLs or other water quality requirements have been established may be subject to additional water quality treatment requirements. In addition, receiving water bodies that have a treatment standard based on TMDLs may require more stringent analysis and treatment regimens. Therefore, it is important to recognize all treatment requirements when designing projects. Additional information on the TMDL program and 303(d) list is provided on the **NDEQ** Website: (http://deq.ne.gov/NDEQProg.nsf/%24%24OpenDominoDocument.xsp?documentId=E238CC319E38A69386257CB500746DCD&action=openDocument).

The **Roadside Development & Compliance Unit** (**RDC**) in the **Project Development Division** (**PDD**) will notify project designers of any potential TMDLs after its Roadside Development Coordination review. This review is completed during the "Scoping Phase" (See the Design Process Outline (DPO), Ref. 3.1), and will be documented on the "Stormwater Treatment within MS4 Communities / Form A - Project Evaluation" (Form A), included in Appendix L. Form A will be the primary document to evaluate projects for STFs and other water quality requirements.

Form A is initiated by **RDC** who places it on OnBase, and will be finalized by the designer prior to the "Plan in Hand Phase" (See the DPO, Ref. 3.1). It is important that this form and other forms included in this chapter are completed and maintained with the project file.

2.B.1 Total Maximum Daily Loads (TMDLs) or Other Water Quality Requirements

Treatment must be provided within a TMDL watershed when discharging into receiving waters for which TMDLs or other water quality requirements have been established, and **NDOT** has been named as a contributor and assigned a Waste Load Allocation. An evaluation of the TMDLs or other water quality requirements will occur during Preliminary Project Evaluation on Form A.

2.C Platte River Depletion

In 2006, the **State of Nebraska** signed an agreement and enacted legislation to restrict water use in the Platte River basin in an effort to comply with the Endangered Species Act (http://www.fws.gov/laws/lawsdigest/esact.html). The goals of the restrictions are to reduce shortages, to target flows in the central Platte River, and to obtain and restore critical habitat for the "target species" (whooping crane, interior least tern, piping plover, and pallid sturgeon).

Stormwater treatment activities conducted within the regulated area may be subject to restrictions if they constitute new surface water or hydrologically connected groundwater actions which may affect the quantity or timing of water reaching the associated habitats of the target species. Two examples of such activities would be those that expose the groundwater table to the atmosphere and those that will impound water such as ponds and wetlands. If a project is being constructed within this regulated area, **RDC** will notify the designer during the Project Evaluation Process detailed in Section 3. Additional information on this program can be obtained by contacting the **Environmental Documents Unit** in **PDD**.

2.C.1 *De Minimis* Threshold for Platte River Species Depletions Consultations

The **U.S. Fish and Wildlife Service** has adopted a policy that water-related activities in the Platte River basin resulting in less than 0.1 acre-foot/year of depletions in flow to the nearest surface water tributary to the Platte River system do not affect the Platte River target species, and thus do not require consultation with the **U.S. Fish and Wildlife Service** for potential effects on those species.

Similarly, detention basins designed to detain runoff for less than 72 hours and temporary withdrawals of water (e.g., for hydrostatic pipeline testing) that return all the water to the same drainage basin within 30 days' time are considered to have no effect, and may not require consultation.

A *de minimis* determination is made only by the **Environmental Documents Unit** in **PDD**.

2.D Legal, Regulatory and Environmental Issues Related to Drainage

Many of the legal, regulatory and environmental issues identified in Chapter One: Drainage also pertain to activities completed in this Chapter. The designer should familiarize him/herself with the corresponding section in Chapter One.

2.E Designation of STFs

STFs are engineered stormwater treatment facilities and will be designated as such on both Design and ROW Plans. Many of these measures will capture and hold water for some period of time and may develop wetland characteristics. This could occur naturally or as a designed feature of the STF. Regular maintenance activities, changes to the STF design and/or relocation of the facility due to future construction may occur at any time at the discretion of the **NDOT** (See Sections 7.A.6.a, “Retention of ROW for STF” and 8.D, “Plan Labeling of STF”).

3. PROJECT EVALUATION PROCESS

This section provides guidance to evaluate a project for STFs. The entire process, outlined in this chapter, is graphically represented in EXHIBITS 3.1, 3.2 & 3.3.

A preliminary project evaluation will be completed for every project by **RDC** using the project criteria provided in Section 3.B. After receiving the preliminary project evaluation, the designer will complete a final project evaluation to determine if the project requires STFs. Coordination between **RDC** and **Roadway Design** will occur several times throughout the project schedule, as determined by the **Unit Head**, to address the stormwater treatment requirements. Specifically, this will occur at the “Environmental Coordination Meetings” (See **EXHIBIT A** of the DPO, Ref. 3.1) but communication can and should occur as needed throughout the project design. To document this process, Form A will be used.

3.A General Project Criteria

When all three criteria outlined below are met, the project must be evaluated for STFs.

1. Project Location – The project is located within or partly within the boundary of a regulated MS4 Community in Nebraska. See Appendix O for a list of regulated MS4s in Nebraska.

2. Project Size – The project results in a land disturbance $\geq$ 1 acre (including projects that disturb < 1 acre if part of a common plan of development).

3. Project Nature – The project is classified as a Resurfacing, Restoration and Rehabilitation (3R) project with a net increase of at least 5,000 square feet of New Pavement, or as a New and Reconstruction project.

New Pavement is defined as an impervious surface (a hard surface area that prevents or retards the entry of water into the soil, thus causing water to run off the surface in greater quantities and at an increased rate of flow) which is placed in an area currently devoid of such surfacing, or the complete removal and replacement of existing surfacing with modification of the base and/or subgrade.

3.A.1 3R and New and Reconstruction Projects

Stormwater treatment is not required unless a project meets the project nature criteria for 3R and New and Reconstruction as stated below.

- 3R projects must result in a net increase of at least 5,000 square feet of New Pavement. This includes, but is not limited to, adding turn lanes, paving shoulders, trench widening, driveways, sidewalks, and side streets. This also includes redevelopment work completed for non-linear facilities such as maintenance yards and rest areas that result in the net increase of at least 5,000 square feet of New Pavement.
- New and Reconstruction projects must result in the placement of New Pavement or Building(s). This also includes the development of new non-linear facilities such as maintenance yards and rest areas.

3.B Preliminary Project Evaluation

During the "Scoping Phase" (See the DPO, Ref. 3.1), **RDC** will complete a Preliminary Project Evaluation and will document this review in Clarity, Environmental Sub-Object Section for each project. If it is determined that an evaluation for STFs is not required for the project, the process will stop and no further consideration will be required. If the Preliminary Project Evaluation determines further review is needed, **RDC** will initiate Form A and will place the file on OnBase. A copy will also be forwarded to the roadway designer for a Final Project Evaluation as discussed in Section 3.C.

RDC will utilize the following criteria when completing the Preliminary Project Evaluation:

- **Are there TMDLs or other water quality requirements within project limits?**
  Construction projects that discharge into a receiving water for which a TMDL or other water quality requirement has been established may be subject to additional water quality regulations such as the Platte River Depletion Implementation Program.
- **Is the project within an MS4 area?**
  Projects and activities within MS4 areas may require the incorporation of STFs. Projects that cross a MS4 boundary may require STFs beyond the MS4 area where factors justify their use, such as watershed drainage, land use, etc.
- **Does the project disturb ≥ 1 acre of soil.**
  Any project that results in a land disturbance equal to or greater than 1 acre. Land disturbance includes any areas where the bare soil will be exposed to weather for any period of time. The one (1) acre value is compared to the cumulative total of exposed soil.
  - **Is the project part of a Common Plan of Development?**
    Projects that disturb less than 1 acre and are part of a larger Common Plan of Development whose total land disturbance activities are 1 acre or more are considered to meet the ≥ 1 acre disturbance criteria. In addition, the **DEQ** can designate projects as part of a common plan of development.
- **Is the project classified as 3R with ≥ 5000 sq. ft. of New Pavement or as New and Reconstruction?**
  Stormwater treatment may be required for 3R projects (with at least 5,000 square feet of New Pavement) and New and Reconstruction projects (where New Pavement or Building(s) are placed in areas currently devoid of such surfacing or which completely remove and replace existing pavement).

3.C Final Project Evaluation

If the Preliminary Project Evaluation determines that STFs are to be determined, the designer will perform a Final Project Evaluation of the project and shall complete the corresponding section of Form A. This should be completed early enough in the "Plan-In-Hand Phase" (See the DPO, Ref. 3.1) that conceptual STF designs can be completed and placed within the Plan-In-Hand plans. Upon completion of the Final Project Evaluation, the designer will forward Form A to his/her **Unit Head** with a recommendation for or against STFs in the project. The **Unit Head** will be responsible for reviewing the recommendation and signing off on the form. The Final Project Evaluation will consider the following items:

- **Is this project classified as 3R and has ≥ 5000 sq. ft. of New Pavement?**
  3R projects that result in the net increase of at least 5,000 square feet of New Pavement require assessment for stormwater treatment needs. Projects redeveloping non-linear facilities such as maintenance yards and rest areas which result in the net increase of at least 5,000 square feet of New Pavement or building(s) also require assessment.
- **Does the project disturb ≥ 1 acre of soil.**
  Any project which results in a land disturbance of equal to or greater than 1 acre. Land disturbance includes any areas where the bare soil will be exposed to weather for any period of time. The 1 acre value is compared to the cumulative total of exposed soil.
  - **Is the project part of a Common Plan of Development?**
    Projects that disturb less than 1 acre and are part of a larger Common Plan of Development whose total land disturbance activities are 1 acre or more are considered to meet the ≥ 1 acre disturbance criteria. In addition, the **DEQ** can designate projects as part of a common plan of development.

Upon completion of his/her review, the **Unit Head** will forward a copy of the signed Form A to **RDC** and return the original to the designer. If STFs are not required to be considered for the project, the form will be closed out and placed in the project file. If STFs need to be considered for the project, the designer shall complete the "Stormwater Treatment within MS4 Communities / Form B - STF" (Form B), included in Appendix M. Form B will be used to document the design decisions made under Section 4, "Stormwater Treatment Facility Design Process", of this Chapter.

3.D RDC Coordination with Adjacent MS4 Community

RDC will contact the adjacent MS4 Community as part of the preliminary project evaluation during the scoping phase, notifying them of the potential for STFs on the highway project. This contact will also be used to query the MS4 Community concerning preferences in STFs. Information provided to **RDC** by the MS4 Community will be provided to the designer on Form A.

3.E Change in Project Scope

A change in project scope that affects one or more of the criteria or considerations in Sections 3.A. and 3.B. requires a re-evaluation of the project for STFs. The designer must contact the **RDC Highway Environmental Program Manager** as soon as possible. A re-evaluation will be completed by **RDC** and **Roadway Design** and Form A will be updated to reflect any changes.

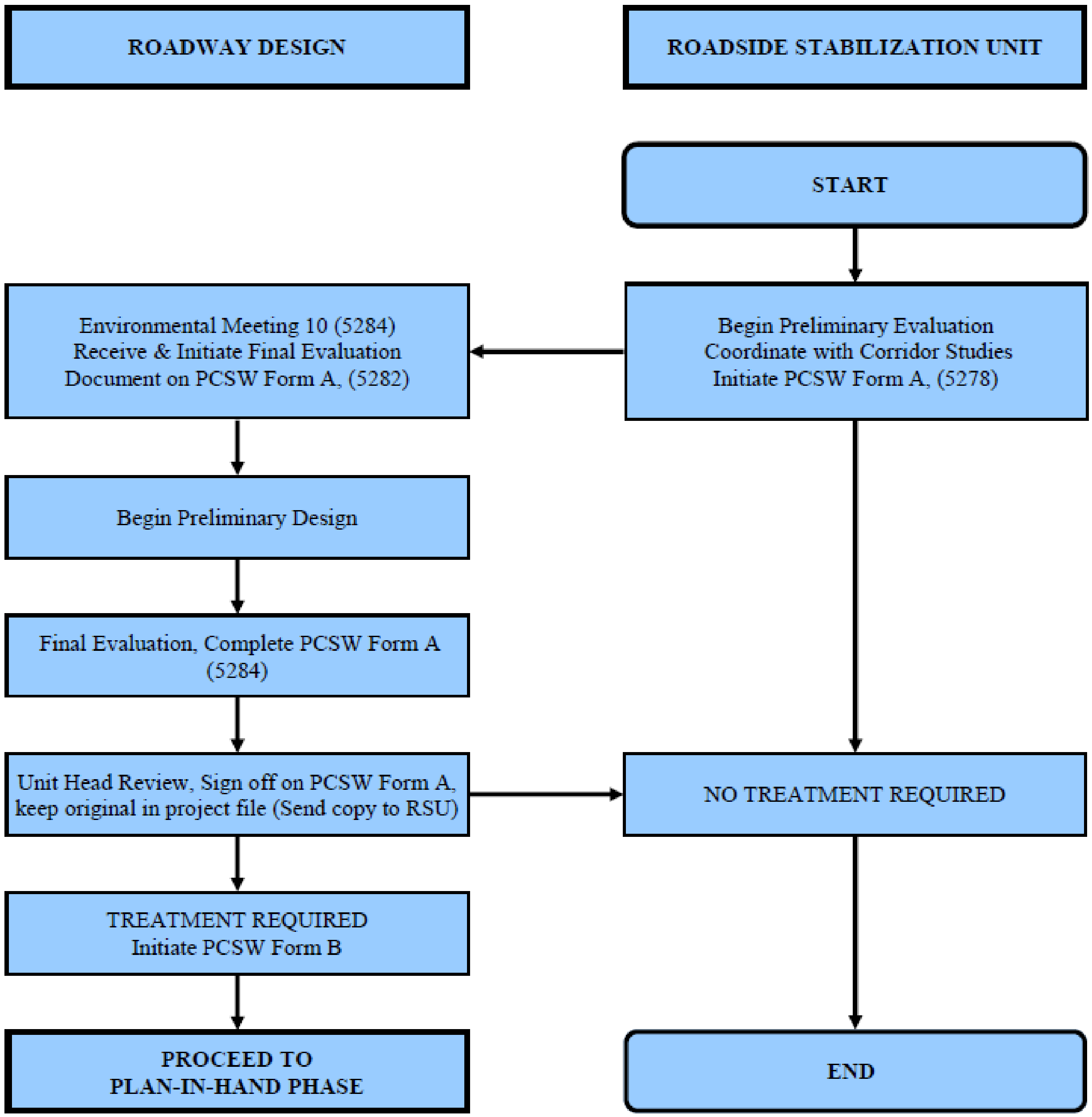


Exhibit 3.1: Stormwater Treatment Process Chart for Scoping Phase

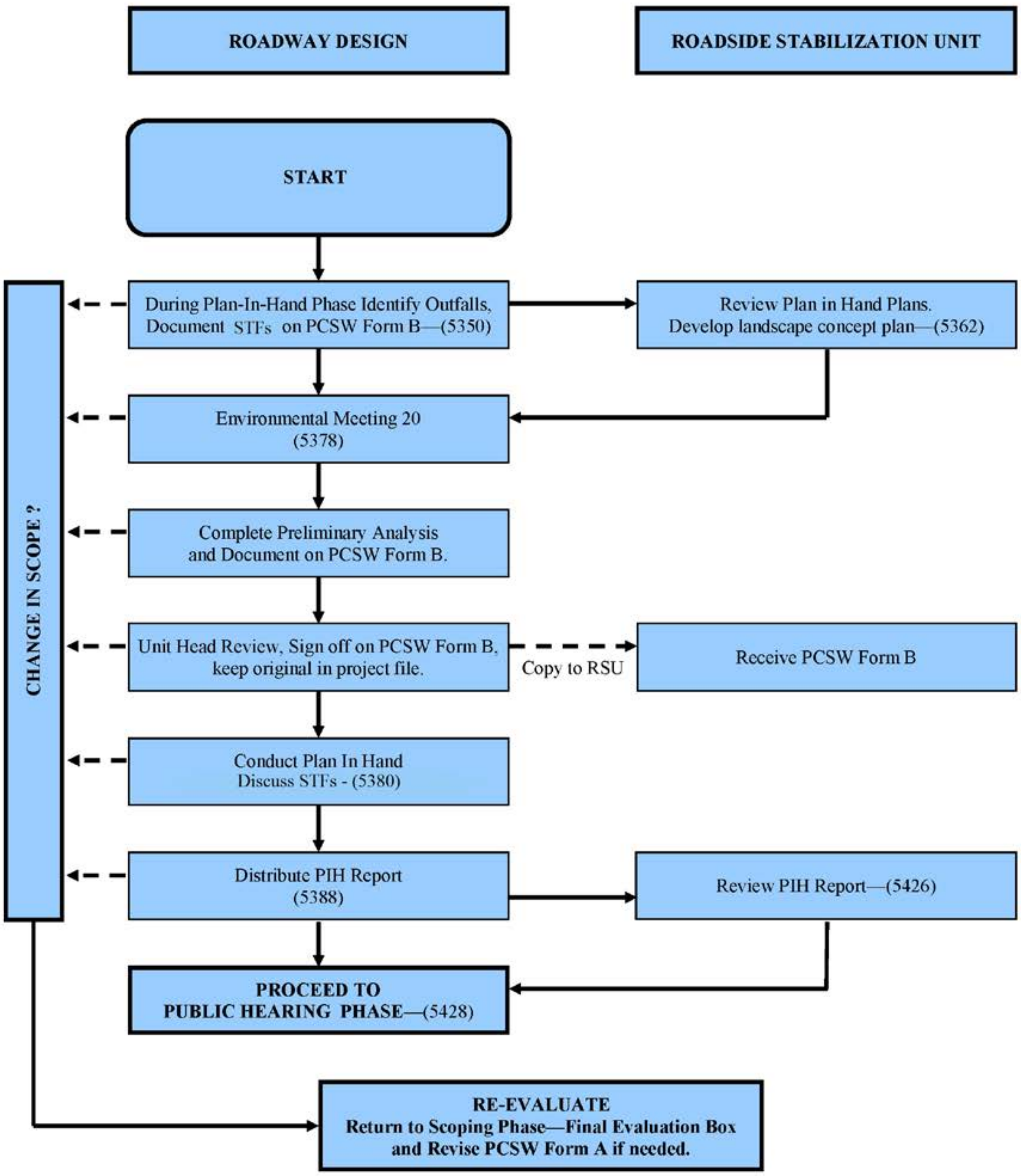


Exhibit 3.2: Stormwater Treatment Process Chart for Plan-In-Hand Phase

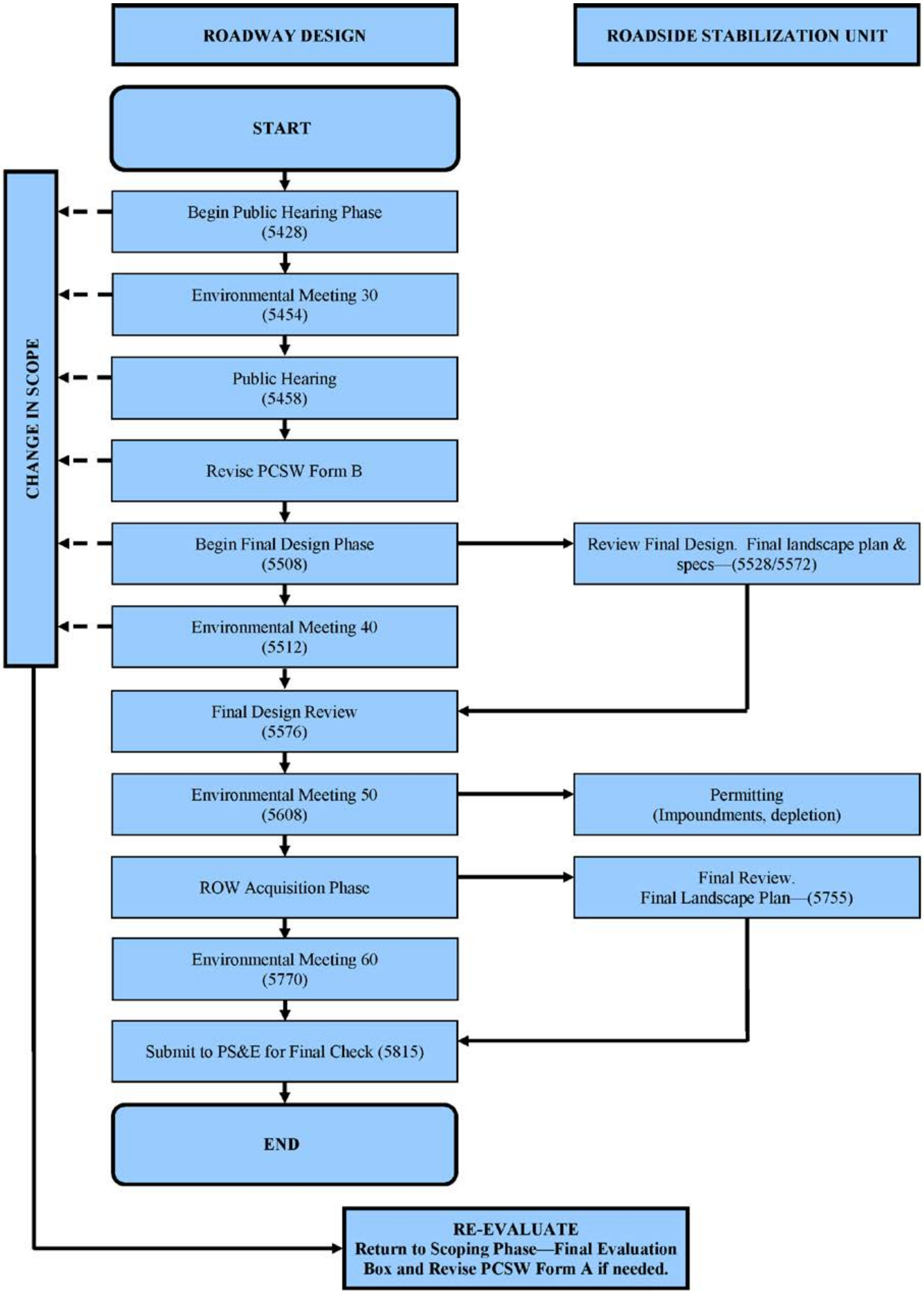


Exhibit 3.3: Stormwater Treatment Process Chart for Public Hearing and Final Design Phases

4. STORMWATER TREATMENT FACILITY DESIGN PROCESS

Stormwater Treatment Facility (STF) Design is a four step process. This process is outlined here and then discussed in detail in the following sections. The designer is responsible for completing and documenting the four steps on Form B.

4.A Plan-In-Hand Phase

During the "Plan-In-Hand Phase" (See the DPO, Ref. 3.1) of the project, the designer will complete the first three steps of the STF Design Process. These steps are:

Step 1. Identify all Stormwater Outfall locations and determine which of these qualify as Priority Stormwater Outfalls, as detailed in Section 5. Document Priority Outfalls on Form B.

Step 2. Calculate the Water Quality Volume and Discharge Rate at each Priority Stormwater Outfall location identified in Step 1, as detailed in Section 6. Document on Form B.

Step 3. Select appropriate STF (s) and complete preliminary design at each Priority Stormwater Outfall identified in Step 1, as detailed in Section 7. Document on Form B.

4.B Public Hearing Phase

During the "Public Hearing Design Phase" (See the DPO, Ref. 3.1) of the project, the designer will complete the final step in the STF Design Process. This step is:

Step 4. Complete design, as detailed in Section 8, including details for STF(s) at each Priority Stormwater Outfall identified in Step 1. Document any changes on Form B.

5. STORMWATER OUTFALLS

5.A Stormwater Outfalls

Step 1 in the STF design process is to complete an assessment of the highway's post project drainage and to identify all stormwater outfalls located both within the project limits and the MS4 boundary. All stormwater outfall locations so located must then either be recorded on Form B or labeled on a project map / plan sheet / aerial.

To complete this task it is important to understand what is considered a stormwater outfall, therefore definitions are provided along with the **NDOT** interpretation of those definitions.

Within this Chapter the terms outfall and priority outfall refer to points where flow discharges from State right-of-way as defined in Sections 5.A.1 and 5.B. The term discharge may be used in place of outfall and the term priority discharge may be used in place of priority outfall. These terms are not intended to meet the definitions of outfall and priority outfall as defined in **NDOT's** Illicit Discharge Detection and Elimination program.

5.A.1 Definitions

The **NDEQ** defines a stormwater outfall as:

> *"A point source at the point where a facility and/or municipal separate storm sewer discharges to waters of the state."*

The **NDOT** interprets this definition to mean that a stormwater outfall occurs anywhere that intentionally collected stormwater flow exits the Right-of-Way (ROW) and discharges to a water of the state.

The **NDEQ** defines Waters of the State as:

> *"Waters within the jurisdiction of the state including all streams, lakes, ponds, impounding reservoirs , marshes, wetlands, water courses, waterways, wells, springs, irrigation systems, drainage systems, and all other bodies or accumulation of water, surface and underground, natural or artificial, public or private, situated wholly or partly within or bordering upon the state."*

5.B Priority Stormwater Outfalls

Due to the linear nature of highway projects, outfall locations may occur at numerous locations along a project and may not directly discharge stormwater to waters of the state. In order to utilize **NDOT** resources as efficiently as possible, the stormwater treatment program will focus its efforts at Priority Stormwater Outfalls. The following definition shall be used to designate an outfall as a priority:

The **NDOT** defines priority stormwater outfalls as:

> Concentrated stormwater flow locations from areas with a net increase of at least 5,000 square feet of New Pavement (including bridge surfaces) directly discharging from State ROW to the following locations within the MS4 boundary:
>
> - Streams – Perennial and Intermittent,
> - Lakes and Ponds,
> - Wetlands,
> - Municipal Separate Storm Sewer System,
> - Ephemeral drainage that directly discharges to one of the above located beyond the ROW line and within the distance identified in Appendix N.

Utilizing these criteria, the designer documents which outfalls, listed on Form B or shown on the project map / plan sheet / aerial, are considered to be priority stormwater outfalls. Priority stormwater outfalls must be documented on Form B.

There are two separate 5000 sq. ft. thresholds that require distinction by the designer.

1. Preliminary project evaluation of 3R projects includes a 5000-sq. ft. of new pavement threshold as described in Section 3.A.
2. Determination of priority outfalls is based on a threshold of a 5000-sq. ft. of new pavement discharging from State ROW as described in this section.

EXHIBIT 3.3a shows the distinction between these thresholds within the **NDOT** MS4 process.

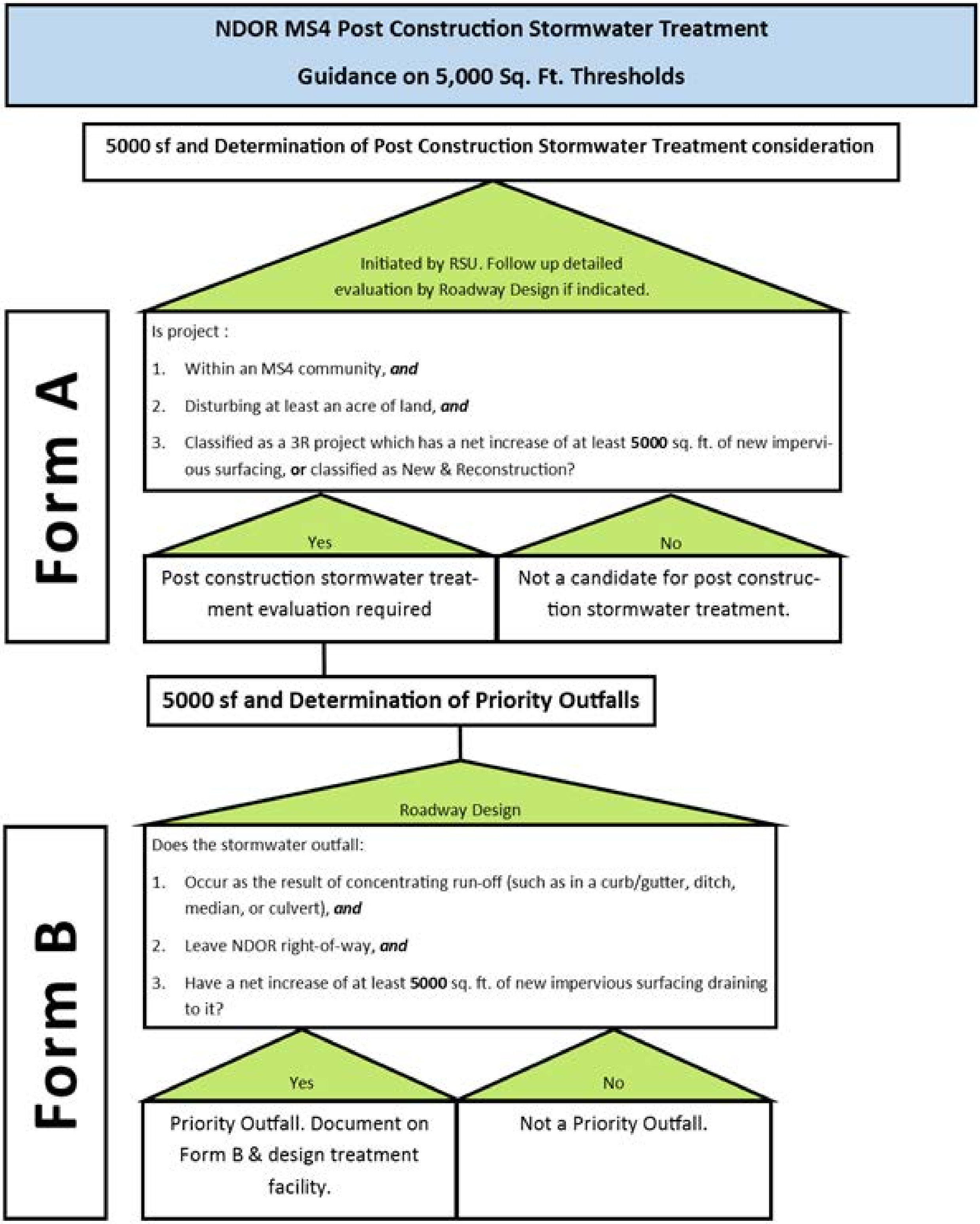


Exhibit 3.3a Guidance on 5000 Sq. Ft. Thresholds

5.B.1 Priority Stormwater Outfalls Off Project

Outfall locations may occur *on* or *off* the project site. Concentrated stormwater runoff from a project must be followed to the point that it leaves State ROW regardless of whether or not that point is within the project boundary. Once it leaves State ROW, *on* or *off* the project site, the determination is then made as to whether the outfall is a priority outfall. If it is determined that the outfall location is a priority outfall, treatment must be addressed per this Chapter, preferably at a treatment point on the project.

5.C Example Cases of Stormwater Outfalls

Case 1 The new highway pavement flows to the highway foreslope which drains to natural ground that continues to flow away from the highway. After the stormwater leaves the highway ROW it flows into the city street gutter and is collected by the city's storm sewer system.

Result: In Case 1 the stormwater discharge from the highway foreslope is not a stormwater outfall for the **NDOT** project; according to the definitions in Section 5.A.1 the stormwater needs to be intentionally collected before it exits the highway ROW to be a stormwater outfall. In this case the stormwater flows off the highway ROW in un-concentrated condition before it is collected (in the street's gutter).

Case 2 The new highway pavement flows to the highway ditch which drains to a low point and discharges off the ROW on to the adjacent field where it disperses (see upper left part of EXHIBIT 3.4).

Result: In Case 2 the stormwater discharge from the low point of the highway ditch is a stormwater outfall for the **NDOT** project, but is not a priority stormwater outfall. According to the definitions in Section 5.A.1 the low point of the ditch is a stormwater outfall; the stormwater is collected by the highway ditch and discharged in a concentrated manner from the highway ROW. However, the stormwater outfall does not meet the requirements given in Section 5.B for it to be a priority stormwater outfall. The stormwater disperses into the adjacent field and does not flow directly into a stream, lake, wetland, storm sewer or into a drainage which leads to one.

Case 3 The new highway pavement flows to the highway ditch and median drain which combine and discharge off the ROW into a ephemeral drainage swale that leads to a perennial stream (see upper right corner of EXHIBIT 3.4). The survey shows the swale to be 8 feet wide, has a grade of 0.8% and the distance to the stream is 180 feet. A total of 0.8 acres of new pavement drains to the median and ditch.

Result: In Case 3 the stormwater discharge from ROW to the drainage swale is a stormwater outfall. According to the definitions in Section 5.A.1 the stormwater collected by the highway ditch and median drain and discharged in a concentrated manner from the highway ROW make the discharge point a stormwater outfall. Because the discharge is to a ephemeral drainage swale that leads to a perennial stream, the table in Appendix N is used to determine if the stormwater outfall is a priority stormwater outfall.

To use the table in Appendix N, it is necessary to know the Swale Width and Grade, the Water Quality Volume (WQV) Discharge Rate, and the distance traveled in the swale before it reaches the stream.

- The swale width is given as 8 feet. This lies between the 10-foot and 5-foot swale values in the table - when the swale width lies between two table values round down - use the 5-foot width.
- The swale grade is given as 0.8%, between the 0.5% and 1% grades given in the table – when the swale grade lies between two table values round up – use the 1% grade.
- The WQV discharge rate is based on the area of new pavement. From EXHIBIT 3.5 it can be seen that the 0.8 acres of New Pavement generate a WQV peak discharge of 0.7 cfs. The 0.7 cfs discharge lies between the 0.45 cfs and 0.9 cfs values in the Appendix N table – when the WQV discharge rate lies between two table values round up – use the 0.9 cfs discharge rate.

Using the values determined in the bullet points above (a 5-feet wide, 180-feet long swale at 1% grade with 0.9 cfs WQV discharge), which when applied to the table in Appendix N, it can be determined that the outfall is a priority stormwater outfall. Based on Appendix N, the swale would need to exceed 225 feet in length to not be categorized as a priority stormwater outfall.

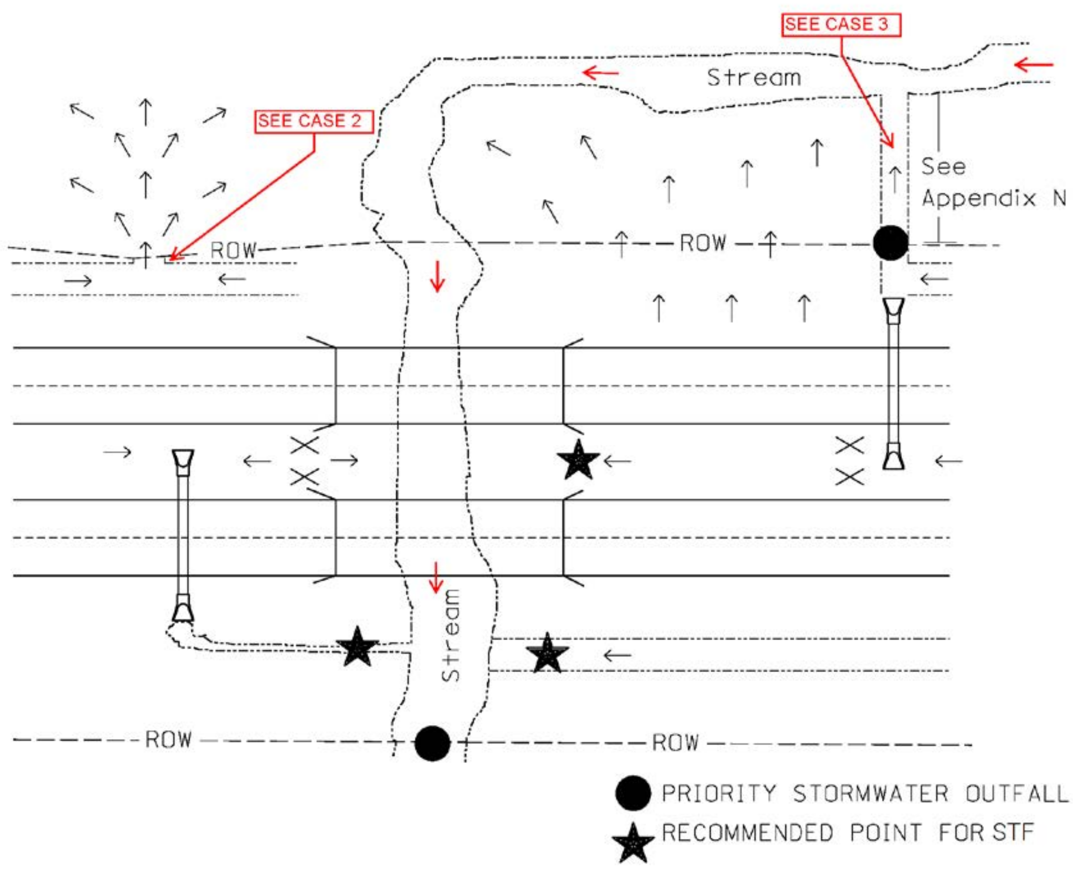


Exhibit 3.4: Examples of Priority Stormwater Outfall Locations

6. STF HYDROLOGY

Step 2 in the STF design process is to calculate the WQV and WQV Discharge Rate of the stormwater runoff from the Treatment Drainage Area, at each stormwater outfall classified as a priority in Step 1. To assist in this task, the **NDOT** has established definitions for the water quality volume and water quality discharge rate, which are provided below. Also provided is a short discussion of how those values were determined.

The designer determines the water quality volume and water quality discharge rate, during the plan-in-hand phase for each of the priority stormwater outfalls. These values are recorded on Form B.

6.A Water Quality Volume (WQV)

WQV is defined as the amount of storm water runoff from a given storm that should be captured and treated in order to remove a majority of storm water pollutants on an average annual basis.

The **NDOT** has determined that the WQV is the first one-half (½) inch of runoff from the Treatment Drainage Area (see Section 6.A.1). The calculation for this volume is provided in the following equation.

WQV = 0.5 inch x Treatment Drainage Area (ft^2) /12 = cu ft of Runoff Requiring Treatment (III.1)

The WQVs for drainage areas up to 5 acres have already been calculated by the **NDOT** and placed in EXHIBIT 3.5. The designer should use these values when sizing STF for these drainage areas.

6.A.1 Treatment Drainage Area

The Treatment Drainage Area is defined as the area of New Pavement placed on the project.

The Treatment Drainage Area may be increased due to run-on intermingling with the runoff from the areas defined above. Run-on is further discussed in Section 6.C.

6.A.2 Selection of Water Quality Volume

In establishing treatment criteria, the **NDOT** is following guidance provided in the EPA document National Management Measures to Control Nonpoint Source Pollution from Urban Areas (Ref. 3.2). This document recommends reducing the post development loadings of Total Suspended Solids (TSS) by a minimum of 80% of the average annual TSS loadings.

The 80% TSS removal criteria for STF selection is based on several factors:

- TSS is a measure of the concentrations of sediment and other particles suspended in water.
- TSS can be an indirect measure of other pollutants carried by runoff, because organic compounds, metals, and nutrients such as phosphorus are typically attached to sediment particles.
- Research has shown that many STFs or combinations or STFs can achieve the 80% removal goal.

The **NDOT** has determined that the 80% goal can be obtained by collecting and treating either the WQV discussed above, or the WQV Discharge Rate discussed below.

6.B Water Quality Volume Discharge Rate

The WQV Discharge Rate is the peak stormwater discharge generated by the water quality volume rainfall event using the **Natural Resources Conservation Service** (**NRCS**) Curve Number (CN) procedure. The water quality volume rainfall event is the 0.75 inch, 24-hour rainfall, assuming a CN value of 98 (pavement) and a Time of Concentration (Tc) of 5 minutes.

The WQV Peak Discharges for drainage areas up to 5 acres have already been calculated by the **NDOT** and placed in the following EXHIBIT 3.5. The highway designer should use these values when sizing STFs for these drainage areas.

To determine a WQV peak discharge for drainage areas greater than 5 acres, the **NRCS** CN procedure must be used. The designer can either assume the entire area is pavement with a CN value of 98 or use the weighted Q method of the CN procedure. The weighted Q method calculates the runoff from each land cover and soil complex individually and then sums the runoff peaks.

Example: A 10 acre residential neighborhood with ¼ acre lots in hydrologic soil group B soils.

Weighted Q Method
The residential neighborhood is assumed to have 38% impervious area, 3.8 acres at CN of 98, and 62% grass cover, 6.2 acres at CN of 61. Using the WQV rainfall event, 24-hour Rainfall = 0.75 inch and the assumed Tc = 5 minutes, calculate the peak discharge for each area (impervious area and grass cover) independently:

Area	Discharge
Impervious area	3.4 cfs
Grass area	0.02 cfs
Add Hydrographs	3.4 cfs (WQV Peak discharge)

Standard Method (Weighted CN Method)
The residential neighborhood is assumed to have 100% ¼ acre lots, 10 acres at CN of 75. Using the WQV rainfall event, 24-hour Rainfall = 0.75 inch and the assumed Tc = 5 minutes, calculate the peak discharge for entire area.

Area	Discharge
Residential ¼ Acre	0.0 cfs.

The weighted Q method calculates a WQV peak discharge as 3.4 cfs. This compares to the standard method of Weighted CN which calculates a discharge rate of 0 cfs. Empirical evidence and common sense both dictate that a 0.75-inch rainfall will generate at least some runoff, therefore the Weighted CN method can be rejected. Under small rainfall rates the Weighted Q method more accurately estimates the peak discharge rates that will be experienced.

Drainage Area (Acres)	WQV ½ inch Runoff (cubic feet)	WQV Discharge Rate (cfs)	Drainage Area (Acres)	WQV ½ inch Runoff (cubic feet)	WQV Discharge Rate (cfs)
0.1	182	0.10	1.25	2269	1.10
0.2	363	0.20	1.5	2723	1.30
0.3	545	0.30	1.75	3176	1.60
0.4	726	0.40	2.0	3630	1.80
0.5	908	0.45	2.5	4538	2.20
0.6	1089	0.50	3.0	5445	2.70
0.7	1271	0.60	3.5	6353	3.10
0.8	1452	0.70	4.0	7260	3.60
0.9	1634	0.80	4.5	8168	4.00
1.0	1815	0.90	5.0	9075	4.40

Exhibit 3.5: Water Quality Volumes and Peak Discharges for Selected Acreages

The values used in EXHIBIT 3.5 were calculated using the **NRCS** CN procedure. The assumptions for the WQV peak discharges were a 5-minute Tc and a CN value of 98.

6.C Addressing Stormwater Run-On

Stormwater run-on is defined as any stormwater which intermingles with the Treatment Drainage Area runoff prior to treatment. This can occur as either overland flow or underground flow via culvert or storm sewer pipe. **NDOT** projects can receive stormwater run-on from both adjacent properties and other parts of the highway or right-of-way.

Stormwater run-on which is allowed to intermingle with the Treatment Drainage Area runoff must be included in the WQV to be treated. Intermingling can occur:

- At the source, such as when existing lanes drain across a new lane
- Along the conveyance path, such as when multiple inlets drain to the same pipe or ditch
- At the treatment point, such as when multiple outlets flow in to the same basin

The additive nature of stormwater run-on is necessary both as a regulatory compliance measure and to avoid over-flowing an under-sized STF system because higher than design volumes / flow rates are directed to it.

Stormwater run-on which can be diverted from intermingling with the Treatment Drainage Area runoff does not require treatment.

Stormwater run-on may be problematic for designers, for instance, considering a widening project where the Treatment Drainage Area drains into the roadside ditch. The stormwater then intermingles with a significant amount of stormwater run-on from a nearby development that is also draining into the **NDOT** ditch. Any number of small, low cost STFs may have been sufficient to treat the Treatment Drainage Area runoff. However, when the additional run-on from the development is factored in, a larger, more expensive STF may be required to treat the ½-inch water quality volume.

In the "Plan-In-Hand Phase:" (See the DPO, Ref. 3.1) the designer in consultation with their **Unit Head** may choose to separate stormwater run-on from the Treatment Drainage Area's stormwater runoff or allow it to intermingle at some point. When stormwater run-on is kept separate from the Treatment Drainage Area runoff, the designer can choose to treat only the Treatment Drainage Area run-off WQV. However, when the two are allowed to intermingle, the WQV of both the run-on and runoff shall be addressed. EXHIBIT 3.6 provides a flow chart showing when stormwater run-on must be treated. See EXHIBIT 3.7 for an example of dealing with stormwater run-on.

The designer documents on Form B the locations where stormwater run-on enters and exits the project and whether and how the stormwater run-on will be separated from project runoff, if discharging to a priority stormwater outfall. The **Unit Head** verifies that any off-site run-on has been accurately accounted for. Appropriate methods for dealing with the off-site run-on can be discussed during the Environmental Coordination Meeting.

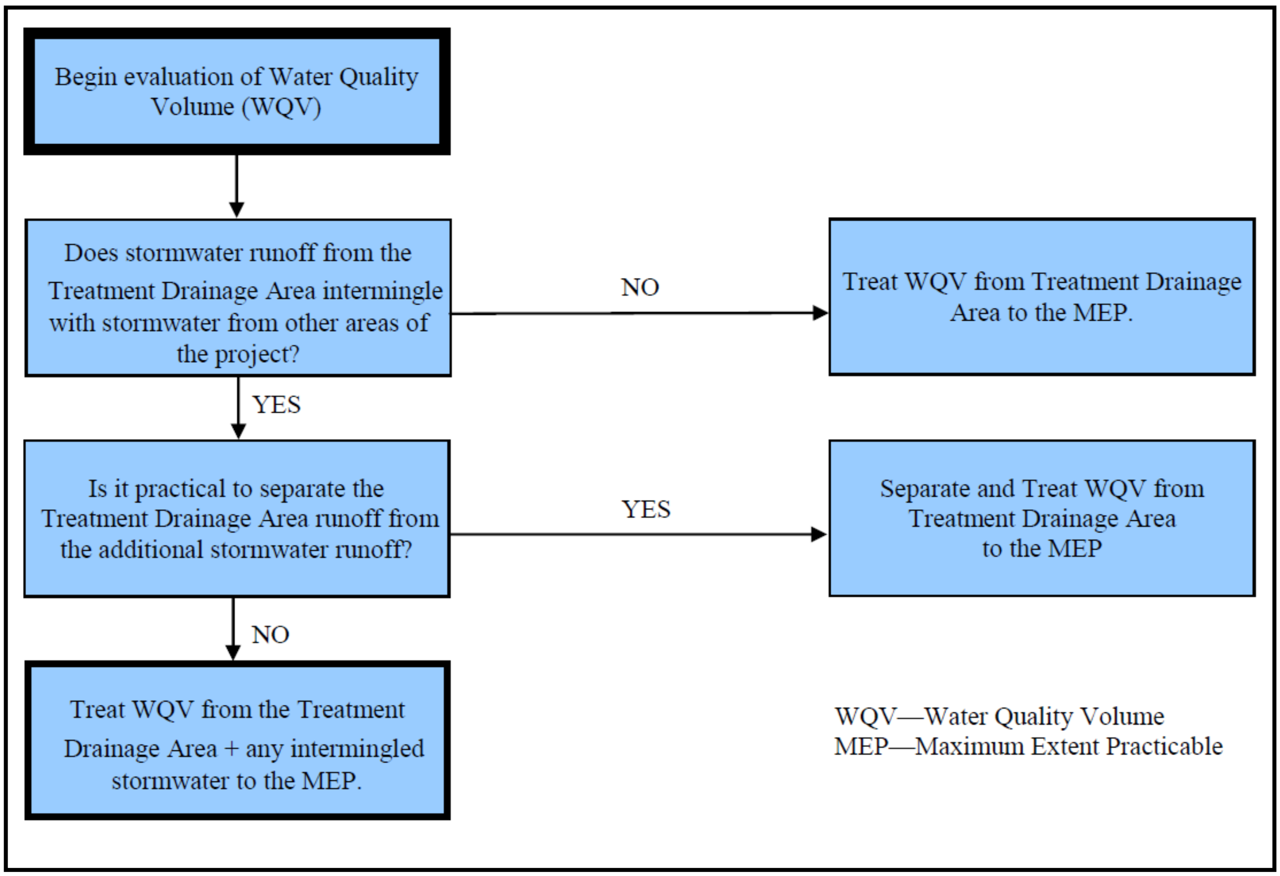


Exhibit 3.6: Stormwater Run-On Flow Chart

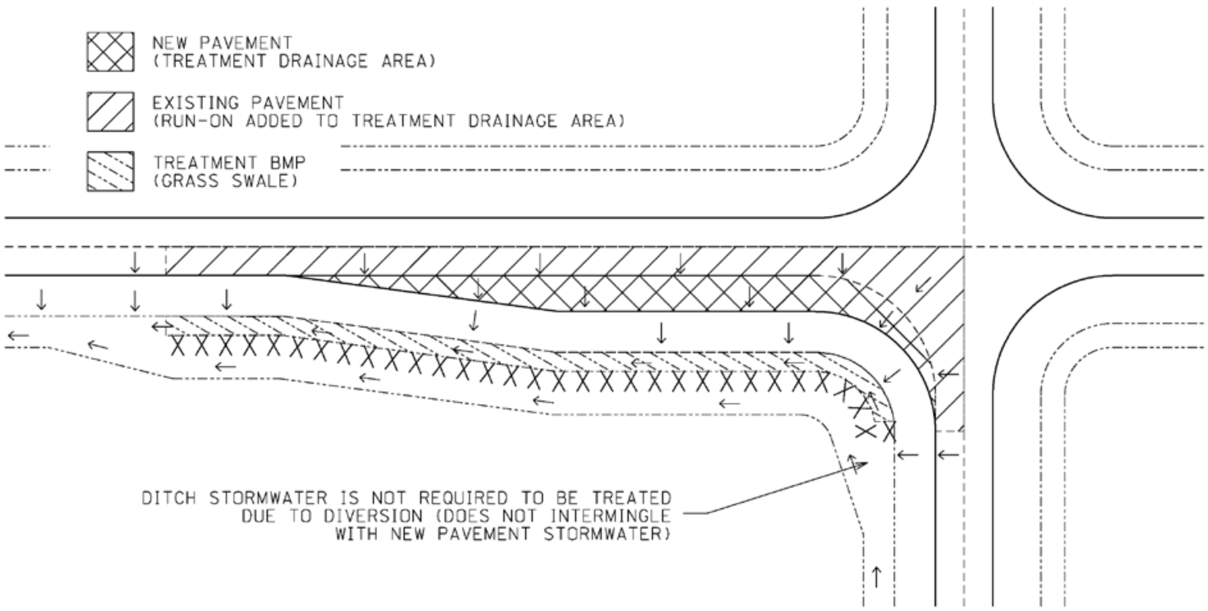


Exhibit 3.7: Example of Stormwater Run-On

It is desirable to avoid having to treat excessive run-on from impervious areas, particularly off site run-on. Designers should strive to treat stormwater prior to it intermingling with run-on from impervious areas on and off the project site. This may be accomplished by establishing a treatment point prior to the intermingling, or by physical separation of the flows as necessary to allow treatment prior to comingling. Designers will take care to maintain the existing drainage outfall pattern when considering separation of flows for stormwater treatment purposes.

If it is not feasible to treat run-on prior to intermingling, such as when adding an outside lane to existing roadway, then stormwater treatment facilities will be sized to account for all stormwater run-off from impervious areas. Providing treatment to the "maximum extent practicable" as indicated in EXHIBIT 3.6 means providing treatment for at least 80% of the Water Quality Volume. If this threshold cannot be achieved, other treatment options such as on-site mitigation (See Section 7.A.4.1) shall be pursued.

Treatment of run-on *from existing impervious areas within **NDOT** right-of-way* will be considered "excess" treatment and may be claimed as a "treatment credit." This credit may be used to offset requirements for new impervious pavement for which treatment cannot be accomplished. Designers will consider treating prior to intermingling before taking a "treatment credit" approach. Documentation will occur on Form B.

If it is not feasible to treat run-on from existing impervious areas within **NDOT** right-of-way or when inseparable stormwater run-on is received from impervious areas off State right-of-way, designers will consult and coordinate with **RDC** and/or the local municipality for other on- or off-site mitigation options.

If stormwater run-on to the project from *pervious* areas occurs, it will not increase the TDA.

Protection and maintenance of stormwater treatment facilities (STFs) will be included in agreements with the local community, who will be responsible for protecting existing STFs from untreated run-on as future community development occurs.

7. STF SELECTION AND PRELIMINARY DESIGN

Step 3 in the STF design process is to evaluate each Priority Stormwater Outfall identified in Section 5, select an appropriate STF or series of STFs, and complete a preliminary design of the STF. To complete this task the **NDOT** has identified several general considerations and established a STF Selection Chart to assist the designer with this process.

The designer should review the general considerations and select a STF for each of the priority stormwater outfall locations. The designer will record this selection on Form B.

7.A General Considerations

7.A.1 Online and Offline Treatment

STF s can be designed as either online or offline. An online STF is one where all stormwater runoff generated by the project is conveyed through the STF. By contrast, an offline STF is one where only a selected amount of stormwater runoff, frequently the WQV, is diverted through the STF, and all additional runoff bypasses the STF. See EXHIBIT 3.8 for a graphical representation.

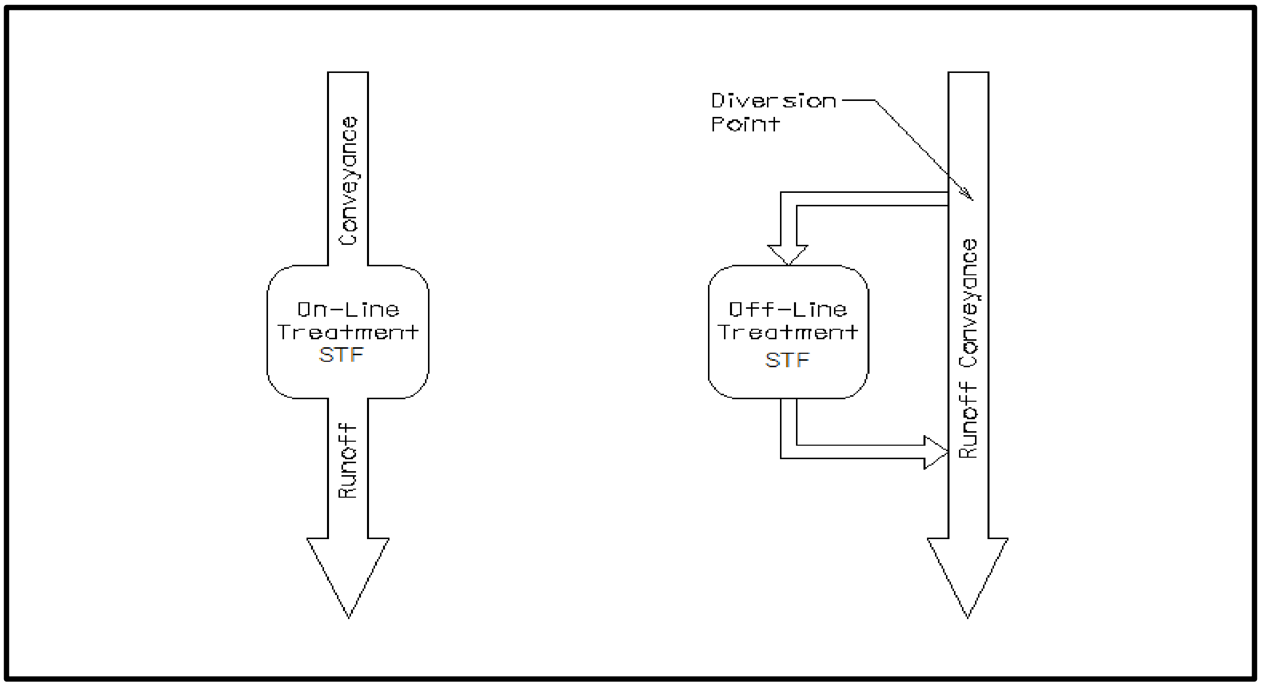


Exhibit 3.8 Schematic of Online and Offline STFs

An online STF is located within the stormwater runoff conveyance pathway. Flows up to the WQV are captured and treated by the STF, while the larger storm events are passed through the STF without treatment. The online STF needs to be able to convey anticipated stormwater flows without adversely impacting the STF, the adjacent highway, highway right-of-way, or adjacent property. Inlet and outlet structures need to be designed to both control the water quality volume and convey the highway design storm.

An offline STF is located outside and separate from the normal stormwater runoff conveyance pathway. Flows up to the WQV are diverted into the STF from the normal stormwater runoff conveyance pathway, where they are captured and treated. Large flow events are generally allowed to bypass or are actively diverted around the STF. In some cases, an offline STF may be designed to collect flows above the WQV, for secondary reasons, but are not required to treat those larger flows. An offline STF is only designed to handle the stormwater flows diverted to it and can be located adjacent to or a distance away from the stormwater runoff conveyance pathway.

The “STF’s Design Guides”, located in Appendix P, include recommendations on whether a STF should be designed as either offline or online.

7.A.2 Safety and Aesthetics

Safety is a consideration when designing STFs. To operate effectively some STF s need to impound water for a period of time. This may introduce the potential for ponding on or near roadways. It is the **NDOT’s** preference that STFs be located outside the lateral obstacle clearance zone. Refer to Chapter Six: The Typical Roadway Cross-Section, Section 3 of the Roadway Design Manual (Ref. 3.3) (http://dot.nebraska.gov/business-center/design-consultant/rd-manuals/), for guidance when placing STFs within the lateral obstacle clearance zone.

The following guidelines apply to the location and application of STFs:

- STFs should be designed so the roadway surface is not subjected to ponded water (see Chapter 1)
- Embankments such as dikes placed within the lateral obstacle clear distance should be constructed with 1:6 or flatter slopes
- Fencing may be needed around impoundments located within urbanized areas

Aesthetics may also be a factor when selecting and designing STF s since many of them require the establishment of vegetation to function properly. Efforts should be made to blend the STF into the surrounding landscape.

7.A.3 Coordination with Adjacent MS4

Coordination with federal, state, and local (city and county) governmental agencies is often necessary due to legal implications or special local drainage ordinances. Communication with adjacent MS4 permit holders and other municipalities (cities and counties) may be required whenever a proposed project results in stormwater discharges from the **NDOT’s** stormwater drainage systems to stormwater drainage systems owned and operated by the local MS4 or municipality, and vice versa.

Coordination with the adjacent MS4 Community may take on the form of an Agreement between the **NDOT** and the Community. When it is necessary to complete an Agreement, the DPO (Ref. 3.1) and **NDOT's** Operating Instruction 45-5 – "Agreements", included in Appendix R, must be followed. Language will be included in the municipal agreement assigning protection, operation, and maintenance of the constructed stormwater treatment facilities (STFs) to the local MS4 community.

7.A.4 Off-Site Stormwater Mitigation

In some cases it may not be practicable to provide stormwater treatment within the project limits due to various constraints such as site limitations (available right of way), costs, or other obstacles. If on-site mitigation is not feasible, off-site mitigation may be an option at other locations within the local watershed or MS4.

When all on-site mitigation options have been exhausted and documented, contact **RDC** to determine if off-site mitigation is feasible.

7.A.4.a On-Site Stormwater Mitigation

Run-off from other existing impervious areas within project limits that do not require treatment may be considered for treatment in the following conditions:

1) When a project is not in a TMDL or other area with defined water quality requirements, and
2) When it is not feasible to treat new impervious areas and/or run-on from any impervious areas, *within the* TDA.

This is considered *on-site mitigation*. Documentation should take place on Form B describing the Priority Outfall, the TDA being mitigated, and the location where on-site mitigation (treatment) will occur.

If a project is not in a TMDL or other area with defined water quality requirements and it is not practical to treat new impervious areas and any run-on, *within project limits*, then off-site mitigation may occur. If the project is in a TMDL or other area with defined water quality requirements, stormwater run-off must be treated at the location and mitigation is not an option.

On-site and off-site mitigation will require the concurrence from **RDC** <u>and</u> **Roadway Design Hydraulics Units** to verify that treatment is not practical within the TDA and/or within the project limits. If off-site mitigation is justified, **RDC** will note the area requiring treatment and account for that treatment in a separate stormwater treatment facility within the community's **NDOT** MS4 boundary. Documentation will occur on Form B.

7.A.5 Maintenance Responsibilities

The responsibility for maintenance of the STF s on a project varies over time and by location.

- **During Construction** – The contractor will be responsible for maintaining the completed STF s
- **During Vegetation Establishment Period** – After the contractor is released from the project, the **NDOT** will be responsible for maintaining the completed STF s until the stormwater permit obligations are complete
- **Post Vegetation Establishment Period** - The maintenance responsibilities for non-freeway highway appurtenances located within corporate limits reside with the municipalities, according to Nebraska State Statutes, Sections 39-1339, 39-1372 and 39-2105 (http://law.justia.com/codes/nebraska/2009/Chapter39/Chapter39.html),
  - **Interstates and Freeways** – The **NDOT** will continue to be responsible for maintaining the STFs
  - **Non-Freeways within Corporate Limits** – The municipality is responsible for continued maintenance of the STFs
  - **Non-Freeways outside Corporate Limits** – The **NDOT** will continue to be responsible for maintaining the STFs

The designer should be aware of the agency ultimately responsible for maintaining the STFs. The adjacent MS4 Community may have limitations and/or concerns about maintenance requirements of particular STFs.

The detailing of maintenance responsibilities may require an Agreement between the **NDOT** and the MS4 Community. When it is necessary to complete an Agreement, the DPO (Ref. 3.1) and **NDOT's** Operating Instruction 45-5 - Agreements must be followed.

7.A.6 Right-of-Way Considerations

Acquisition of property rights may be necessary to incorporate STFs in the roadside environment. This should be discussed in preliminary design to provide adequate time to determine if additional ROW will be available.

7.A.6.a Retention of ROW for STFs

STFs should be identified on the ROW plans to protect them against sale as excess land. Since many of these STFs will detain/retain water and may develop wetland conditions, it is necessary to permanently label and maintain these STFs as "Stormwater Treatment Facilities".

7.A.7 Compliance with Chapter One – Drainage Design

The designer and **Unit Head** are reminded that the STFs designed under Chapter Three must also comply with the policies and procedures of Chapter One – Drainage Design.

In Chapter Three, STFs are designed to minimize the potential discharge of pollutants in highway stormwater runoff to waters of the state during the WQV Rainfall Event (0.75-inch 24-hr storm). Chapter One is used to define and direct the peak discharges of stormwater runoff during the highway Design Storms (2-year to 100-year storm events).

STFs must be designed to the requirements of both Chapter Three and Chapter One of this Manual. The STF must be able to both collect / treat the WQV Rainfall Event and convey the peak discharges occurring during the highway Design Storm.

7.B STF Selection Process

7.B.1 Existing Conditions

The first step in the STF Selection Process is to determine the effectiveness of the existing site conditions in treating the WQV.

To complete this step the designer should evaluate the conditions between the Treatment Drainage Area and the Priority Stormwater Outfall to determine if there are existing features at the site (e.g. vegetated swales or detention basins, etc.) that may be utilized for water quality treatment. These existing features may not have been originally designed to treat stormwater, but as long as they meet the criteria, or can be enhanced to meet the criteria outlined in this Chapter, a designer may take credit for the water quality treatment they are providing.

Existing features evaluated and used as STFs are documented on Form B.

7.B.2 STF Selection Guidance

The second step in the STF Selection Process is to select and size the STF for the Treatment Drainage Area WQV.

A flowchart illustrating the STF selection process is shown in EXHIBIT 3.9. The designer can use this flowchart to select an appropriate STF at each priority outfall location. It may be necessary to combine STFs (e.g., a Detention Basin may be preceded by a grassed swale) to attain the treatment goal.

EXHIBIT 3.10 provides a tabulation of the available STFs and their suitability to various locations and conditions.

The selection of STFs is documented on Form B. The designer utilizes the corresponding STF Design Guide given in Appendix P to complete preliminary design.

When the designer has completed a preliminary STF design for the project, the designer's **Unit Head** will review and sign off on Form B. The **Unit Head** will then forward a copy of the Form B to **RDC** for review. Environmental Coordination Meeting 30 can be used to discuss the preliminary STF designs.

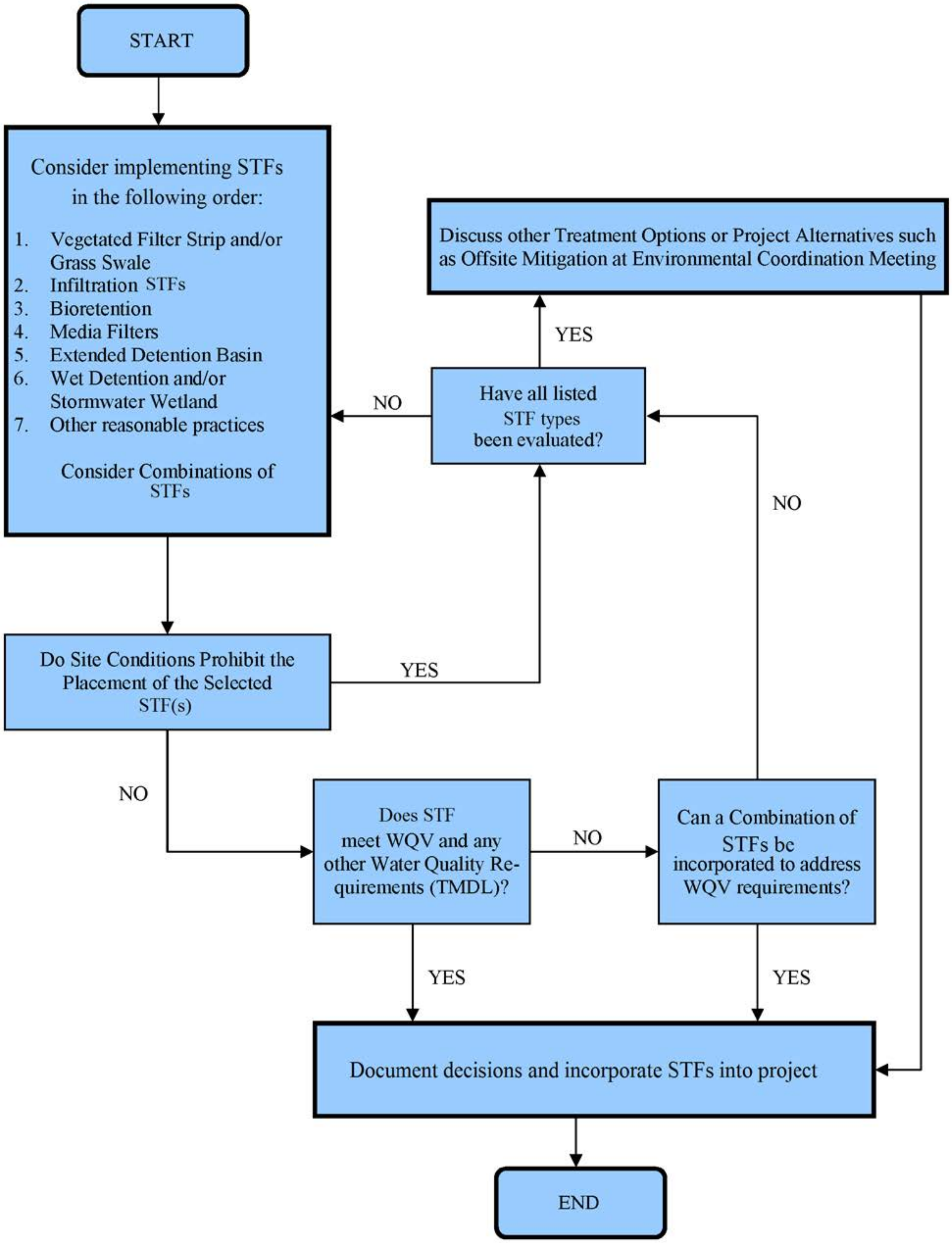


Exhibit 3.9 STF Selection Chart

7.B.2.a Order of STF(s)

The order of STFs listed in the selection chart is a suggested ranking which shows the preference of the **NDOT** for treating stormwater discharges. The suggested order moves from less intrusive methods, both from a visual and maintenance perspective, to methods that require more area, are more costly to build, and require more maintenance. Some of the least intrusive designs may provide the best means to obtain the **NDOT's** water quality objectives. The designer, in consultation with their **Unit Head**, may choose to implement any of the STFs without going through the listed order.

See Section 7.B.2.d for a discussion on Combining STFs.

7.B.2.b Site Conditions

Site-specific conditions can affect operations, maintenance, construction costs, safety and aesthetics of STFs. When designing a STF, the designer needs to give proper consideration to recovery zones, setbacks, hydraulic head, and maintenance access roads and ramps.

Each STF has minimum design parameters that need to be met in order to treat the stormwater discharge effectively. For example, grass swales require ditch slopes of 4% or flatter, extended detention basins require both a specific amount of volume and a minimum width to length ratio, and infiltration basins require adequate space and a relatively high native soil infiltration rate. The site conditions where a STF will be located often determine whether or not the minimum design parameters are achievable for a particular STF.

When a desired STF cannot be placed due to a site limitation, the designer should select a different STF and evaluate it for the site conditions. The designer may choose to place multiple STFs that each treat a limited amount of the WQV (see next section), but when added together treat the entire WQV. See Section 7.B.2.d for additional discussion on combining STFs.

7.B.2.c Design STF for WQV

The "STF Design Guides" provide the specifics for designing each of the approved STFs. The designer uses these guides and the calculated WQV goal as discussed in Section 6 to establish the design for the selected STF.

Contact the **Roadway Design Hydraulics Engineer** and **RDC** for assistance with designing STFs for water quality requirements other than the 0.5-inch WQV goal (example: TMDL requirements).

When site conditions allow the placement of a STF, but do not allow for it to treat the entire WQV goal, the designer in consultation with their **Unit Head** may either choose to evaluate another STF or add multiple STFs in series to attain the WQV goal (see Section 7.B.2.d Combining STFs).

7.B.2.d Combining STFs

When the calculated WQV or other water quality goal exceeds the capacity of a single STF, the designer may be able to combine several of the same or multiple STFs in sequence to achieve the desired WQV or goal.

Where STFs are used in combination, the designer shall first determine the parameters of each of the STFs that can be placed, and then using the "STF Design Guides" calculate the amount of WQV each STF can treat. The sum of the WQV from each STF in the sequence should total 100% of the Treatment Drainage Area WQV goal.

7.B.2.e Other Treatment Options or Project Alternative

There may be occasions when the site conditions are such that the WQV goal cannot be met utilizing the process provided and the STFs approved by the **NDOT**. The designer should bring this issue to the project's next Environmental Coordination Meeting (See **EXHIBIT A** of the DPO, Ref. 3.1) where additional options can be discussed. The designer is also encouraged to consult with **RDC** and the **Roadway Design Hydraulics Section** for assistance.

7.C STF Summary

The following is a brief description of all approved STFs. Detailed design guides for each STF are available in Appendix P.

7.C.1 Vegetated Filter Strips

Vegetated Filter Strips are zones of vegetation through which stormwater runoff is directed. Vegetated filter strips intercept and convey sheet runoff from an impervious area (i.e. highway) and treat the runoff by filtration through vegetation, sediment deposition, and infiltration and adsorption by soil. They are effective if the velocity of sheet flow is slow, providing an opportunity for sediments and other pollutants to settle and be filtered. Vegetated filter strips can be used to enhance the water quality of stormwater runoff on small sites, or as pre-treatment for another structural stormwater STF. Maintaining sheet flow and preventing concentrated flows are important components in the design of vegetated filter strips.

Examples:

- Roadside vegetated foreslope
- Flattened backslope receiving runoff from an adjacent parking area or development

Design Notes:

- Required dense vegetative growth
- Appropriate for small drainages (<1 acres)
- Water must enter as sheet flow across the entire filter strip (Level Spreader)
- Minimum slope length approx. 20 ft
- Slope range between 2-17%
- Commonly used as pretreatment for other STFs
- Low maintenance and low cost

7.C.2 Grass Swales

Grass swales are open, shallow channels with vegetation covering the side slopes and bottom that collect and slowly convey runoff flow to downstream discharge points. A grass swale is designed to meet nominal treatment of runoff for the WQV and can be an important component in a combined STF system. In general, grass swales are very similar to roadside ditches with mild slopes and dense vegetation and are therefore well suited for treating runoff from roads, highways and impervious surfaces. When properly incorporated into a stormwater treatment design, grass swales help to reduce impervious cover, accent the natural landscape and provide aesthetic benefits.

Examples:

- Roadside ditch.

Design Notes:

- Ditch Slopes 0.5-6%, Side slopes no greater than 1:3
- Good addition to a combined STF system
- Relatively low maintenance STF

7.C.3 Infiltration Trench

The infiltration trench is an excavated trench 3 to 8 ft. deep, backfilled with rock or stone aggregate and lined with filter fabric. This practice temporarily stores small water quality volumes and allows it infiltrate into the soil over a prescribed period. The WQV passes through a combination of pretreatment measures (vegetated filter strip, vegetated swale, sediment forebay, etc.) and into the infiltration trench, creating an underground reservoir for the runoff. The runoff gradually infiltrates through the bottom and sides of the trench and eventually reaches the underground water table.

Examples:

- Series of trenches cut across the roadside ditch or median at defined intervals
- At the outlet of a flume or small storm sewer system (one to three inlets)
- Along the edge of a parking area or development

Design Notes:

- Recommended in soils with a minimum infiltration rate of 0.5 in/h
- Underdrains can be incorporated in soils with low permeability
- Depth to water table must be considered
- Maintenance is variable
- Most effective for smaller drainages (<5 acres)

7.C.4 Infiltration Basin

An infiltration basin is an excavated impoundment with no primary outlet that captures and temporarily stores larger water quality volumes until it can infiltrate into the soil over a prescribed period. The basin has a flat floor with an optional underdrain system to allow draining in the event of standing water. A secondary outfall may be provided to pass higher volumes of flow.

Examples:

- Basin at the end of larger storm sewer system or extended length of ditch
- Widened and flattened section of ditch with amended soils
- Capture runoff from edge of facilities (rest areas, maintenance yards, etc)

Design Notes:

- Pre-treatment is recommended to avoid premature clogging of the system
- Typically serves drainage areas from 5-10 acres (20 acres if offline)
- Recommended in soils with a minimum infiltration rate of 0.5 in/h.
- Secondary or emergency outlet should be incorporated
- Depth to water table must be considered
- Regular maintenance is required

7.C.5 Bioretention

The bioretention STF is a shallow stormwater basin or landscaped area that utilizes engineered soils and vegetation to capture and treat runoff. Stormwater runoff is temporarily ponded within the basin and seeps through the engineered soils. The biomass in the system retains nutrients and other pollutants, and the stormwater is filtered through the surface vegetation, mulch layer, and pervious soil layer. The filtered stormwater is then either allowed to infiltrate in to the surrounding soils, or more commonly, discharged through an aggregate base layer and perforated pipe sub-drain.

Examples:

- Curb cut into rain garden
- A series of offline plantings adjacent to the roadside ditch
- Sections of or the entire length of a median
- At the outlet of a flume or small storm sewer system (one to three inlets)
- Along the edge of a parking area or development

Design Notes:

- Pre-treatment is optional but recommended
- Depth to water table must be considered
- Incorporate an underdrain system
- Regular maintenance is required

7.C.6 Media Filter

A media filter is a structural stormwater STF system that temporarily stores stormwater runoff and passes it through a filter bed of sand or other filtering media. The system usually consists of two chambers. The first is the sediment forebay or sedimentation chamber, which collects and allows heavier sediment to settle. The second is the filtration chamber, which filters the runoff through a bed of sand or other filtering media. An underdrain is used to return the filtered runoff to the conveyance system or other receiving waters.

Examples:

- An underground vault associated with a storm sewer system in ultra-urban location
- An open air vault or basin located at the end of a culvert/storm sewer system
- A ditch check with a sand filled drain

Design Notes:

- Pre-treatment is recommended
- Permeability of filtering media controls design
- Regular maintenance required
- Typically serves drainage areas up to 2 acres (5 acres if offline)

7.C.7 Extended Dry Detention

The extended dry detention facility is designed to temporarily store the WQV, and slowly release it to receiving waters. This allows the suspended solids in the stormwater enough time to settle out of suspension. Stormwater flows are collected in the shallow basin and constricted by an outlet structure to produce low flow rates calculated to delay the drawdown of the basin over a 24 to 48 hour (72 hour maximum) period. Extended detention basins are typically end-of-system STFs designed to limit the impact of a site's stormwater discharge on downstream property and receiving waters.

Examples:

- Basin at the end of larger storm sewer system or extended length of ditch
- At the outlet of a flume or small storm sewer system
- Along the edge of a parking area or development

Design Notes:

- Pre-treatment forebay may improve performance and lower maintenance frequency
- Outlet control structures determine the extent of detention/retention and treatment
- Appropriate basin size and outlet design are critical to the effectiveness of these systems
- Typically serves larger drainage areas (>5 acres)
- Regular maintenance is required

7.C.8 Wet Detention Pond

A wet detention pond is designed to intercept stormwater runoff, temporarily impound that runoff and release it at a reduced rate to the receiving stream or stormwater system. Wet detention ponds use a permanent pool of water to aid in achieving water quality control. The pool may cover the entire pond bottom or may be located in only a portion of the pond. A wet detention pond can be used to reduce the post-development discharge to that of the pre-development rate under the 2-year, 10-year and 100-year event storms and can provide relatively good water quality improvement along with rate control. Stormwater runoff entering the pond intermixes with the standing pool reducing the sediment concentration and diluting the dissolved constituents. Effectiveness of the Wet Detention Pond is dependent on the size of the standing pond relative to the water quality volume.

Examples:

- Pond at the end of larger storm sewer system
- At the ends of ditches with larger drainage areas
- Pond located at the end of midsize culverts
- Located on small flowing drainages

Design Notes:

- Design outlet control structures for both WQV and larger detention and determine the extent of detention/retention and treatment.
- Basin size and outlet design are critical to the effectiveness of these systems
- Extended wet detention ponds typically detain water for 24-48 hours to provide settling of particulates.
- Requires larger drainage areas (10 acres minimum)
- Pre-treatment forebay may improve performance and lower maintenance frequency

7.C.9 Stormwater Wetland

A stormwater wetland is a constructed wetland planted with emergent vegetation to treat stormwater. Stormwater wetlands can also function in reducing the peak discharge of storm events by providing temporary water storage above the normal pool elevation. Stormwater treatment is achieved by settling of particulates and biological uptake by the wetland vegetation. Stormwater wetlands are among the most effective STFs for pollutant removal while providing aesthetic and wildlife benefits with relatively low maintenance costs.

Examples:

- Basin at the end of larger storm sewer system
- At the ends of ditches with larger drainage areas
- Pond located at the end of midsize culverts
- Located on low flow drainages

Design Notes:

- Relatively large contributing drainage area is required to maintain an adequate water source (10 acres minimum)
- Forebay should be incorporated to decrease velocity and reduce sediment loading
- Wetland should be designed with varied depths to support a diverse range of vegetation
- High maintenance requirements during the first couple years while vegetation is establishing

7.C.10 Pervious Pavement

Pervious pavement systems allow the infiltration of stormwater runoff through a pavement surface into an aggregate base. The aggregate base provides temporary storage of captured rainfall where it then infiltrates into underlying soils or is collected by an underdrain. It also provides the structural and functional features needed for the roadway, parking lot or sidewalk. The paving surface, subgrade and installation requirements of porous pavements are more complex than those for conventional asphalt or concrete surfaces.

Examples:

- Potential applications in low volume traffic areas such as rest areas, maintenance yards, weigh stations, etc.
- Other uses include emergency stopping areas, traffic islands, sidewalks, road shoulders, vehicle cross-overs on divided highways, and low-traffic roads

Design Notes:

- Generally not suited for areas with high traffic volumes or loads
- Requires underdrain system
- Regular maintenance is necessary to reduce the potential for clogging

7.C.11 Proprietary Structural Treatment Control

Proprietary Structural Treatment Control are commercially available, often prefabricated units, designed and sized for stormwater treatment by the manufacturer based on criteria provided. These systems treat stormwater in various ways ranging from mechanisms to enhance particle settling to oil and water separation. Some devices use settling and surface oil separation mechanisms, whereas others use filtration or vortex motion separating mechanisms. Structural treatments may be placed online or offline depending on manufactures specifications.

Examples:

- Catch basin insert
- Oil/grit separators built into the inlet structure
- Hydrodynamic separator

Design Notes:

- Used as pre-treatment in storm sewer systems
- Particularly suitable for ultra-urban environment
- Size dependent upon the amount of stormwater needing treatment
- High maintenance STF

7.C.12 Other Reasonable Practices

Stormwater management is an evolving field and STFs will need to be updated as technologies advance and research provides additional information. If a designer would like to incorporate a treatment device not described in this chapter, please contact **RDC** for approval.

NDOT STF Suitability Matrix										
STF	**Description**	**Treatment Type**	**Typical Drainage Area**	**Site Suitability**	**Soil Permeability**	**Groundwater Limitations**	**Best at Removing....**	**Construction Cost**	**Maintenance Cost**	**Comments**
Vegetated Filter Strips	Densely vegetated strip of land designed to treat sheet flow	Water Quality Treatment Rate	Based on Sheet Flow Loading	Rural and Urban Section	Any (with soil conditioning)	≥ 2 feet	Suspended Solids	Low	Low	Maximum slope generally 6H:1V - longer filter strip needed on steeper slopes
Grass Swale	Densely vegetated drainage way designed to convey runoff slowly to allow for treatment	Water Quality Treatment Rate	≤ 5 acres	Rural and Urban Section	Any (with soil conditioning)	≥ 2 feet	Suspended Solids, Heavy Metals, Hydrocarbons	Low	Low	Limited depth of flow and velocity for effective treatment
Infiltration Trench	Aggregate-filled trench designed to capture runoff in the void space and infiltrate it	Water Quality Volume	≤ 5 acres	Rural and Urban Section	0.5 - 12 in/hr	≥ 4 feet	Suspended Solids, Nutrients, Heavy Metals	Moderate-High	Moderate	Width of Trench ≥ Depth
Infiltration Basin	Shallow basin designed to capture runoff above ground and infiltrate it through natural soils	Water Quality Volume	10-20 acres	Rural and Urban Section	0.5 - 12 in/hr	≥ 4 feet	Suspended Solids, Nutrients, Heavy Metals, Hydrocarbons	Moderate	Moderate	Drainage area limited if constructed online
Bioretention Basin	Shallow basin designed to capture runoff above ground and infiltrate it through an amended soil zone with underdrain	Water Quality Volume	≤ 5 acres	Rural and Urban Section	0 - 12 in/hr	≥ 4 feet	Suspended Solids, Heavy Metals	Moderate	Moderate	Landscape plantings typical with this BMP
Media Filter	Structure that includes a sedimentation chamber and sand filtration chamber to treat runoff	Water Quality Volume	≤ 2 acres	Urban and Ultra-Urban	Any	n/a	Suspended Solids	High	High	Typically a cast-in-place structure - check hydraulic grade lines and possibly buoyancy issues
Extended Dry Detention	Dry basin designed to capture runoff with drawdown to the basin bottom over an extended period	Water Quality Volume	≤ 5 acres	Rural and Urban Section	0 - 12 in/hr	0	n/a (see comments)	Low	Low	Only moderate treatment efficiency in general
Wet Detention	Basin with permanent pool of water designed to capture runoff with drawdown to the normal pool elevation over an extended period	Water Quality Volume	10 acres + (typ)	Rural and Urban Section	Depends on Design	Depends on Design	Suspended Solids	Low	Low	Minimum drainage area needed to maintain permanent pool
Stormwater Wetland	Basin or drainage way with a pool of water of varying depths designed to support wetland vegetation and treat water flowing through the system	Water Quality Volume	10 acres + (typ)	Rural and Urban Section	Depends on Design	Depends on Design	Suspended Solids, Heavy Metals	Moderate	Moderate	Minimum drainage area needed to maintain wetland
Pervious Pavement	Various types of pavement with the ability to pass stormwater through the surface to an underlying aggregate bed - the aggregate bed is designed to capture runoff in the void space and release it slowly through an underdrain	Water Quality Volume	≤ 5 acres	Urban and Ultra-Urban	0 - 12 in/hr	≥ 4 feet	Suspended Solids, Heavy Metals, Hydrocarbons	High	High	Protect adjacent pavement
Proprietary Structural Treatment Controls	Various types of proprietary devices designed to treat stormwater runoff	Water Quality Treatment Rate (Typical)	≤ 2 acres	Urban and Ultra-Urban	Any	n/a	Varies	High	High	Approval of device needed - check hydraulic grade lines and possibly buoyancy issues

Exhibit 3.10 STF Suitability Matrix

8. COMPLETING STF DESIGN

During the “Final Design Phase” of the project (See the DPO, Ref. 3.1), the designer will complete the STF design. This will include finalizing the STF size, designing the discharge structure, and applying build notes, etc. The designer also addresses landscaping, construction, maintenance, and a number of miscellaneous items related to the STFs.

At the end of the public hearing phase the designer reviews and revises Form B. The **Unit Head** will then sign off on the completed form and forwards a copy to **RDC**. **RDC** will review the form along with final plans for the STF(s) during the erosion control plan review. The final stormwater treatment plans will be presented and discussed during Environmental Coordination Meeting 40.

8.A Landscaping

The final landscape plan will be completed by **RDC** and provided to the designer for inclusion with project plans. The landscape plan will include the planting scheme necessary to provide both functional and aesthetic value to the stormwater STFs.

8.B Construction Phasing

STFs are intended to be operational when a project is completely stabilized. This eliminates the potential for early failure due to sedimentation during the vegetation establishment period of the site. The following three options are available to a designer for phasing construction of STFs:

- Option 1 – STF is constructed as the project is built and is isolated from stormwater flows until the Treatment Drainage Area is completely stabilized
- Option 2 –STF is partially constructed with the project and utilized as a temporary sediment control measure until the Treatment Drainage Area is completely stabilized
- Option 3 –STF is constructed after its Treatment Drainage Area is completely stabilized

The designer documents on Form B, which option(s) is appropriate for each selected STF. Phasing will also need to be documented on the construction plans or by special provision. Phasing decisions are made in consultation with the **District**.

8.C Maintenance Schedule

Maintenance schedules for each STF are provided in the Design Guides provided in Appendix P. If a designer has a maintenance requirement that differs from those provided in the STF Design Guide, he/she needs to document them on Form B. **RDC** will utilize that information and the schedules in the Design Guides to complete “Stormwater Treatment within MS4 Communities / Form C – Maintenance” (Form C), included in Appendix Q. **RDC** will forward Form C to the **Operations Division**.

8.D Plan Labeling of STF

The designer must coordinate the labeling of the STFs on the design plans (See Section 7.A.6.a). Label STFs on both design plans and ROW plans as a “Stormwater Treatment Facility” followed by the specific STF title. STF titles are given in the STF Design Guidelines in Appendix P.

For example: a sand filter used as a STF will be labeled as “Stormwater Treatment Facility – Sand Filter”.

8.E Miscellaneous

8.E.1 Fencing

The majority of STFs will be located along highways within urbanized areas. With this in mind the designer may choose to include fencing around the STF to limit access by the general public. This may be of particular importance on STFs which pond water more than a foot deep. The **Unit Head** will review the use of fencing and request approval from the **District**.

8.E.2 Signage

In addition to stormwater treatment, the MS4 permit requires the **NDOT** to implement a stormwater education program. The use of STFs may provide the **NDOT** with an outreach opportunity to showcase environmental commitments. This could be accomplished by placing a sign adjacent to a STF that explains how the measure functions to provide water quality benefits. **RDC** will coordinate with the designer and the **Traffic Engineering Division** to determine if sign(s) are feasible and what information should be included.

9. REFERENCES

3.1 Nebraska Department of Transportation, Design Process Outline (DPO), Current Edition. (http://dot.nebraska.gov/media/6761/design-process-outline.pdf)

3.2 United States Environmental Protection Agency, National Management Measures to Control Nonpoint Source Pollution from Urban Areas, Washington, D.C., November 2005

3.3 Nebraska Department of Transportation, Roadway Design Manual, Current Edition (http://dot.nebraska.gov/business-center/design-consultant/rd-manuals/)

NDOT required metric conversions to be in accordance with the International System of Units (SI) as published by ASTM 380, unless modified by this document.

Hydraulics

The following list provides relevant physical constants and expressions for drainage applications.

Manning's equation (SI): $V = 1/n\ (R^{2/3}\ S_f^{\ 1/2})$

where:
- V = Velocity, m/s;
- R = Hydraulic radius, m;
- S_f = Longitudinal friction slope, m/m;
- n = Manning's roughness coefficient, dimensionless.

Rational formula (SI): $Q = KCiA$

where:
- Q = Flow, m^3/s;
- i = Rainfall intensity, mm/h;
- A = Drainage area, hectares;
- K = Coefficient, 1/360;
- C = Runoff coefficient, dimensionless.

Acceleration due to gravity (SI): $g = 9.81\ m/s^2$

Pipe/Conduit

Steel pipe and copper tube sizes will not change by switching to the metric system. American sizes are used in many parts of the world. Initially, they are simply classified by the nominal mm size, but in the future, hard metric pipe metric pipe sizes will probably be utilized.

ASTM B88M, which gives standardized hard metric copper tube sizes, will not be utilized until ample product availability can be established. The ASHRAE SI Guide gives the nominal ISO size for American pipe, but the **NDOT** has opted to use a hard conversion of 1 in. = 25 mm. Schedule designations remain the same (example: Schedule 40, and Type K, L, M).

Concrete pipe diameters will be expressed in hard converted sizes. These sizes are found in ASTM C14M for non-reinforced concrete pipe. The English-sized pipes fit into the tolerances of the metric sizes.

The following table lists the inch-pound names for pipe products (called NPS or "nominal pipe size") and their metric equivalents (called DN or "diameter nominal"). The metric names conform to ISO usage and apply to all piping used in buildings and civil works projects.

DN (mm)	**NPS (in.)**	**DN (mm)**	**NPS (in.)**
3	1/8	375	15
6	¼	450	18
9	3/8	525	21
13	½	600	24
16	5/8	750	30
19	¾	900	36
25	1	1050	42
31	1¼	1200	48
38	1½	1350	54
50	2	1500	60
63	2½	1650	66
75	3	1800	72
88	3½	1950	78
100	4	2100	84
113	4½	2250	90
125	5	2400	96
150	6	2550	102
200	8	2700	108
250	10	2850	114
300	12	3000	120

Note: For pipe sizes over 120 in., use 1 in. = 25 mm.

Nominal Pipe Sizes in Inches and Millimeters

Pipe Thickness			
AASHTO SI (mm)	English Value (in.)	Nominal Rounded Up (mm)	Rounded Down (mm)
1.02	0.040	1.1 (0.04331)	1.0 (0.03937)
1.32	0.052	1.4 (0.05512)	1.3 (0.05118)
1.63	0.064	1.65 (0.06496)	1.6 (0.06299)
2.01	0.079	2.1 (0.08268)	2.0 (0.07874)
2.77	0.109	2.8 (0.11024)	2.7 (0.10630)
3.51	0.138	3.6 (0.14173)	3.5 (0.13780)
4.27	0.168	4.3 (0.16929)	4.2 (0.16535)

Pipe Thickness			
AASHTO SI (mm)	English Value (in.)	Nominal Rounded Up (mm)	Rounded Down (mm)
0.91	0.036	1.0 (0.03931)	0.9 (0.03543)
1.17	0.046	1.2 (0.04724)	1.1 (0.04331)
1.45	0.057	1.5 (0.05906)	1.4 (0.05512)
1.83	0.072	1.9 (0.07480)	1.8 (0.07087)
2.57	0.101	2.6 (0.10236)	2.5 (0.09843)
3.28	0.129	3.3 (0.12992)	3.2 (0.12598)
4.04	0.159	4.1 (0.16142)	4.0 (0.15748)

Pipe Corrugation Size		
AASHTO SI (mm)		English Equivalent (in.)
68 x 13	(2.68 in. x 0.512 in.)	$2^2/_3$ x ½
76 x 25	(2.99 in. x 0.984 in.)	3 x 1
125 x 25	(4.921 in. x 0.984 in.)	5 x 1
19 x 19 x 190	(0.748 in. x 0.748 in. x 7.48 in.)	¾ x ¾ x 7½
19 x 25 x 292	(0.748 in. x 0.984 in. x 11.496 in.)	¾ x 1 x 11½

The length of pipes can be varied as needed and may depend on limitations of haulers, handlers, designers and installers. The standard length for corrugated steel pipe as proposed by NCSPA is 6 meters. All pipe design lengths for **NDOT** should preferably be to the nearest meter, (to the nearest 0.5 meter is acceptable).

Pipe Lengths		
Metric Value (m)	English Equivalent (ft.)	Shown As (m)
2.4384	8	2.5 (8.2 ft.)
3.0480	10	3.0 (9.8 ft.)
4.8768	16	5.0 (16.4 ft.)
6.0960	20	6.0 (19.7 ft.)
7.3152	24	7.5 (24.6 ft.)
9.1440	30	9.0 (29.5 ft.)

CONVERSION FACTORS				
FROM		**MULTIPLY BY**	**TO OBTAIN**	
Unit	Abbreviation		Unit	Abbreviation
cubic foot per second	cfs	0.02832	**cubic meter per second**	m^3/s
foot	ft.	0.3048	**meter**	m
square foot	sq. ft.	0.0929	**square meter**	m^2
cubic foot	cu. ft.	0.0283	**cubic meter**	m^3
inch	in	25.4	**millimeter**	mm
square mile	sq. mi.	2.59	**square kilometer**	km^2
acre	Ac	0.4047	**hectare**	Hec
foot per second	fps	0.3048	**meter per second**	m/s
gallon	gal	3.7854	**liter**	L

Metric Conversion Factors

UNIT ACCURACY		
UNIT	**APPLIES TO**	**ACCURACY**
cubic meter per second	All items with a unit of cubic meter per second	Nearest m^3/s
meter	All items with a unit of meter except: Culvert Pipe	Nearest 0.1 m Nearest 0.5 m
square meter	All items with a unit of square meter	Nearest m^2
cubic meter	All items with a unit of cubic meter	Nearest m^3
millimeter	All items with a unit of millimeter	Nearest mm
square kilometer	All items with a unit of square kilometer	0.1 km^2
hectare	All items with a unit of hectare	Nearest 0.5 Hec
meter per second	All items with a unit of meter per second	Nearest m/s
liter	All items with a unit of liter	Nearest 10 L
Kiloliter	All items with a unit of 1000 liters	Nearest KL

Metric Unit Accuracy

Manning Equation Coefficients of Roughness

	Manning's n range
I. Closed Conduits:	
A. Concrete Pipe	0.011 - 0.013
B. Corrugated-metal pipe or pipe arch:	
1. 2-2/3 by 1/2 in corrugation (riveted pipe):	
a. Plain or fully coated	0.024
b. Paved invert (ranged values are for 25% and 50% of circumference paved):	
(1) Flow full depth	0.021 - 0.018
(2) Flow 08 depth	0.021 - 0.016
(3) Flow 06 depth	0.019 - 0.013
2. 6 by 2-in corrugation (field bolted)	0.030
C. Cast-iron pipe, uncoated	0.013
D. Steel pipe	0.009 - 0.011
E. Monolithic concrete:	
1. Wood forms, rough	0.015 - 0.017
2. Wood forms, smooth	0.012 - 0.014
3. Steel forms	0.012 - 0.025
F. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017 - 0.022
2. Natural floor	0.019 - 0.025
II. Open Channels, Lined (straight alignment):	
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. Asphalt

1. Smooth 0.013
2. Rough 0.016

E. Concrete-lined excavated rock:

1. Good section.. 0.017 - 0.020
2. Irregular section .... 0.022 - 0.027

III. Open Channels, Excavated (straight alignment, 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.. 0.080 - 0.120
2. Clean bottom, brush on sides.. 0.050 - 0.080
3. Clean bottom, brush on sides, highest stage of flow.. 0.070 - 0.110
4. Dense brush, high stage.. 0.100 - 0.140

IV. Channels & Swales w/Maintained Vegetation (Values shown are for velocities of 2 & 6 fps):

A. Depth of flow up to 0.7 foot:

1. Bermudagrass, Kentucky bluegrass, buffalograss
   a. Mowed to 2 inches .... 0.045 - 0.070
   b. Length 4-6 inches .... 0.050 - 0.090
2. Good stand, any grass:
   a. Length about 12 inches .... 0.090 - 0.180
   b. Length about 24 inches .... 0.150 - 0.300
3. Fair stand, any grass:
   a. Length about 12 inches .... 0.0800 - 0.140
   b. Length about 24 inches .... 0.1300 - 0.250

B. Depth of flow 0.7 - 1.5 feet:

1. Bermudagrass, Kentucky bluegrass, buffalograss
   a. Mowed to 2 inches .... 0.030 - 0.050
   b. Length 4-6 inches .... 0.040 - 0.060
2. Good stand, any grass:
   a. Length about 12 inches .... 0.070 - 0.120
   b. Length about 24 inches .... 0.100 - 0.200
3. Fair stand, any grass:
   a. Length about 12 inches .... 0.060 - 0.100
   b. Length about 24 inches .... 0.090 - 0.170

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 the above values of x by 0.002

VI. Natural Stream Channels:

A. Minor streams (surface width at flood stage less than 100 feet):

1. Fairly regular section:

a. Some grass & weeds, little or no brush .... 0.030 - 0.035

b. Dense growth of weeks, depth of flow materially greater than weed height .... 0.035 - 0.050

c. Some weeks, light brush on banks .... 0.035 - 0.050

d. Some weeks, heavy brush on banks .... 0.050 - 0.070

e. Some weeks, dense willows on banks....... 0.060 - 0.080

f. For trees within channel with branches submerged at high stage, increase all above values by .... 0.010 - 0.020

2. Irregular sections, with pools, slight channel meander; increase values given in 1 a-e about....... 0.010 - 0.020

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 .... 0.040 - 0.050

b. Bottom of cobbles, with large boulders .... 0.050 - 0.070

B. Flood plains (adjacent to natural streams):

1. Pasture, no brush:

a. Short grass .... 0.030 - 0.035

b. High grass .... 0.035 - 0.050

2. Cultivated areas:

a. No crop .... 0.030 - 0.040

b. Mature row crops .... 0.035 - 0.045

c. Mature field crops .... 0.040 - 0.050

3. Heavy weeds, scattered brush .... 0.050 - 0.070

4. Light brush and trees:

a. Winter .... 0.050 - 0.060

b. Summer .... 0.060 - 0.080

5. Medium to dense brush:

a. Winter .... 0.070 - 0.110

b. Summer .... 0.100 - 0.160

6. Dense willows, summer, not bent over by current....... 0.150 - 0.200

7. Cleared land w/ tree stumps, 100-150 per acre:

a. No sprouts .... 0.040 - 0.050

b. With heavy growth of sprouts....... 0.060 - 0.080

8. Heavy stand of timber, a few down trees, little undergrowth:
    a. Flood depth below branches .... 0.100 - 0.120
    b. Flood depth reaches branches .... 0.120 - 0.160

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 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

MANNING'S ROUGHNESS COEFFICIENTS FOR SHEET FLOW

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

Source: Chow, V.T., 1959, Open Channel Hydraulics, McGraw-Hill, New York, NY

NEBRASKA
Good Life. Great Journey.
DEPARTMENT OF TRANSPORTATION

Jim Pillen, Governor

The Nebraska Department of Transportation Drainage Design and Erosion Control Manual Appendix C, "Pipe Material Policy", October 2023, has been approved for use.

Approved by: ______2023.10.24 13:24:07-05'00'______/______________
Mick Syslo, Roadway Design Engineer, P.E. Date

DocuSigned by:
Brendon J Schmidt
0374BA31999148F...

Approved by: ______________________/ 10-26-2023
Brendon Schmidt, Materials and Research Engineer, P.E. Date

Approved by: ______________________/______________
David Mraz, FHWA Date
Engineering and Operations Team Leader, NE FHWA

PIPE MATERIAL POLICY

Policy: This policy will replace all previous policies regarding the selection of pipe material for cross drains, drive pipe, drop pipe, storm sewers, and railroad pipe. Under this policy, designers will select the allowable pipe material options for each installation. The contractor will choose the final pipe material from the list of options provided.

The following topics are discussed in more depth:
- Types of pipe specified in this policy
- Maximum permissible diameter of standard pipe
- Design values for Manning Coefficient, (n)
- Flared end sections
- Minimum and maximum fill heights
- Excavation, bedding, and backfill requirements
- Functional use of different pipes
- Connections

Per the guidance given in the FHWA Memo “Section 1525 of MAP-21 – State Autonomy for Culvert Pipe Selection” (Culvert Selection - Construction Program Guide - Contract Administration - Construction - Federal Highway Administration (dot.gov), Revised September 27, 2019), LPAs may select only one type of pipe within each category of the Pipe Policy and are not required to specify several choices when developing Plans and Specifications for Federal Aid highway construction projects.

TYPES OF PIPE SPECIFIED IN THIS POLICY

RCSP Reinforced Concrete Sewer Pipe

RCP Reinforced Concrete Pipe

MCCMP Metallic Coated Corrugated Metal Pipe, which includes:
GCCMP Galvanized (Zinc) Coated Corrugated Metal Pipe and
ACCMP Aluminum Coated Corrugated Metal Pipe

PCCMP Polymer Coated Corrugated Metal Pipe

HDPE or PP High Density Polyethylene Pipe, which includes:
HDPE-CI (Corrugated Interior) and
HDPE-SI (Smooth Interior), including Steel Reinforced Ribbed Pipe
PP-CI (Polypropylene Pipe, Corrugated Interior)
PP-SI (Polypropylene Pipe, Smooth Interior)

PVC Polyvinyl Chloride Pipe

The numerical designations shown below will be used by designers for specifying the various types of pipe:

Type	1	2	3	4	5	6	7	8
	RCSP	RCP	GCCMP	ACCMP	PCCMP	HDPE or PP CI	HDPE or PP SI	PVC

Designers shall identify all pipes by their corresponding Type number. For example, if RCSP, PCCMP and PVC are appropriate options, the designation would be 1-5-8. The Type "1" in this example is a RCSP.

Note: Normally, the contractor has the option of selecting the class of pipe, and type of installation in accordance with the fill height tables shown on the plans. However, for fill heights less than one foot, the designer must specify the class of pipe required. Refer to the Drainage Design and Erosion Control Manual for live load information. Use Attachment 2 for maximum fill height data.

MAXIMUM PERMISSIBLE DIAMETER OF STANDARD PIPE

The maximum allowable inside diameter for the various pipes are shown in Attachment 2. These pipes are standard manufactured sizes. Sizes other than those shown are considered special designs that must be submitted to NDOT for approval.

DESIGN VALUES FOR MANNING COEFFICIENT, (n)

The selected (n) value for design purposes when using corrugated pipe (MCCMP, PCCMP, and HDPE or PP-CI) is 0.024. The design (n) value for smooth interior pipes (RCSP, RCP, PVC, and HDPE or PP-SI) is 0.012. When it is necessary to determine the true magnitude of the pipe outlet flow velocity, designers should use the actual (n) value recommended by the manufacturer to perform computations. When designing for outlet control and *both* corrugated and smooth pipe are selected, the designer will use an (n) value of 0.024. A Manning (n) value of 0.012 shall be used when *only* smooth interior pipe are specified.

FLARED END SECTIONS

Use concrete flared end sections (CFES) for all concrete pipes. Specify metal flared end sections (MFES) for metal and plastic pipes when flared end sections are required. Flared end sections are not required for drive pipes unless they are to be installed within the Horizontal Clear Zone. Safety flared end sections manufactured with a 10:1 slope and equipped with protective cross bars must be provided for the approach end of all drive pipes placed within the Horizontal Clear Zone.

MINIMUM AND MAXIMUM FILL HEIGHTS

Fill height determines the amount of dead load (or live load) that is imposed upon a culvert pipe. Minimum fill height is defined as the vertical distance measured from the top of the conduit to the bottom of the pavement or shoulder surfacing at its lowest point. Maximum fill height is defined as the vertical distance measured from the top of the conduit to the top of the pavement at its highest point. Minimum fill height for all culverts is one foot except for HDPE, PP or PVC which have a minimum fill height of two feet. The designer should review the live load computations as shown in the Drainage Design and Erosion Control Manual for special circumstances when this one foot minimum cannot be maintained. The maximum fill height that a pipe can withstand depends greatly on the type of bedding and backfill, pipe size, and pipe material. Refer to Attachment 2, along with the appropriate Standard Plans, for guidelines in specifying various types of pipe.

EXCAVATION, BEDDING, AND BACKFILL REQUIREMENTS

Refer to Standard Plan 411, "Bedding and Backfill Requirements for Concrete Pipe" (http://www.roads.nebraska.gov/business-center/design-consultant/stand-spec-manual/), for installation details. Standard Plan 411, Sheet 4, "Bedding and Backfill Requirements for MCCMP, PCCMP, and Plastic Pipe", shows details for installing flexible pipe. Granular material is required for all flexible MCCMP, PCCMP, and plastic pipe installed under surfaced roadways. Unless special circumstances exist, granular material is not required for drive pipe, drop pipe, or temporary pipe installed outside the surfaced roadway prism.

On trench installations, the trench width depends on the outside diameter of the pipe and the side clearance requirement on each side of the pipe as shown on the Standard Plans. Trench depth depends on the size of the pipe and the flow line location relative to the ground surface. On embankment installations, where the flowline of the pipe is above the natural ground, the culvert contractor is required to raise the ground along the centerline of the pipe to an appropriate elevation above the flowline (See Standard Plan 411). This embankment must be wide enough to excavate to the proper depth and install the pipe at the flowline shown on the cross sections. A contractor may choose to provide an embankment deep enough to use a trench installation. All excavations will be determined as established quantities using the method of measurement as shown in the current Nebraska Standard Specifications for Highway Construction.

FUNCTIONAL USE OF DIFFERENT PIPES

The functional usage of the various pipes that designers specify is summarized in Attachment 3. The plus sign shown in the Functional Usage table signifies that the use of a particular pipe material is acceptable for that function. The minus sign indicates where material use is prohibited. Designers may refer to the flow chart in Attachment 1 for assistance in the pipe selection process.

CONNECTIONS

All RCP and RCSP connections under the roadway prism (or back-to-back of curb-line on urban projects) shall be Tongue and Groove (T&G) or modified T&G type, and have watertight joints (using cement mortar, fibered roof coating, or gaskets) in accordance with the Nebraska Standard Specifications for Highway Construction. All plastic and CMP pipe under the roadway prism (or back-to-back of curb-line) must be installed with approved watertight joints. CMP and plastic pipe outside the roadway prism (or back of curb-line) may be installed with soil tight connecting bands or other approved soil tight joints. All pipe used for sewer applications must be installed with approved watertight connections.

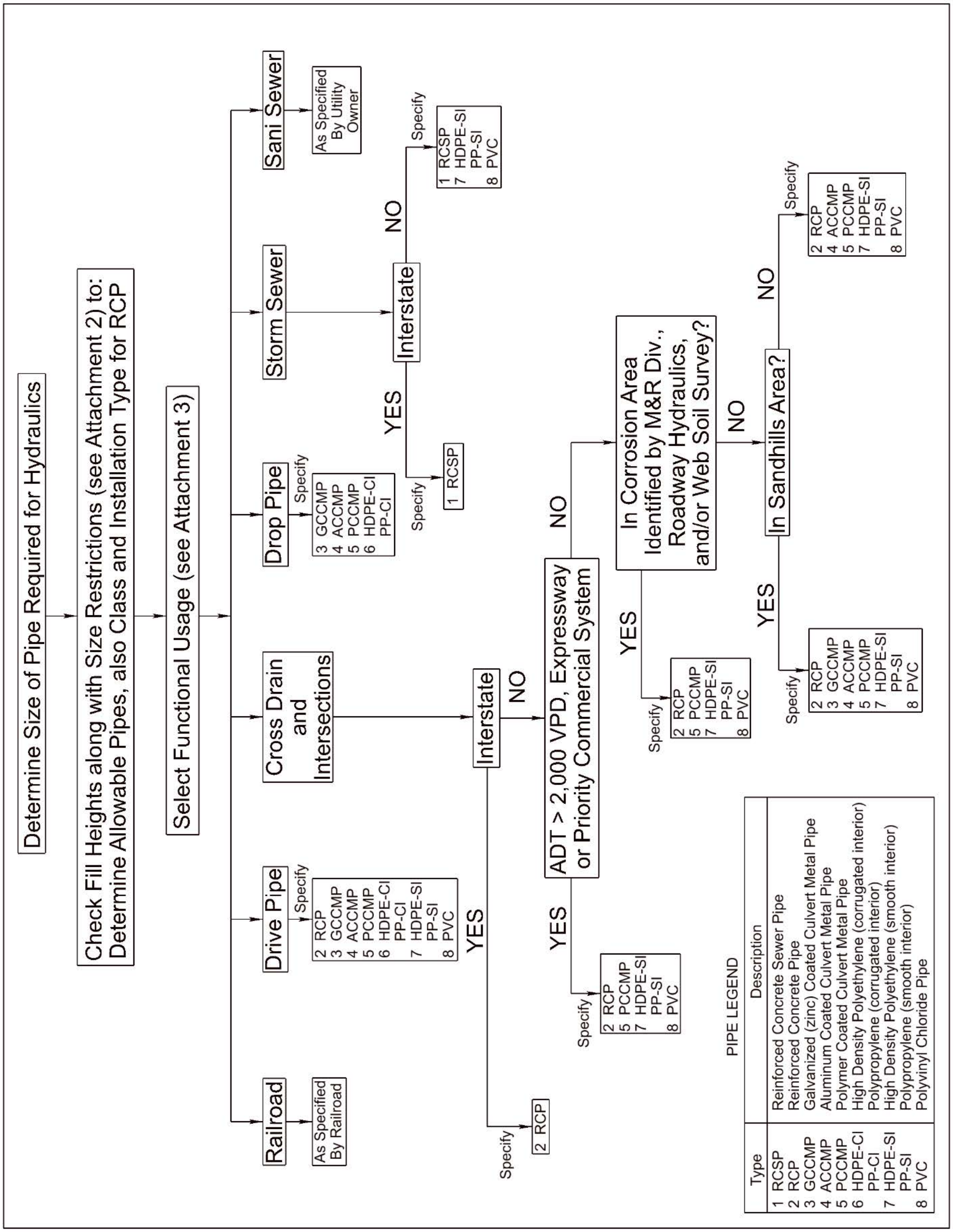
CULVERT EXTENSIONS

Existing culverts will be extended using the same material as the existing structure. If the plans call for extension of Corrugated Metal Pipe Culvert, the pipes shall be connected by the contractor with an approved connecting band. When the plans designate the extension of a Concrete Pipe Culvert, the connection shall be made by enclosing the connecting joint with a concrete collar.

TEMPORARY PIPE

The designer should contact the District to determine culvert type when temporary pipe is to be placed under a temporary (2 years or less) facility such as a temporary road or median crossover. If the pipe is to be furnished by the State, the construction note will call for INSTALLING the pipe. If the pipe is to be furnished by the contractor, the construction note will call for BUILDING the pipe. The District will also determine if the pipe will be salvaged to the State when the temporary roadway is removed. Corrugated metal pipe (Type 3, 4 or 5) will be allowed for temporary roadways. Granular backfill for temporary structures Type 3, 4 or 5 is not required, unless called for in the plans.

ATTACHMENT 1 PIPE SELECTION FLOW CHART



ATTACHMENT 2 PIPE SIZE AND FILL HEIGHT REQUIREMENTS

Maximum Permissible Diameters Of Standard Pipe

The following interior pipe diameters (in inches) are considered maximum standard sizes. Larger sizes may be allowed by special design approved by NDOT.

Type	1		2	3	4	5	6	7	8
	RCSP		RCP	GCCMP	ACCMP	PCCMP	HDPE or PP		PVC
	III	IV	V				CI	SI	
	108	72	48	84	84	84	60	60	48

Maximum Fill Heights (feet) For Round Concrete Pipe

	Installation Type 3			Installation Type 2			Installation Type 1		
Pipe Size (in)	Class III	Class IV	Class V	Class III	Class IV	Class V	Class III	Class IV	Class V
15	12	15	21	15	19	26	23	28	40
18	12	17	24	16	22	30	24	32	45
21	13	19	26	16	24	32	25	37	48
24	13	19	26	17	24	33	25	32	45
27	13	17	26	17	21	34	23	26	51
30	12	14	25	15	17	32	20	21	49
36	10	16	24	13	21	31	20	31	47
42	10	15	23	13	19	29	20	29	44
48	10	14	22	13	18	29	20	28	43
54	10	14		13	17		20	27	
60	9	14		12	18		19	28	
66	9	14		12	18		19	28	
72	9	14		12	18		19	28	
78	9			12			19		
84	9			12			19		
90	9			12			20		
96	9			12			19		
102	10			13			20		
108	10			14			22		

The Type 3 Installation (shaded) is the NDOT Standard. See Standard Plan 411 (Bedding and Backfill for Concrete Pipe) for additional information about table development and usage.

Maximum Fill Heights For Flexible Pipe

The maximum dead load fill height for HDPE, PP, PVC, and CMP is set at 40 feet, using the bedding and backfill requirements as shown in Standard Plan 411 (http://www.roads.nebraska.gov/business-center/design-consultant/stand-spec-manual/). Consult with the pipe manufacturer when designing for fills greater than 40 feet, or when special situations are encountered that are beyond the scope of this policy. When installing flexible pipe outside the roadway prism (or back of curb-line on urban projects), and when granular materials are not used as shown in this policy, the maximum fill height is set at 20 feet (standard proctor test density for non-granular material must be greater than 95%).

ATTACHMENT 3 PIPE DESIGN APPLICATIONS AND EXAMPLES

FUNCTIONAL USAGE

Type	1	2	3	4	5	6	7	8
Functional Usage	**RCSP (All Classes)**	**RCP (All Classes)**	**GCCMP Galvanized (Zinc) Coated CMP**	**ACCMP Aluminum Coated CMP**	**PCCMP Polymer Coated CMP**	**HDPE-CI or PP-CI Corrugated Interior**	**HDPE-SI or PP-SI Smooth Interior**	**PVC**
Cross Drain & Intersections	-	+	See Footnotes		+	-	+	+
Drive Pipes	-	+	+	+	+	+	+	+
Drop Pipe	-	-	+	+	+	+	-	-
Railroad	As Specified by the Railroad							
Storm Sewer	+	-	-	-	-	-	+	+
Sani. Sewer	As Specified by the Utility Owner							

Cross Drain and Intersection Footnotes:

- Galvanized and Aluminized Coated (Types 3 & 4) corrugated metal pipes will not be permitted in the southeast counties of Gage, Nemaha, Richardson, Pawnee, Johnson, Otoe or any other locations that are unsuitable for corrugated metal pipe as designated by M&R Division, Roadway Design Hydraulics, and/or Web Soil Survey information ("high" risk of corrosion).
- Galvanized CMP (Type 3)---Allowed for ADT < 2,000 VPD in the Sandhills, unless identified corrosion areas exist.
- Aluminum Coated CMP (Type 4)---Allowed for ADT < 2,000 VPD, unless identified corrosion areas exist.

EXAMPLES OF CULVERT TYPES SPECIFIED*

Pipe Dia. (in.)	Max. Fill Height (ft.)	Pipe Function	Location	Type Specified
48	20	Cross drain	Sandhills/ ADT < 2,000 VPD /no corrosive areas	2-3-4-5-7-8
54	20	Cross drain	ADT < 2,000 VPD /corrosive area	2-5-7
36	24	Cross drain	Statewide/ ADT ≥ 2,000 VPD	2-5-7-8
30	5	Cross drain	Statewide/Priority Commercial System	2-5-7-8
42	25	Cross drain	Not Sandhills/ ADT < 2,000 VPD /no corrosion	2-4-5-7-8
54	5	Storm sewer	Statewide	1-7
36	5	Storm sewer	Statewide	1-7-8
48	15	Drive pipe	Statewide	2-3-4-5-7-8
24	15	Drive pipe	Statewide	2-3-4-5-6-7-8
24	5	Drop pipe	Statewide	3-4-5-6

* Assuming bedding and backfill requirements shown in Standard Plan 411

PIPE LEGEND

Type	Description
1 **RCSP**	Reinforced Concrete Sewer Pipe
2 **RCP**	Reinforced Concrete Pipe
3 **GCCMP**	Galvanized (zinc) Coated Culvert Metal Pipe
4 **ACCMP**	Aluminum Coated Culvert Metal Pipe
5 **PCCMP**	Polymer Coated Culvert Metal Pipe
6 **HDPE-CI** **PP-CI**	High Density Polyethylene (corrugated interior) Polypropylene (corrugated interior)
7 **HDPE-SI** **PP-SI**	High Density Polyethylene (smooth interior) Polypropylene (smooth interior)
8 **PVC**	Polyvinyl Chloride Pipe

COMMENTARY

Plastic Pipe

Plastic pipe (HDPE, PP and PVC) may be used for driveway, underdrain, sewer and roadway cross drain as well as other drainage applications. The Standard Plans for flexible pipe show: installation, material, and backfill requirements. Also, due to the non-corrosive nature of these materials, plastic pipe is included for use in areas where corrosion of metallic coated culverts is a concern.

Pipe Installations under Pavement

In order to extend the design life (by improving structural performance, reducing settlement and joint movement etc.) of surfaced roadways, cross section details for all pipes (concrete, metal and plastic) have been developed. Flexible pipes require a granular material envelope to improve in-place structural performance, while reducing compaction effort and pipe movement during installation. Use of granular bedding and backfill with metallic coated pipes also has the benefit of reducing soil-side corrosion. Use of polymer coated CMP effectively reduces interior corrosion as well as exterior soil-side corrosion, and therefore, is allowed for use under all roadways.

Standard Plans for concrete pipe installations have been developed using computer programs such as SIDD and PIPECAR (*such programs have been created through the efforts of organizations such as ANSI, FHWA and the Concrete Pipe Association*). These plans provide options for the designer, contractor, and pipe manufacturer in regard to pipe class selection and installation. Under this policy the contractor will be allowed to select the type of installation and class of pipe based upon available fill height information shown on the plans. In addition, the requirements shown for bedding and backfill eliminate previous requirements for shaping the trench bottom to fit the contour of the pipe.

The information contained in Appendix D: Storm Sewer Policy, dated August 2006, has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

STORM SEWER POLICY

Purpose: To provide policy clarification for applicable design guidelines and apportionment of costs for highway drainage design and for the upgrade of Municipal Drainage Facilities on projects to be constructed in whole or on part within the corporate limits of a municipality.

Definitions:

Project – A State Highway construction project.

Highway Drainage Facilities – The curb inlets, storm sewers, drainage ditches and other facilities designed to collect and drain waters from the highway and other land. These Highway Drainage Facilities are usually located within the highway right-of-way.

Municipal Drainage Facilities – The storm sewers, drainage ditches, drainageways and other facilities used by the city for the drainage of waters including waters draining from the Highway Drainage Facilities. These Municipal Drainage Facilities are usually located outside of the highway right-of-way.

Apportionment of Costs: The State and the Municipality will share all costs associated with the project except for any costs designated herein to be solely the responsibility of the Municipality. The policy regarding the cost sharing on projects of this type is found in the current Department of Roads Operating Instruction 60-11, entitled “Municipal Cost Sharing”.

Highway Drainage Design: The State will design its’ Highway Drainage Facilities to collect and discharge stormwater based on the design guidelines set out in Chapter One of the Drainage Design and Erosion Control Manual, if feasible based on all applicable considerations. When the Highway Drainage Facilities connect or drain into or are intended to connect or drain into the Municipal Drainage Facilities, the State will calculate and notify the Municipality of the capacity of the Municipal Drainage Facilities to convey waters away from the highway. When the capacity of the Municipal Drainage Facilities prevents the State from complying with the design guidelines, and when the Municipality will not upgrade its’ Municipal Drainage Facilities, the State will design its’ project to collect and discharge stormwater based on the hydraulic capacity of the existing Municipal Drainage Facilities, except for the instances described below.

A. When the Municipality provides the State with reasonable written assurances of a present plan for a future upgrade of its’ Municipal Drainage Facilities. The Municipality shall provide the State with the details of its’ proposed improvement and the Highway Drainage Facilities will be designed to either match the capacity of the Municipality’s proposed drainage facilities or to convey the design event determined by the State to be feasible based on all applicable considerations feasible.

B. When the Municipality requests that the project include an upgrade of its' Municipal Drainage Facilities to be paid for solely by the Municipality. The Municipality shall enter into an agreement with the State concerning this upgrade of its' facilities prior to the State beginning the design of the project. The Municipality will pay all necessary costs associated with the upgrade of its' Municipal Drainage Facilities, including the costs of engineering, right-of-way and construction.

C. Examples:

Example 1:

Given: The project's storm sewer will outlet or drain into an existing municipal storm sewer, located downstream of the project's right-of-way, that can convey a 5-year storm. The Municipality has no plans or desire to upgrade their downstream storm sewer in the foreseeable future.

Result: The project's storm sewer will be designed to convey a 5-year storm, if feasible, based on all applicable considerations.

Example 2:

Given: The project's storm sewer will outlet or drain into an existing municipal storm sewer, located downstream of the project's right-of-way, that can convey a 5-year storm. The Municipality desires to upgrade their downstream storm sewer to convey a 10-year storm and have it upgraded as a part of the project.

Result: The project's storm sewer will be designed to convey a 10-year storm, if feasible, based on all applicable considerations. The Municipality will be responsible for 100 percent if the costs associated with the upgrade of the storm sewer located downstream of the project.

Example 3:

Given: The project's storm sewer will outlet or drain into an existing municipal storm sewer, located downstream of the project's right-of-way, that can convey a 5-year storm. The Municipality plans to upgrade their downstream storm sewer to convey a 10-year storm within 8 years of the completion of the project.

Result: After documentation for the future downstream storm sewer upgrade is received from the Municipality, the project's storm sewer will be designed to convey a 10-year storm, if feasible, based on all applicable considerations.

Additional Drainage Waters: If the State determines that significant additional drainage waters will be conveyed in the Highway Drainage Facilities because of the design of the project, the State will determine, on a case-by-case basis, whether the Municipal Drainage Facilities will be upgraded and whether the State will share in any portion of the cost of such upgrade. The extent of the upgrade to the Municipal Drainage Facilities and the division of cost for such upgrade will be a matter of negotiation to be resolved and set forth in an agreement with the Municipality.

APPENDIX E
DESIGN FORMS AND CHECKLISTS

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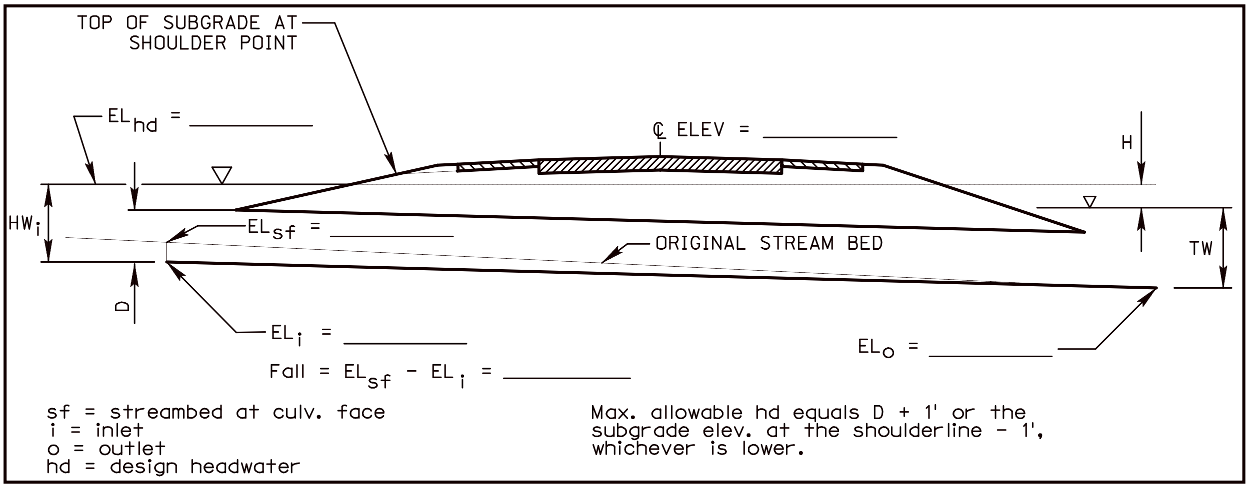


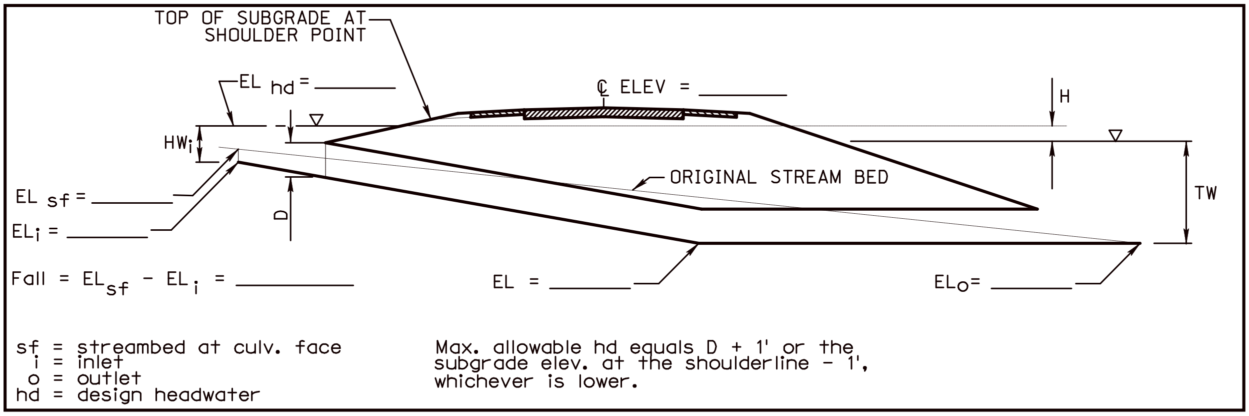
Project Location: ______________________________

Project No: ______________ Control No: __________ Designer: __________

Culvert Station: ______________ Stream Name: ______________

1- Roadway Design Standard ______________
2- Average Daily Traffic (and date) ______________
3- Design Frequency ______________
Comments ______________
4- Design Discharge ______________
5- Design Tailwater Level ______________
6- Vertical Orientation (Broken Back, Breaks?) ______________
7- Slope:
Upstream: ________ Culvert: ________ Downstream: ________
8- Horizontal Orientation ______________
9- Skew ______________
10- Fill Height (See the "Pipe Material Policy"):
Maximum ________ Minimum ________
11- Culvert Shape:
Circular: _____ Box or Rectangular: _____ Arch: _____ Elliptical: _____
12- Culvert Type:
Concrete: _____ Metal: _____ Plastic: _____
13- End Treatment, (FES, Headwall, etc.) ______________
14- Design Headwater:
Depth: ________ Elevation: ________
15- Elevations of Buildings / Upstream Considerations ______________
Comments ______________
16- Outlet Velocity ______________
17- Downstream Considerations ______________
18- Velocity Protection Device ______________
19- Velocity Control Device ______________
20- Alternates Considered ______________
21- Economic Comparison of Alternatives ______________
22- Comments ______________
____
____
____
____

Exhibit E.1 Culvert Design Checklist/Data



Project Location: ____________________

Project No: ____________ Control No: ________ Designer: ________

Culvert Station: ____________ Stream Name: ____________

1- Roadway Design Standard ____________
2- Average Daily Traffic (and date) ____________
3- Design Frequency ____________
   Comments ____________
4- Design Discharge ____________
5- Design Tailwater Level ____________
6- Vertical Orientation (Broken Back, Breaks?) ____________
7- Slope:
   Upstream: ________ Culvert: ________ Culvert: ________
   Culvert: ________ Downstream: ________
8- Horizontal Orientation ____________
9- Skew ____________
10- Fill Height (See the "Pipe Material Policy"):
   Maximum________ Minimum________
11- Culvert Shape:
   Circular: _____ Box or Rectangular: _____ Arch: _____ Elliptical: _____
12- Culvert Type:
   Concrete: _____ Metal: _____ Plastic: _____
13- End Treatment (FES, Headwall, etc.) ____________
14- Design Headwater:
   Depth: ________ Elevation: ________
15- Elevations of Buildings / Upstream Considerations ____________
   Comments ____________
16- Outlet Velocity ____________
17- Downstream Considerations ____________
18- Velocity Protection Device ____________
19- Velocity Control Device ____________
20- Alternates Considered ____________
21- Economic Comparison of Alternatives ____________
22- Comments ____________

____
____
____
____

Exhibit E.2 Culvert Design Checklist/Data

CULVERT DESIGN FORM

PROJECT: ______________________________

C.N. ______________________________

STATION: ____________________

SHEET ________ OF ________

DESIGNER/DATE: __________ / __________

REVIEWER/DATE: __________ / __________

HYDROLOGICAL DATA

☐ METHOD: ____________________

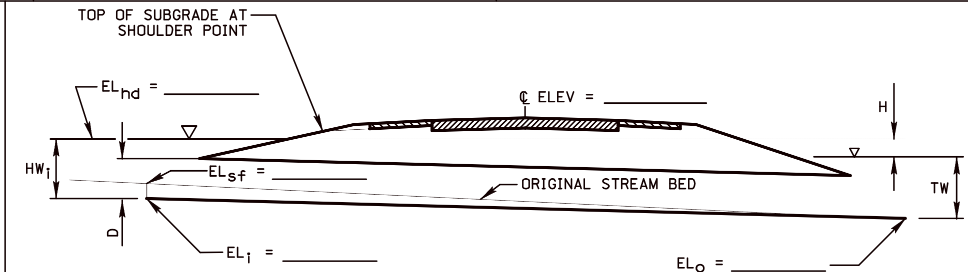
☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______

☐ CHANNEL SHAPE: ____________________

☐ ROUTING: __________ ☐ OTHER: __________

DESIGN FLOWS/TAILWATER

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



Max. allowable hd equals D + 1' or the subgrade elev. at the shoulderline - 1', whichever is lower.

L_a = ________

$S = EL_i - EL_o / L_a$ = ________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	Headwater Calculations: Inlet Control HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	Outlet Control TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATIONS	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) Use Q/NB for box culverts

(2) $HW_i/D = HW/D$ or HW_i/D from design charts

(3) Fall $= EL_{sf} - EL_i$; fall is zero for culverts on grade

(4) $EL_{hi} = HW_i + EL_i$ (Invert of inlet control section)

(5) TW based on downstream control or flow depth in channel.

(6) $h_o = TW$ or $(d_c+D/2)$ (Whichever is greater)

(7) $H = [1+k_e+(29n^2 L)/R^{1.33}]\ V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

COMMENTS/DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____________________

SHAPE: ____________________

MATERIAL: __________ n ________

ENTRANCE: ____________________

DEFINITIONS:

A. Cross-Sectional Area of the Barrel
a. Approximate
dc. Critical Depth
D. Interior Height of Culv. Barrel
f. Culvert Face
g. Acceletration Due to Gravity (32.2 ft/s/s)
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
ke. Entrance Loss Coefficient
L. Length of Culvert Barrel
N. Number of Culvert Barrels
NB. Number of Boxes
o. Outlet
Q. Uniform Discharge, cfs
S. Slope of Culvert
sf. Streambed at Culvert Face
TW. Tailwater Depth Above the Outlet Invert
V. Average Velocity in Culv. Barrel (V=Q/A)

Exhibit E.3 Culvert Design Form

Drainage Computation Form

Location: ______________________
Project Name: ______________________
Project No.: ______________________

Return Period: ______________________
Zone: ______________________
Drainage Area: ______________________
Pipe Material: ______________________
N Value: ______________________

Computed By: ______________________
Date: ______________________
Checked By: ______________________
Date: ______________________
Page: ______________________

HYDROLOGY												STORM SEWER HYDRAULICS					
							INCREMENTAL			CUMULATIVE							
STR. No.	Sta.	From / To	Inlet Time T_i (min)	Travel Time T_t (min)	Time of Conc. T_c (min)	Rainfall Intensity I (in/hr)	Area A (acres)	Runoff Coeff. C	C x A	C x A	Runoff Q (cfs)	Length (ft)	Slope (%)	Size (in)	Capacity Q_{full} (cfs)	Vel V_{full} (fps)	Remarks

Exhibit E.4 Drainage Computation Form

Hydraulic Grade Line Computation Form

Location: ____________
Project Name: ____________
Project No.: ____________

Return Period: ____________
Zone: ____________

Computed By: ____________
Date: ____________
Checked By: ____________
Date: ____________
Page: ____________

HYDRAULIC GRADE LINE CALCULATIONS

		PIPE DATA														ENERGY LOSSES							
STR. No.	Sta.	From / To	Pipe Size (in)	n	Pipe Length (ft)	Pipe Slope (%)	Area A (acres)	Runoff q (cfs)	V (q) (fps)	Capacity Q full (cfs)	q/Q full	d/D	Normal Depth (ft)	Critical Depth (ft)	Type of Flow	H_f (ft)	H_{j2} (ft)	H_e (ft)	H_m (ft)	H_b (ft)	EGL Eleva.	Vel. Head (ft)	HGL Eleva.

Exhibit E.5 Hydraulic Grade Line Computation Form

APPENDIX F
NOMOGRAPHS AND CHARTS FOR CULVERT DESIGN

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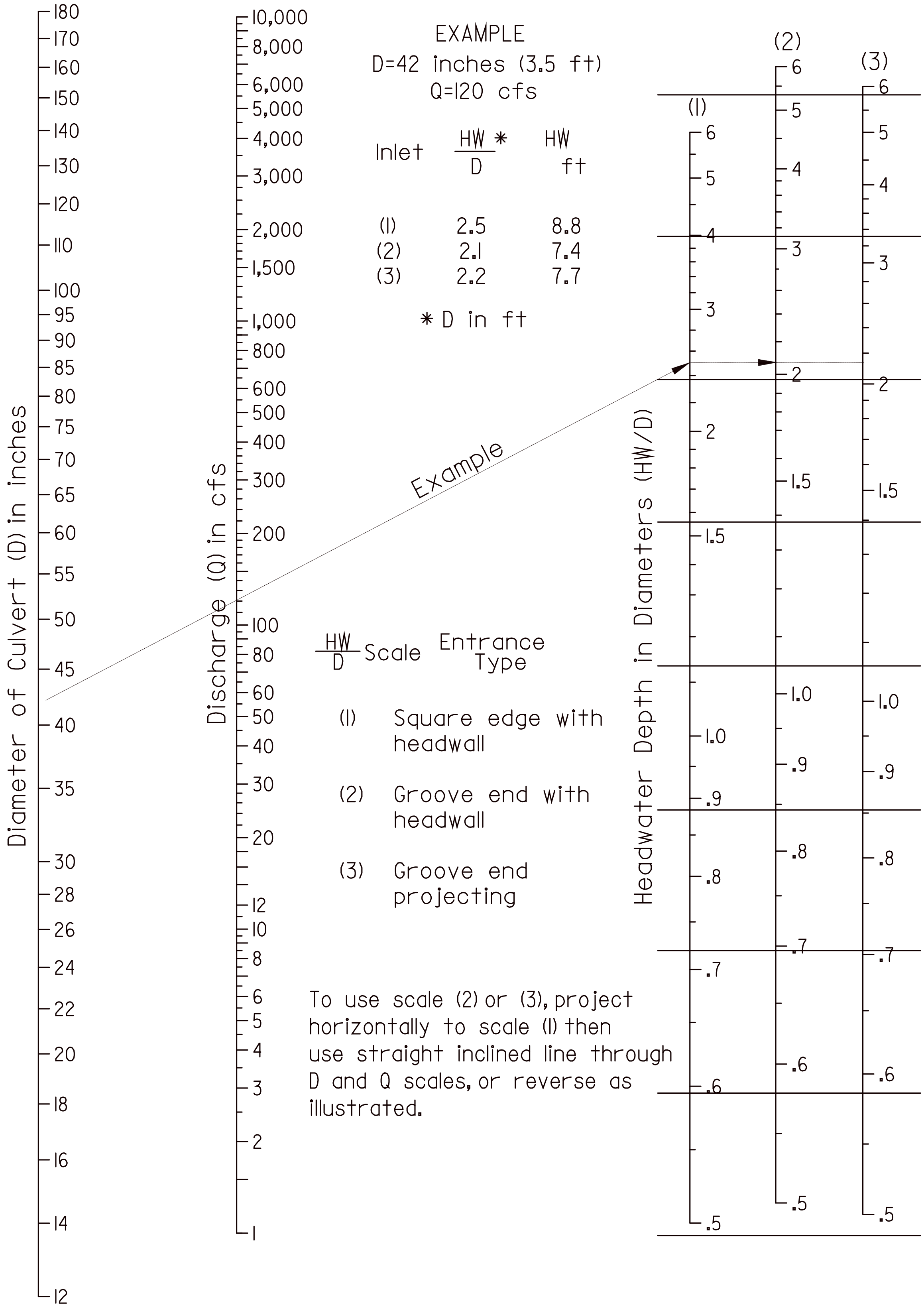


Exhibit F.1 Headwater Depth for Concrete Pipe Culverts with Inlet Control (Source: Reference F.1)

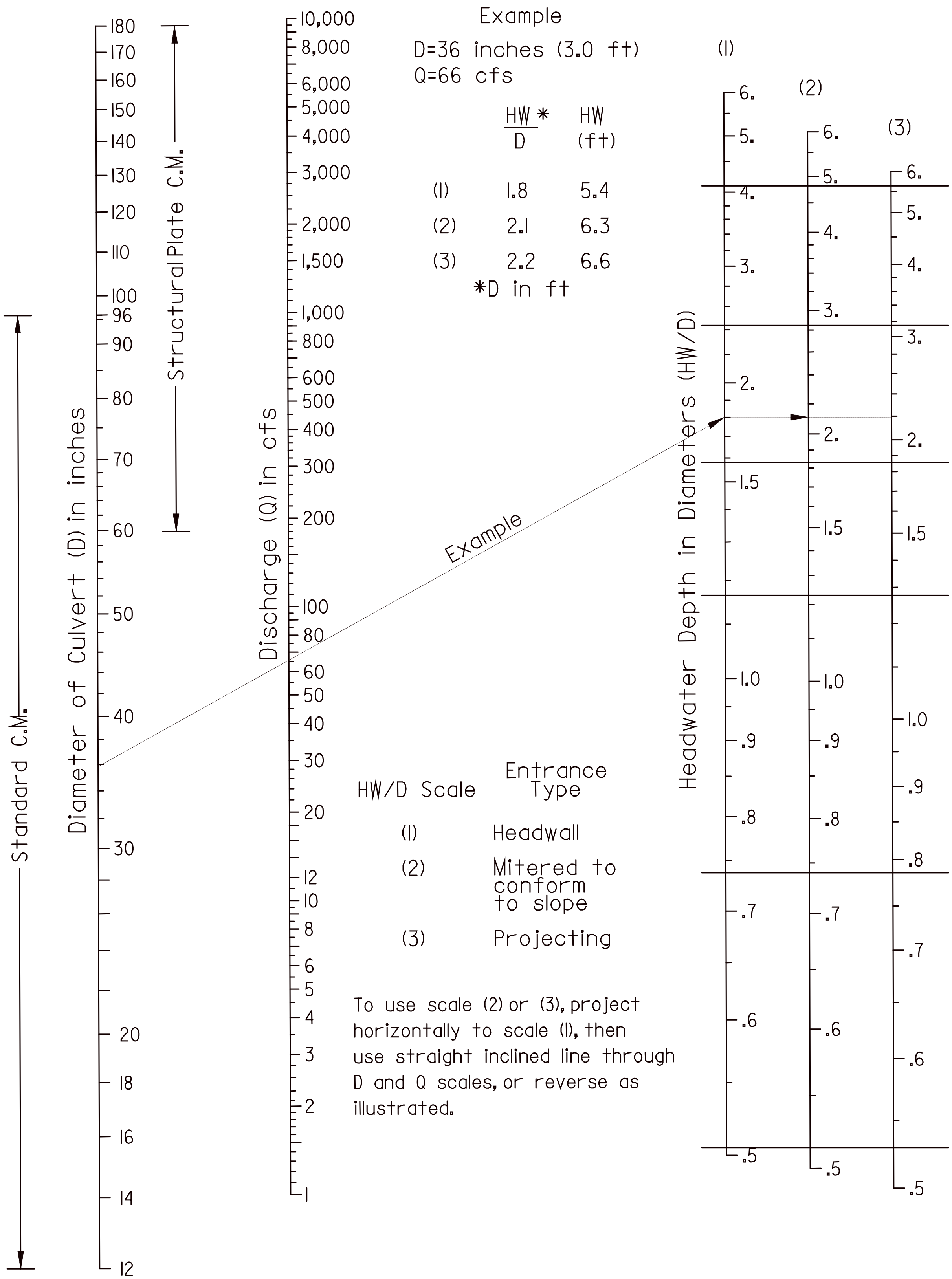


Exhibit F.2 Headwater Depth for CMP Culverts with Inlet Control
(Source: Reference F.1)

b/D	a/D	c/D	d/D	Entrance Type
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B

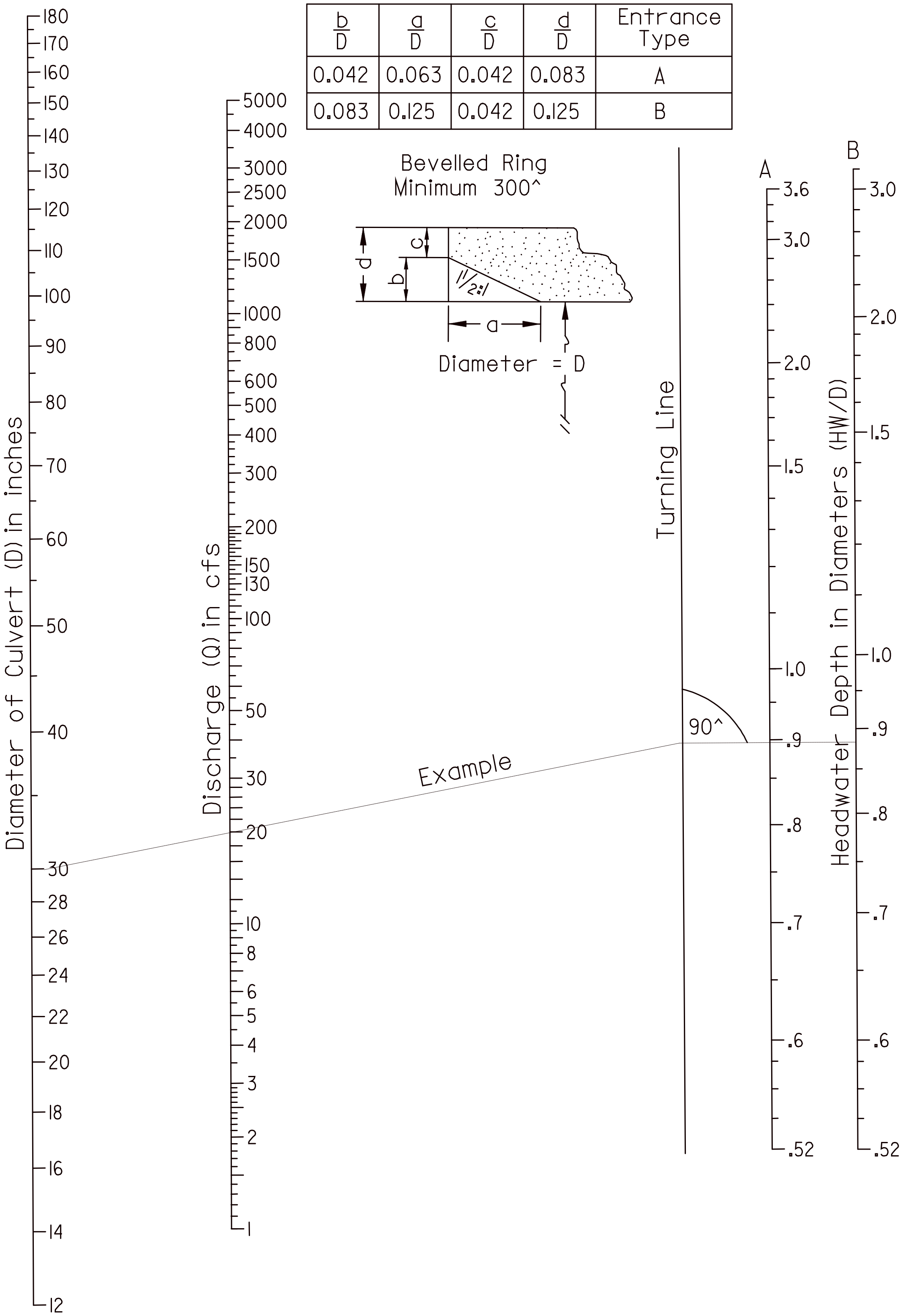


Exhibit F.3 Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control (Source: Reference F.1)

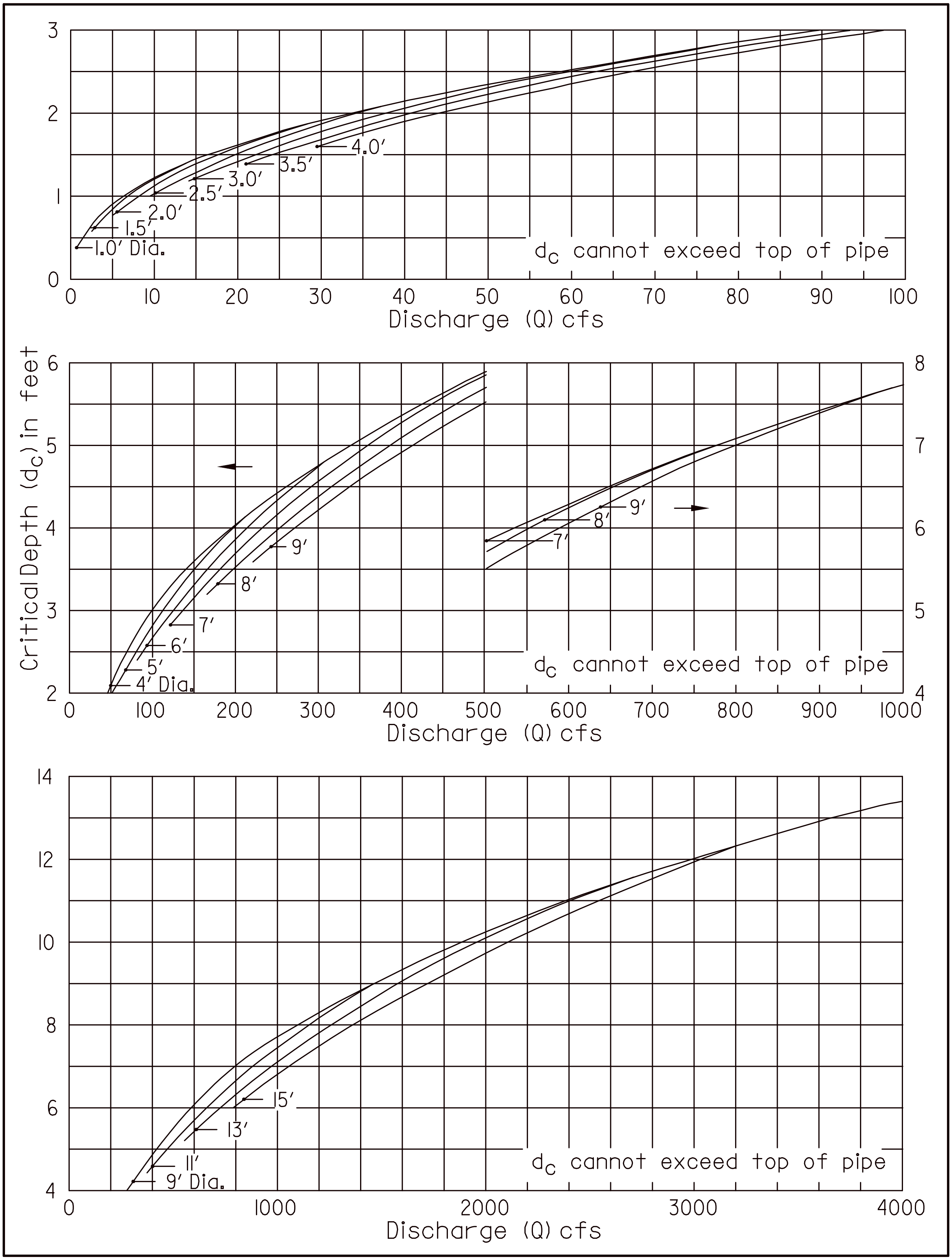


Exhibit F.4 Critical Depth for Circular Pipe
(Source: Reference F.1)

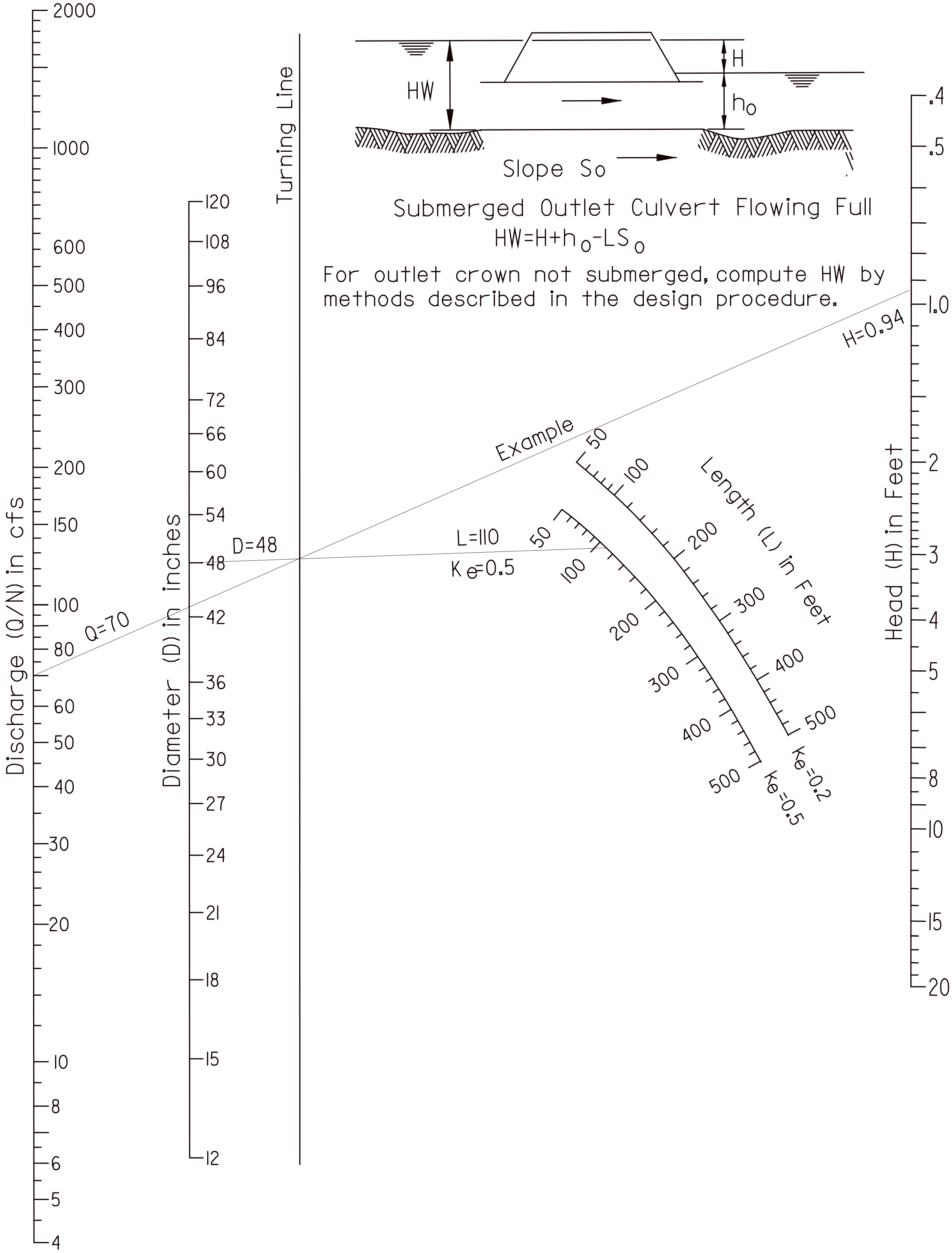


Exhibit F.5 Head for Concrete Pipe Culverts Flowing Full (n=0.012)
(Source: Reference F.1)

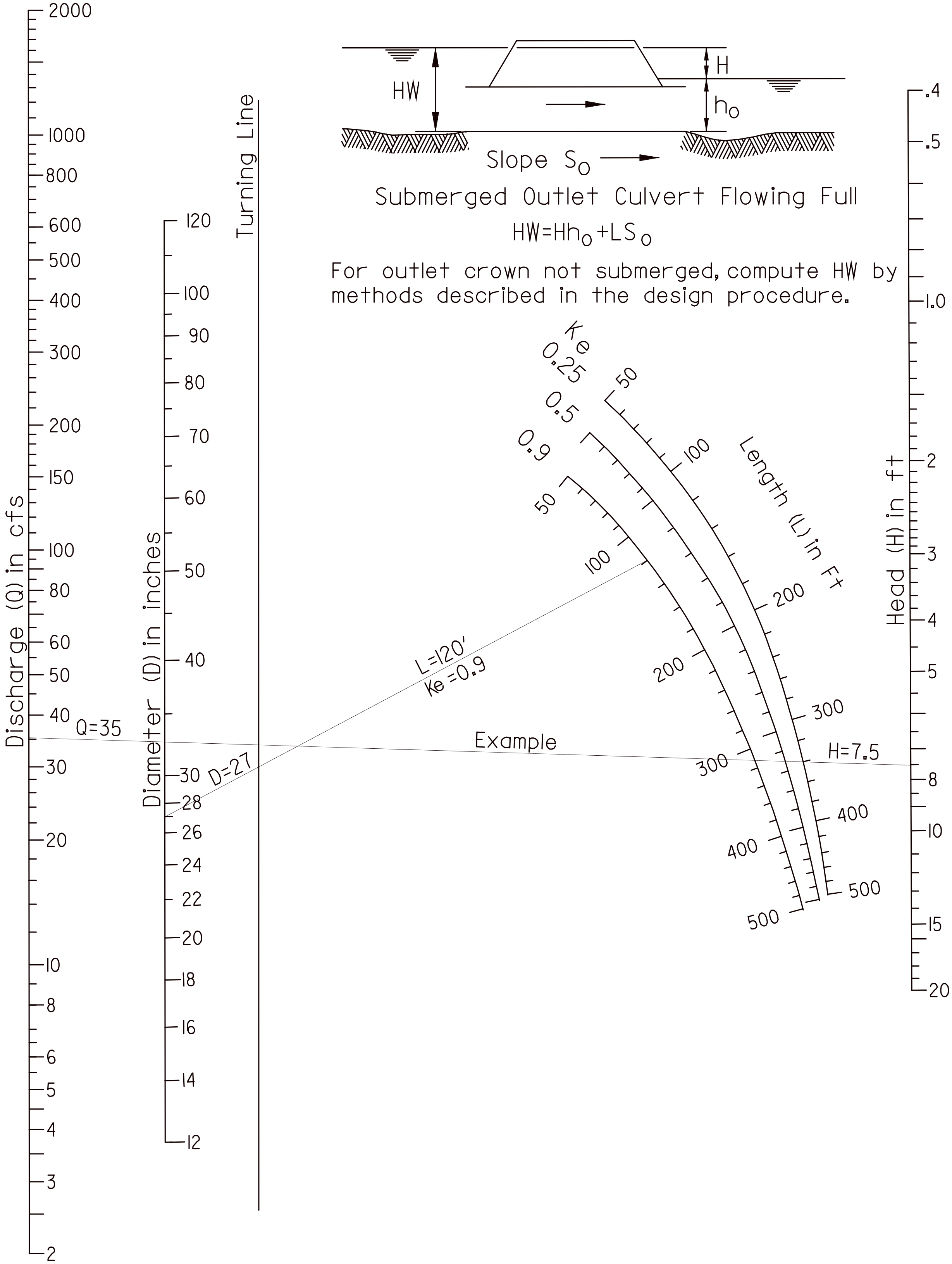


Exhibit F.6 Head for Standard CMP Culverts Flowing Full (n=0.024)
(Source: Reference F.1)

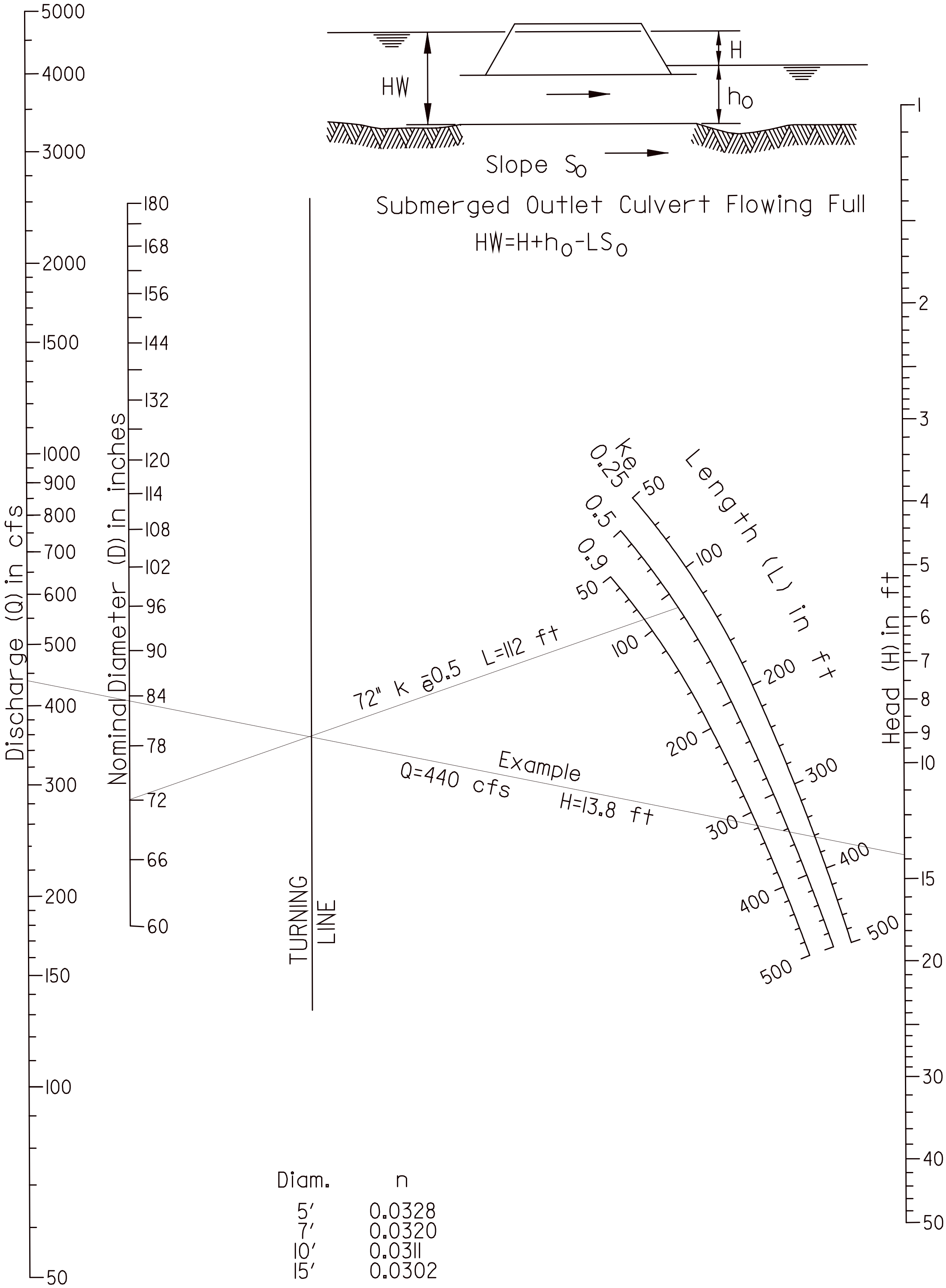


Exhibit F.7 Head for Structural Plate CMP Culverts Flowing Full (n=0.0328 to 0.0302)
(Source: Reference F.1)

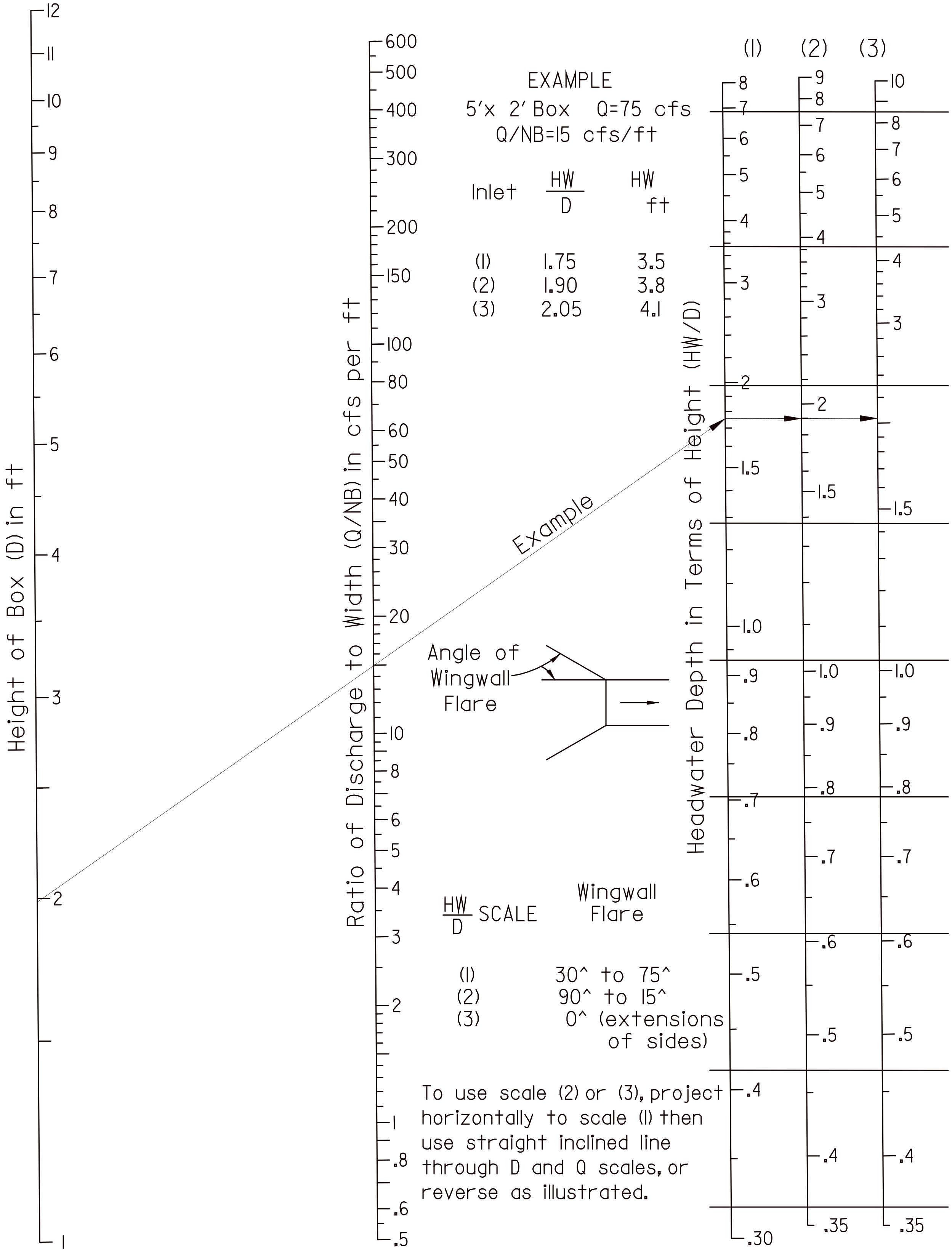


Exhibit F.8 Headwater Depth for Box Culverts with Inlet Control
(Source: Reference F.1)

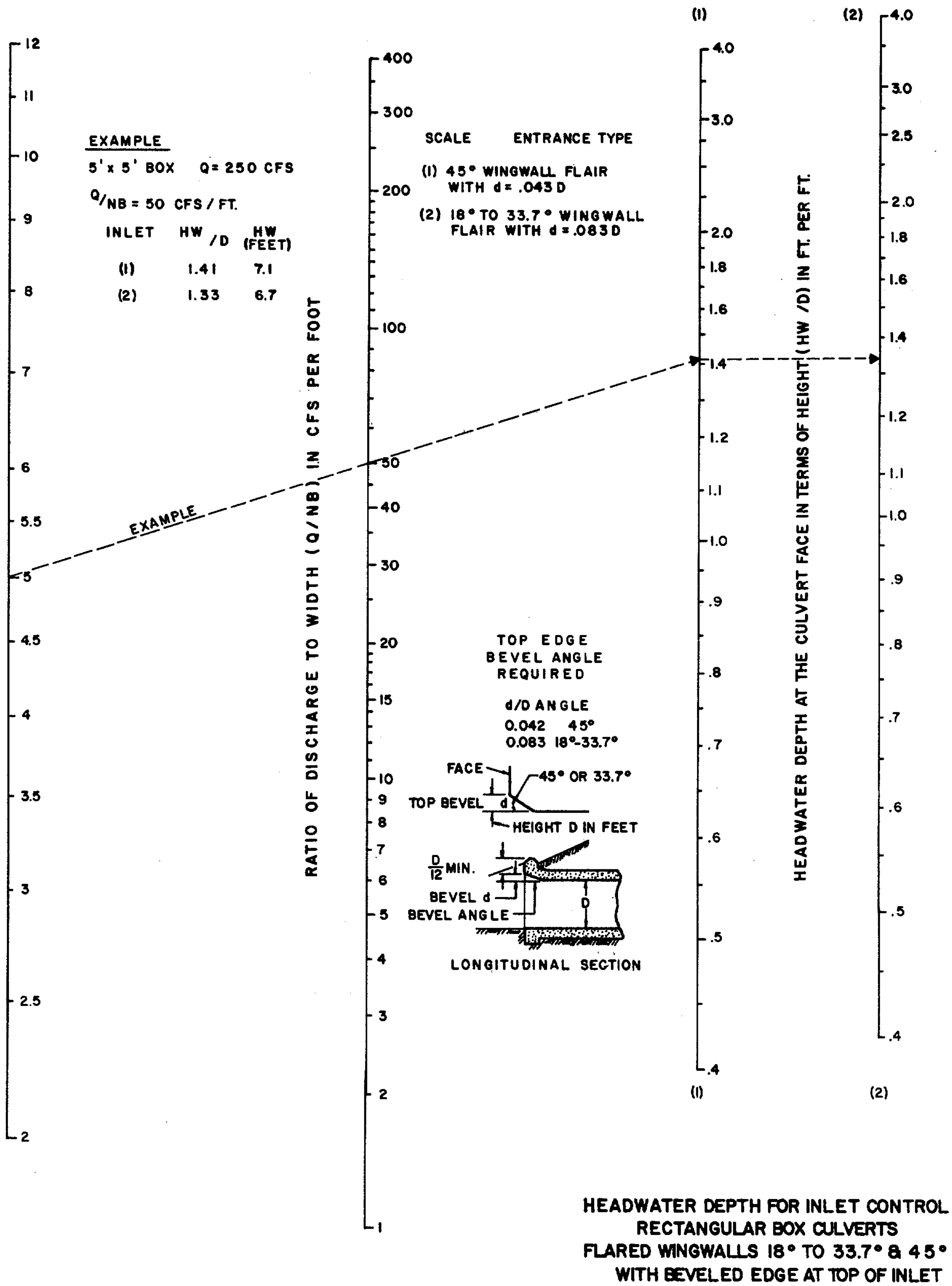


Exhibit F.9 Headwater Depth for Inlet Control Rectangular Box Culverts (Flared Wingwalls 18° to 33.7° & 45° with Beveled Edge at Top of Inlet) (Source: Reference F.1)

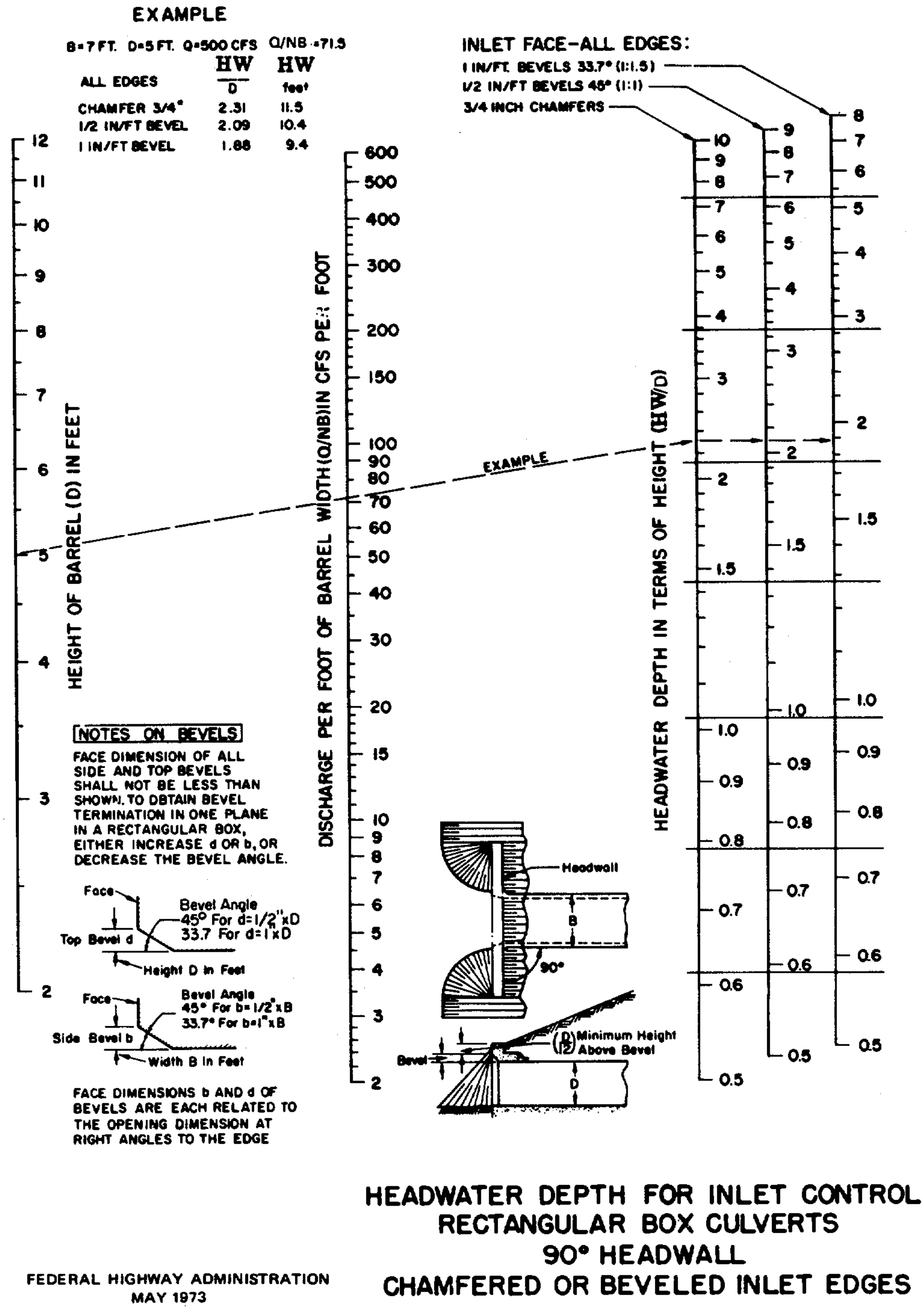


Exhibit F.10 Headwater Depth for Inlet Control Rectangular Box Culverts (90° Headwall – Chamfered or Beveled Inlet Edges) (Source: Reference F.1)

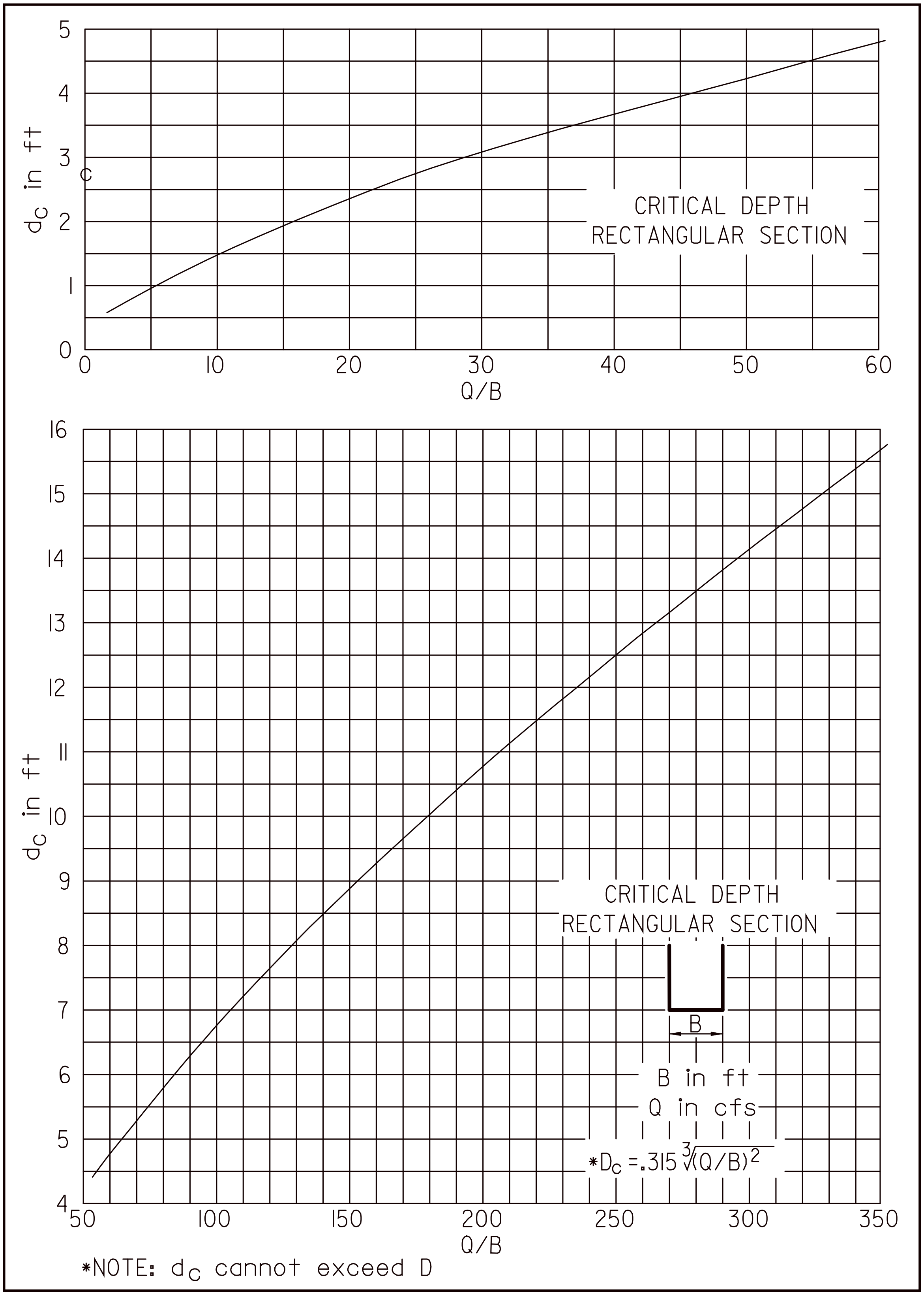


Exhibit F.11 Critical Depth for Box Culvert
(Source: Reference F.1)

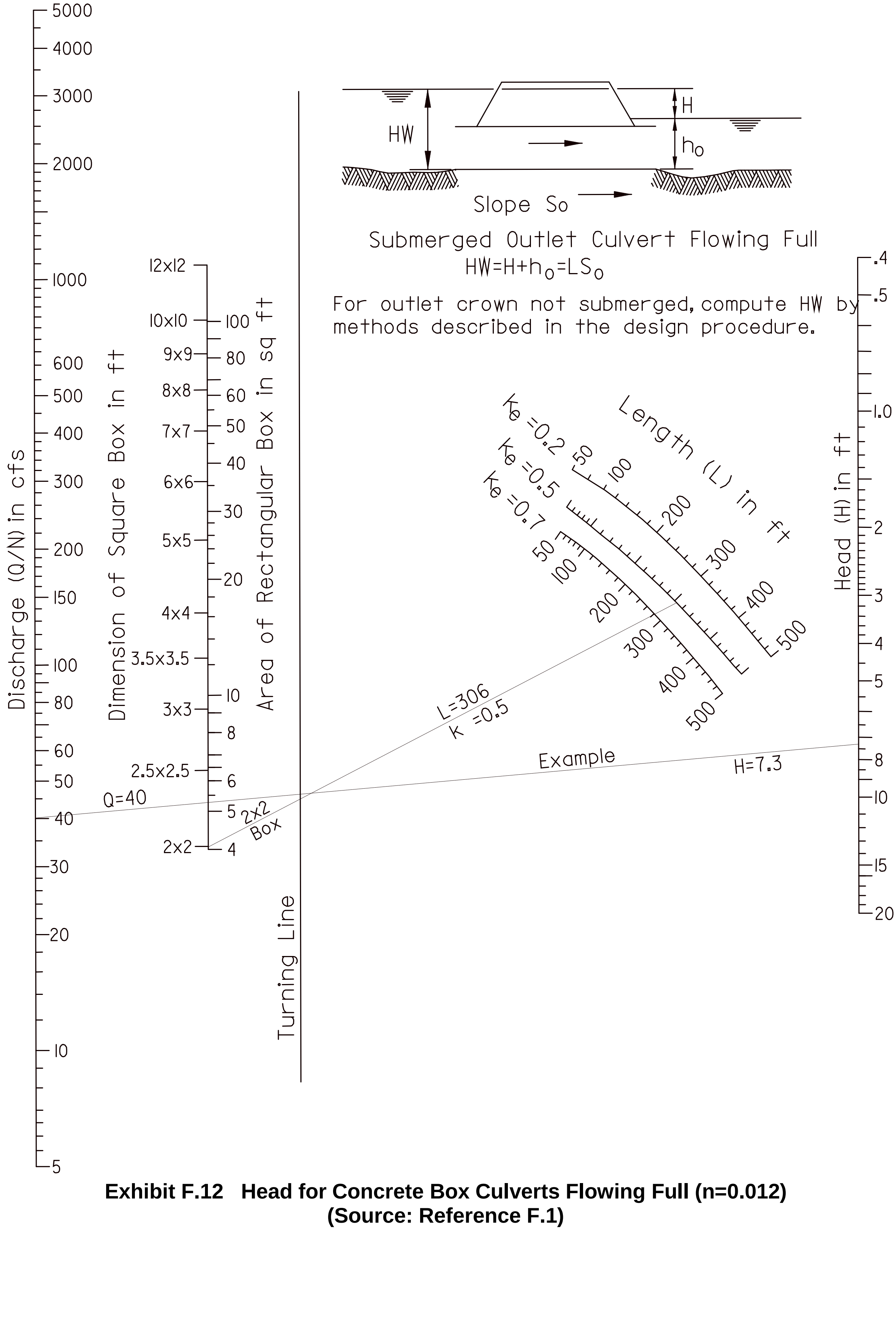


Exhibit F.12 Head for Concrete Box Culverts Flowing Full (n=0.012)
(Source: Reference F.1)

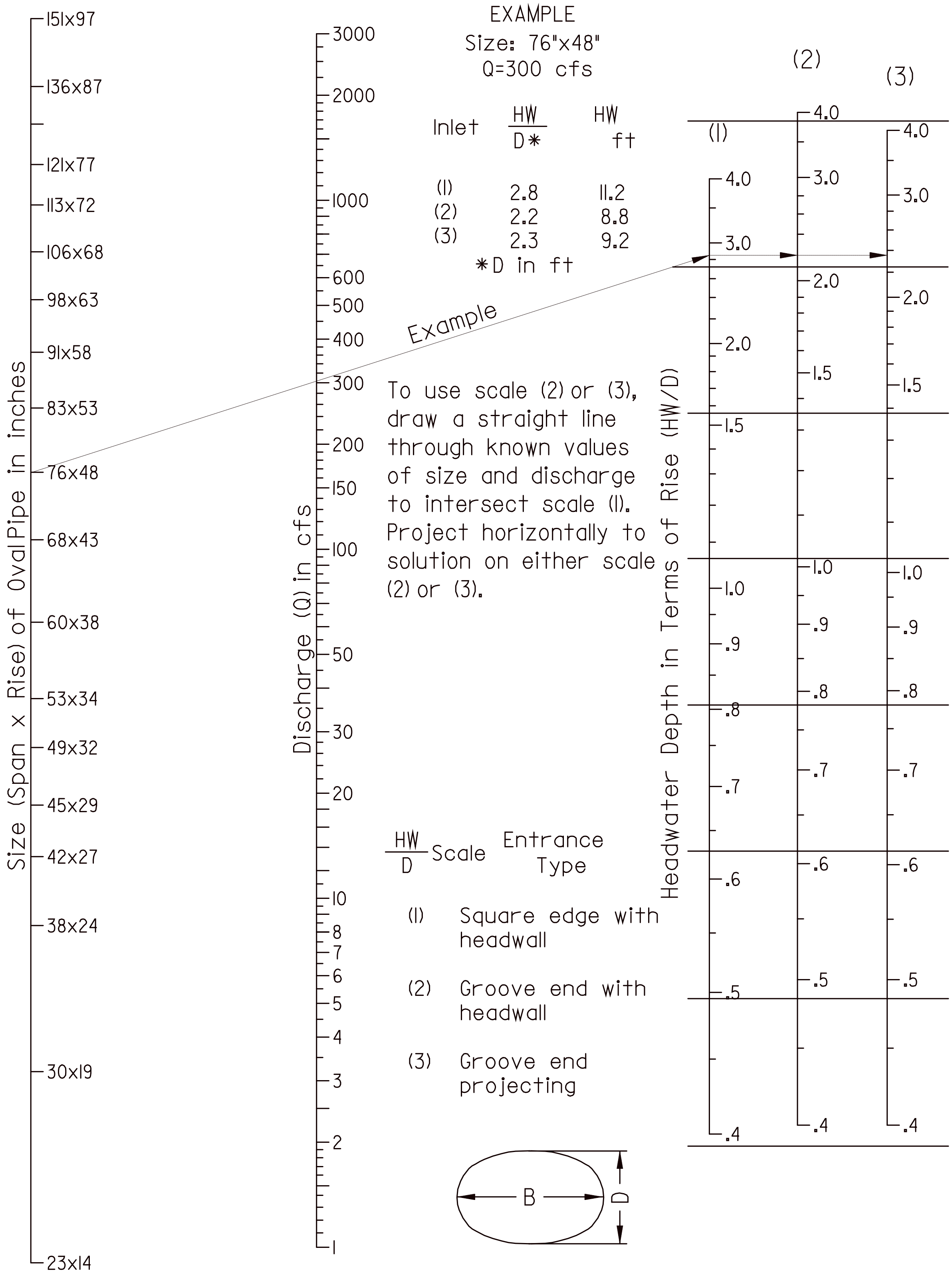


Exhibit F.13 Headwater Depth for Elliptical Concrete Pipe Culverts
Long Axis Horizontal with Inlet Control
(Source: Reference F.1)

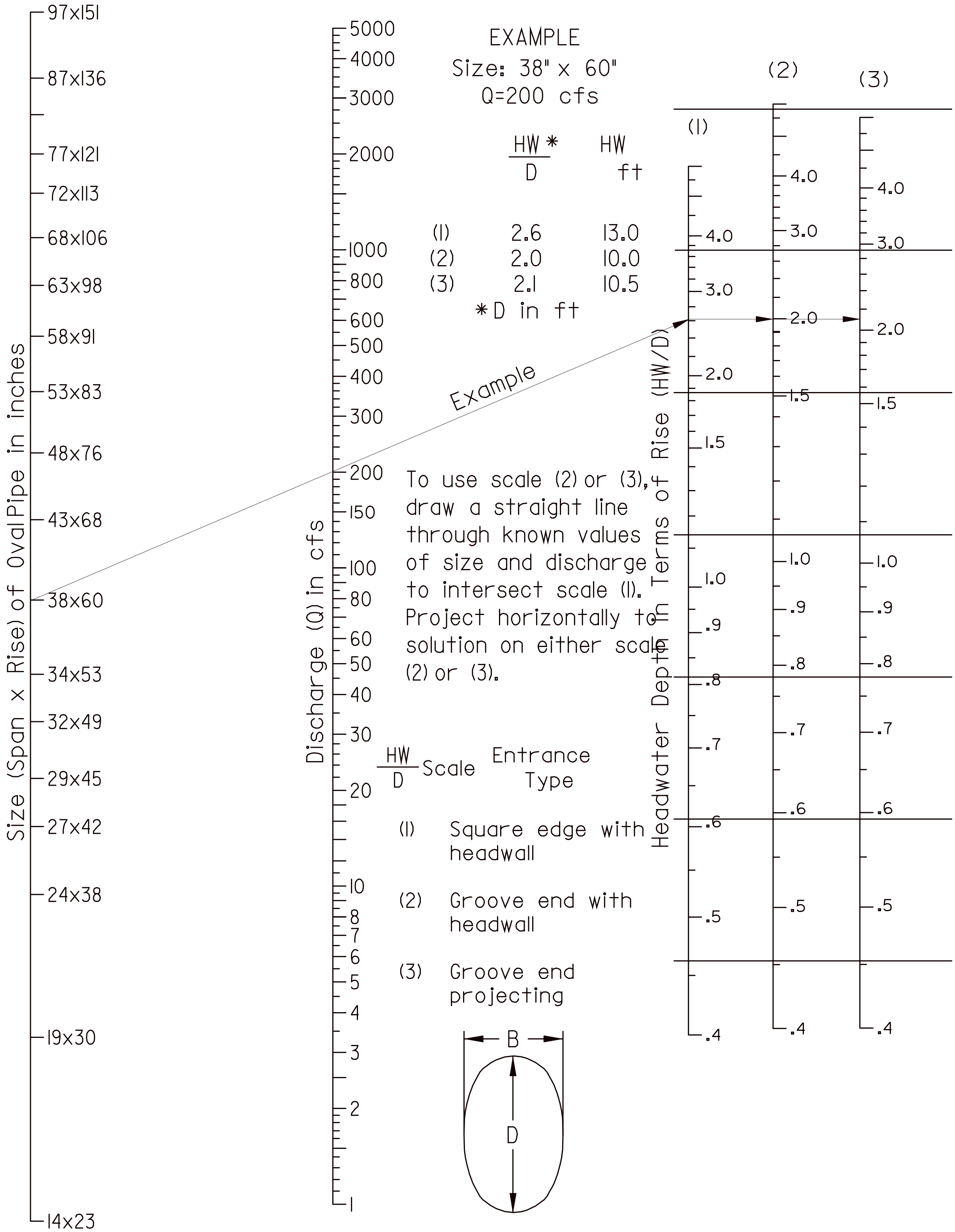


Exhibit F.14 Headwater Depth for Elliptical Concrete Pipe Culverts
Long Axis Vertical with Inlet Control
(Source: Reference F.1)

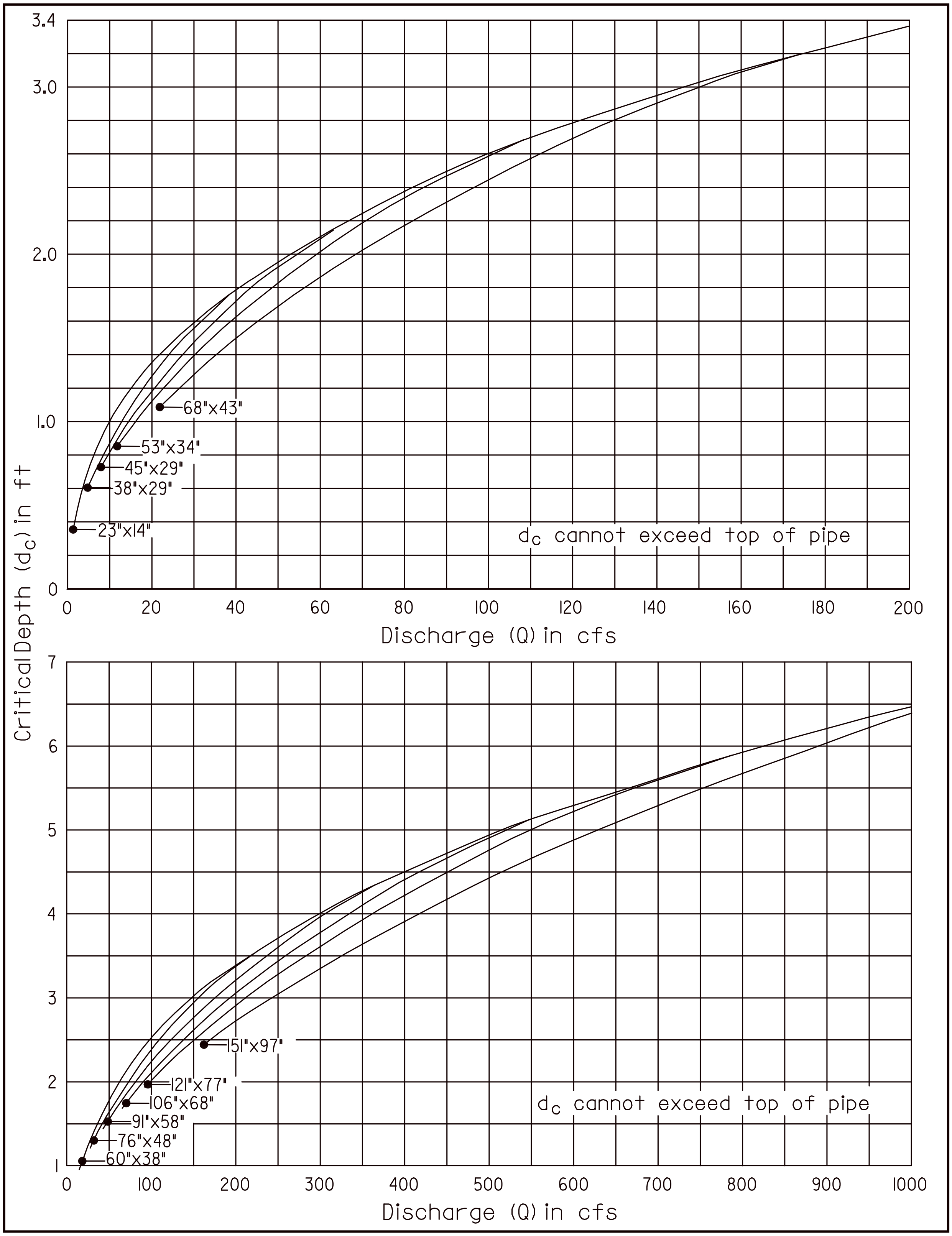


Exhibit F.15 Critical Depth for Elliptical Concrete Pipe
Long Axis Horizontal
(Source: Reference F.1)

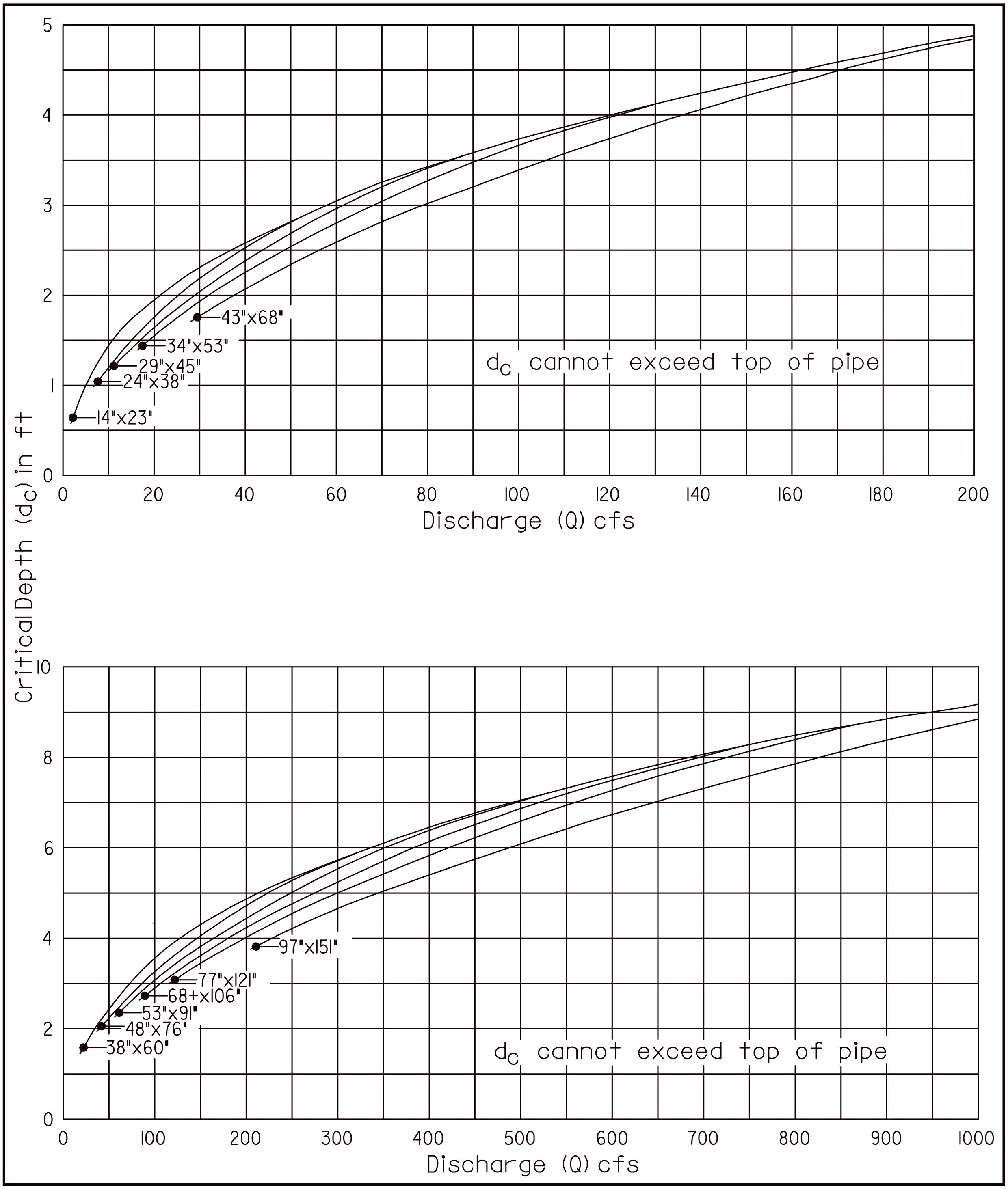


Exhibit F.16 Critical Depth for Elliptical Concrete Pipe
Long Axis Vertical
(Source: Reference F.1)

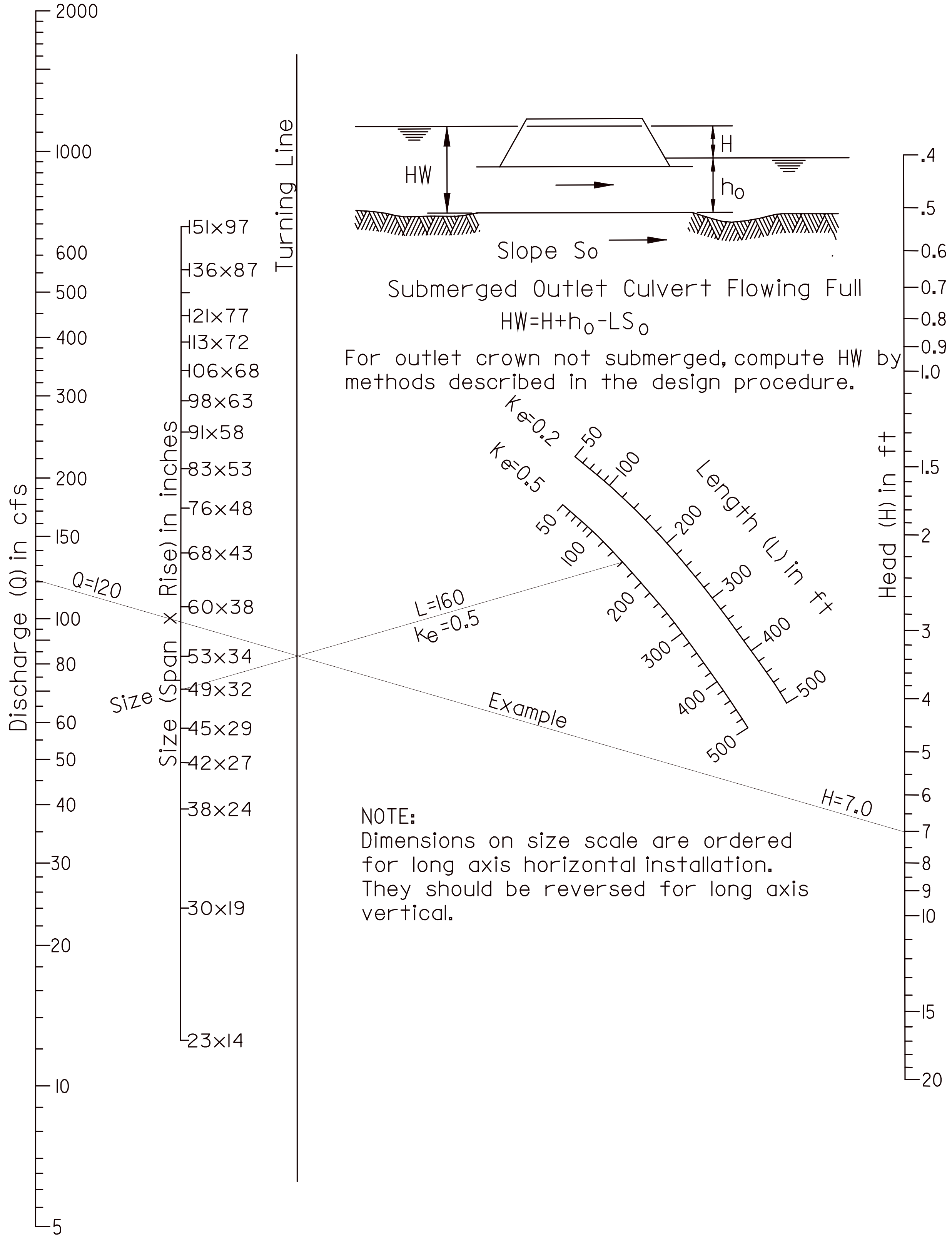


Exhibit F.17 Head for Elliptical Concrete Pipe Culverts Long Axis Horizontal or Vertical Flowing Full (n=0.012)
(Source: Reference F.1)

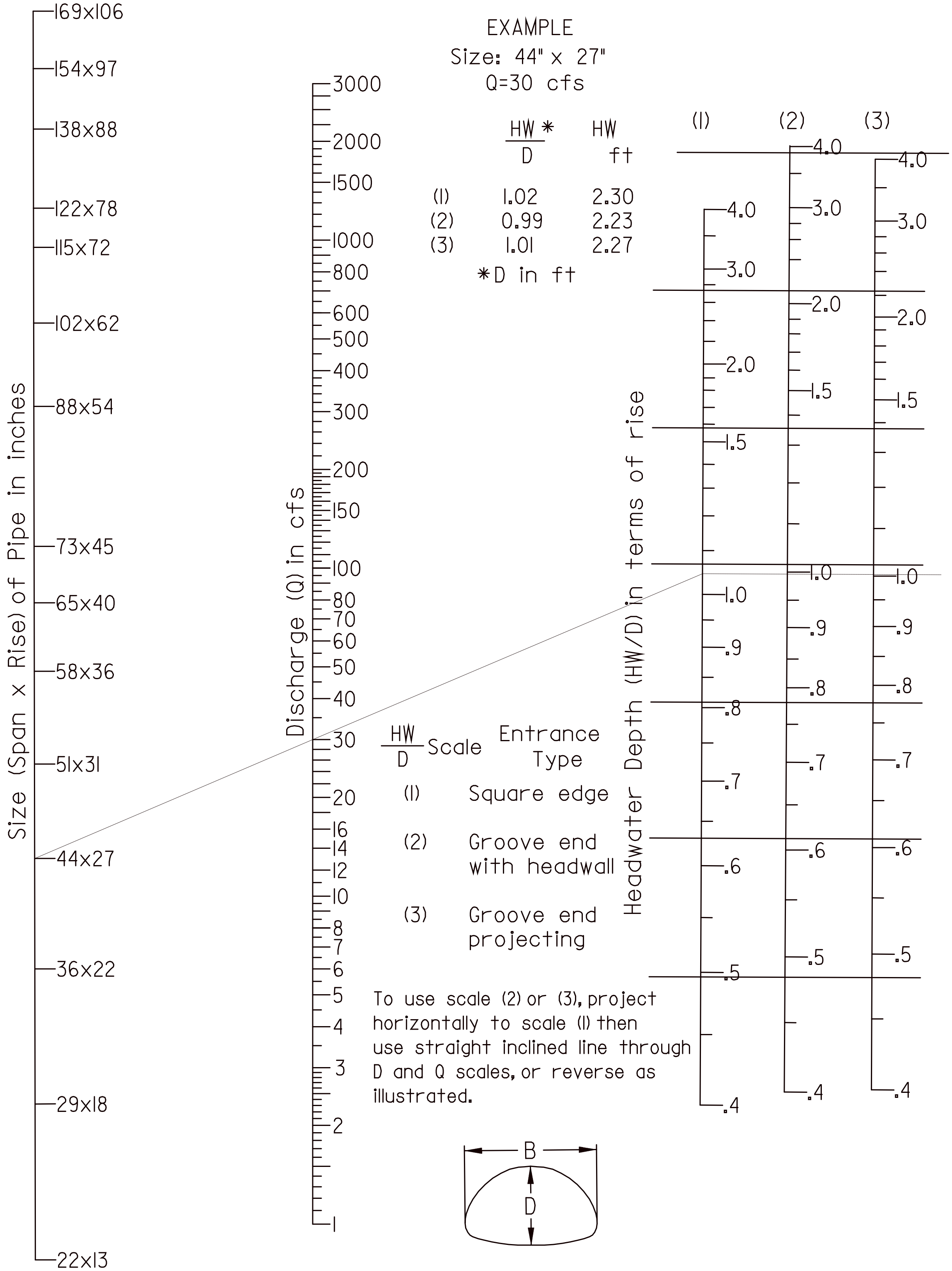


Exhibit F.18 Headwater Depth for Concrete Arch Culverts with Inlet Control
(Source: Reference F.2)

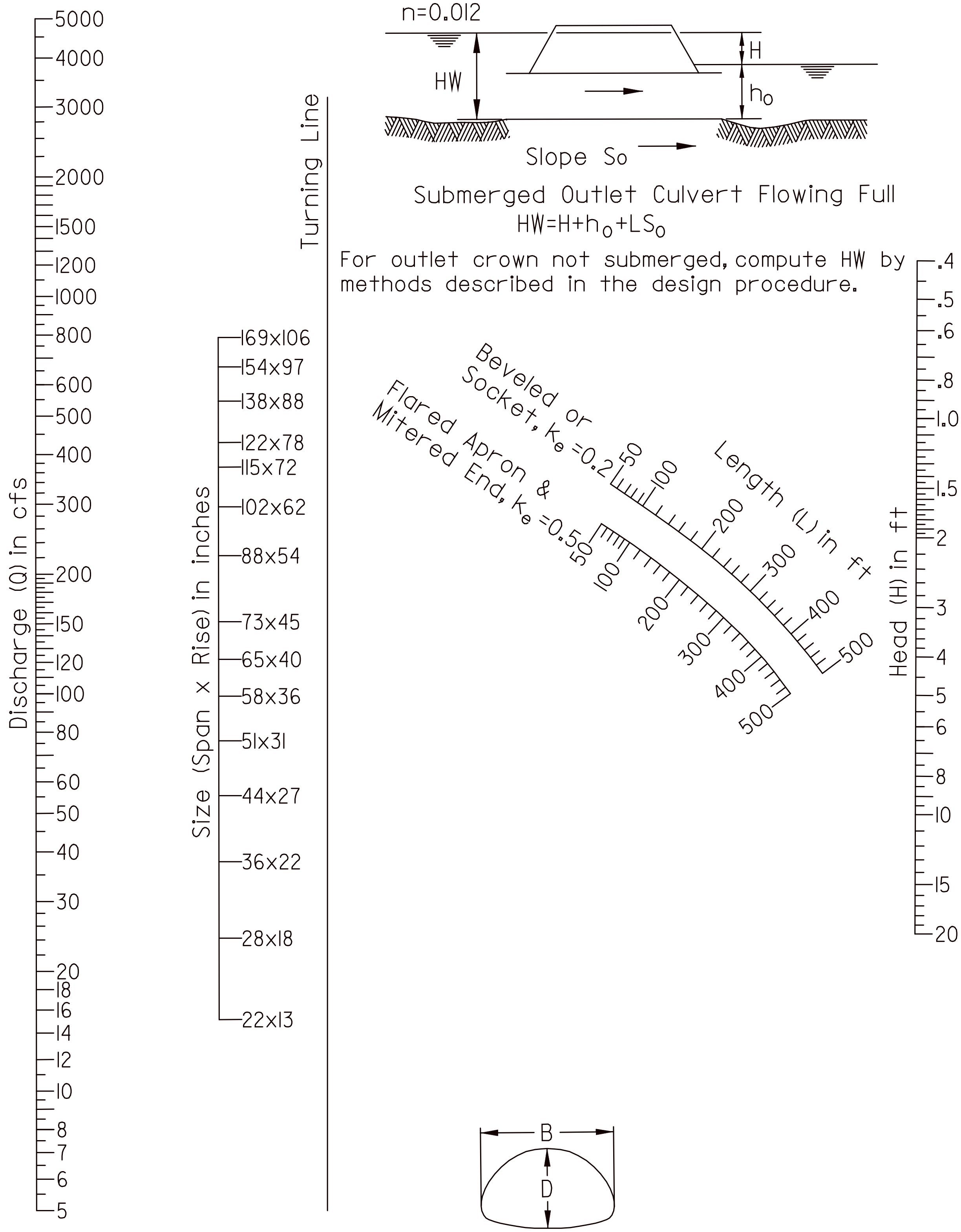


Exhibit F.19 Head for Concrete Arch Culverts Flowing Full
(Source: Reference F.2)

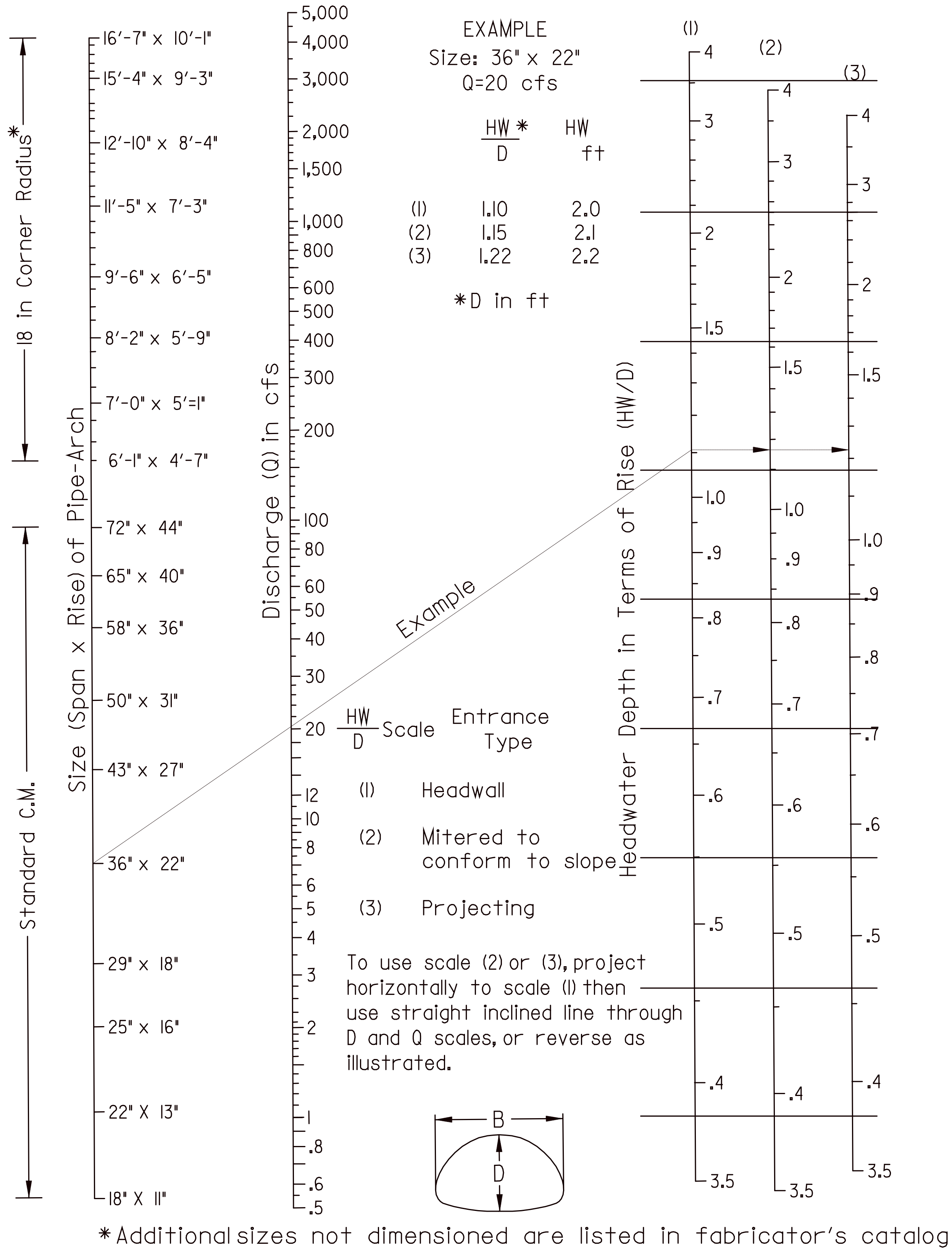


Exhibit F.20 Headwater Depth for CMP-Arch Culverts with Inlet Control
(Source: Reference F.1)

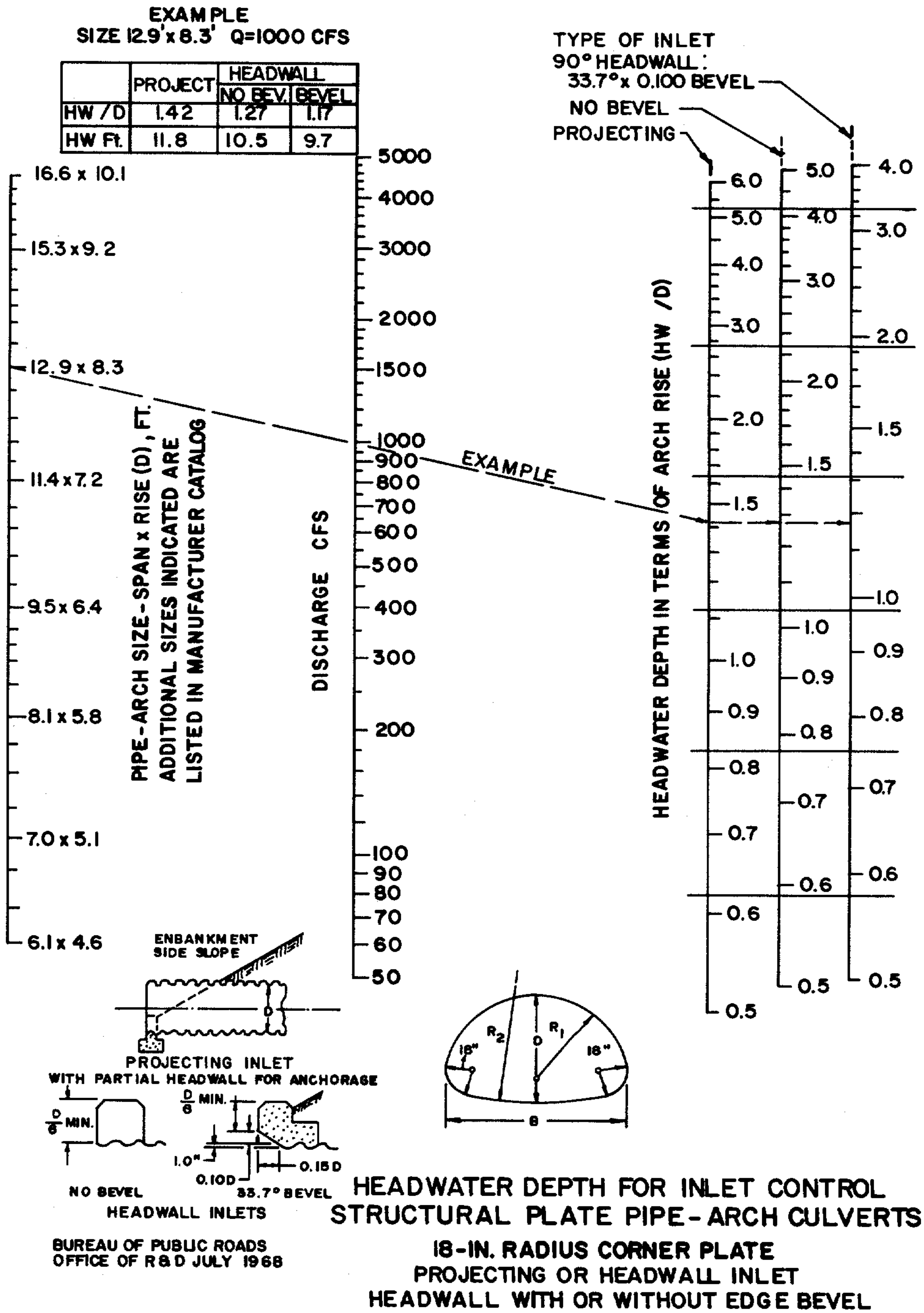


Exhibit F.21 Headwater Depth for Inlet Control Structural Plate Pipe-Arch Culverts With 18 in. Radius Corner Plate (Source: Reference F.1)

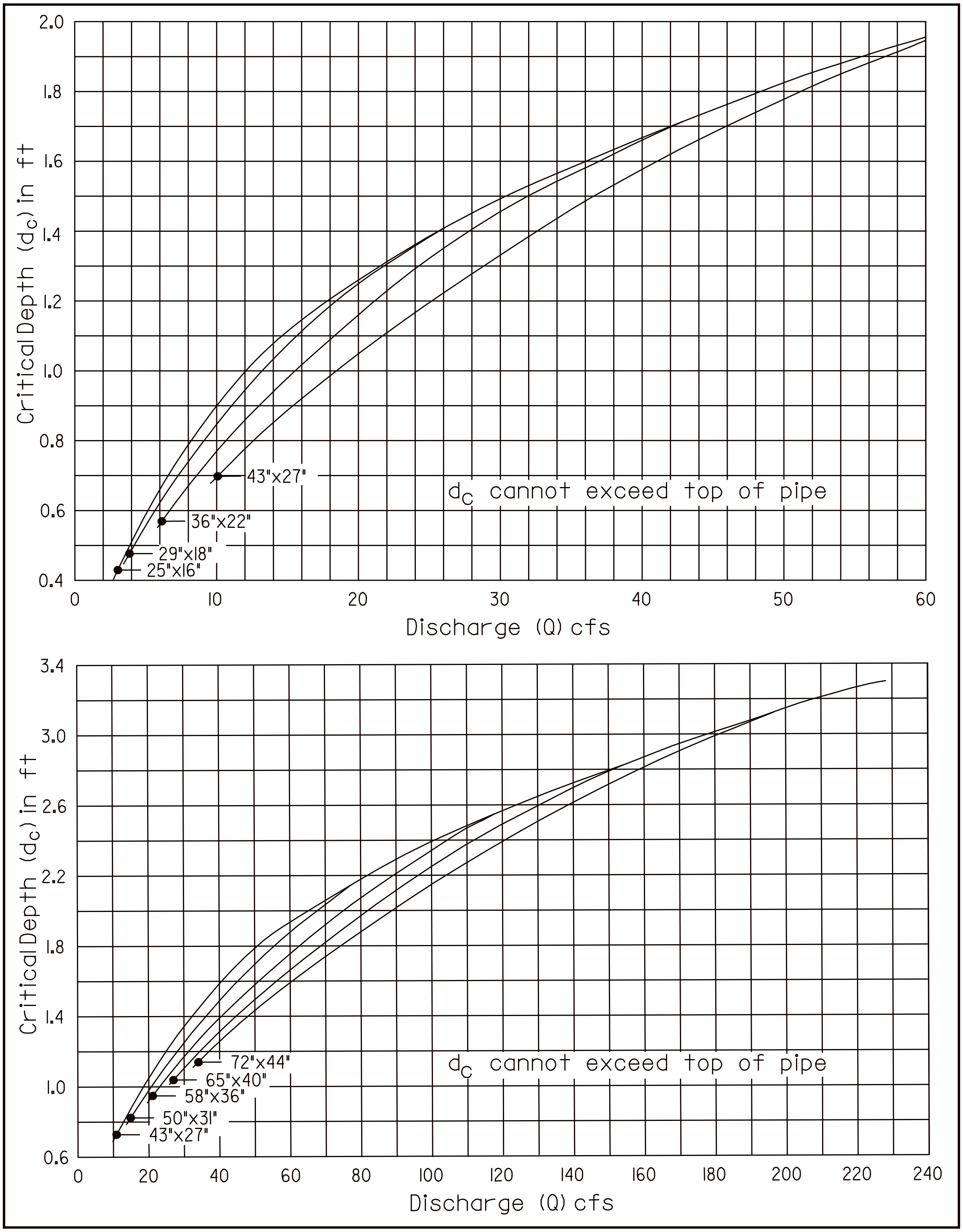


Exhibit F.22 Critical Depth for Standard CMP-Arch Culverts
(Source: Reference F.1)

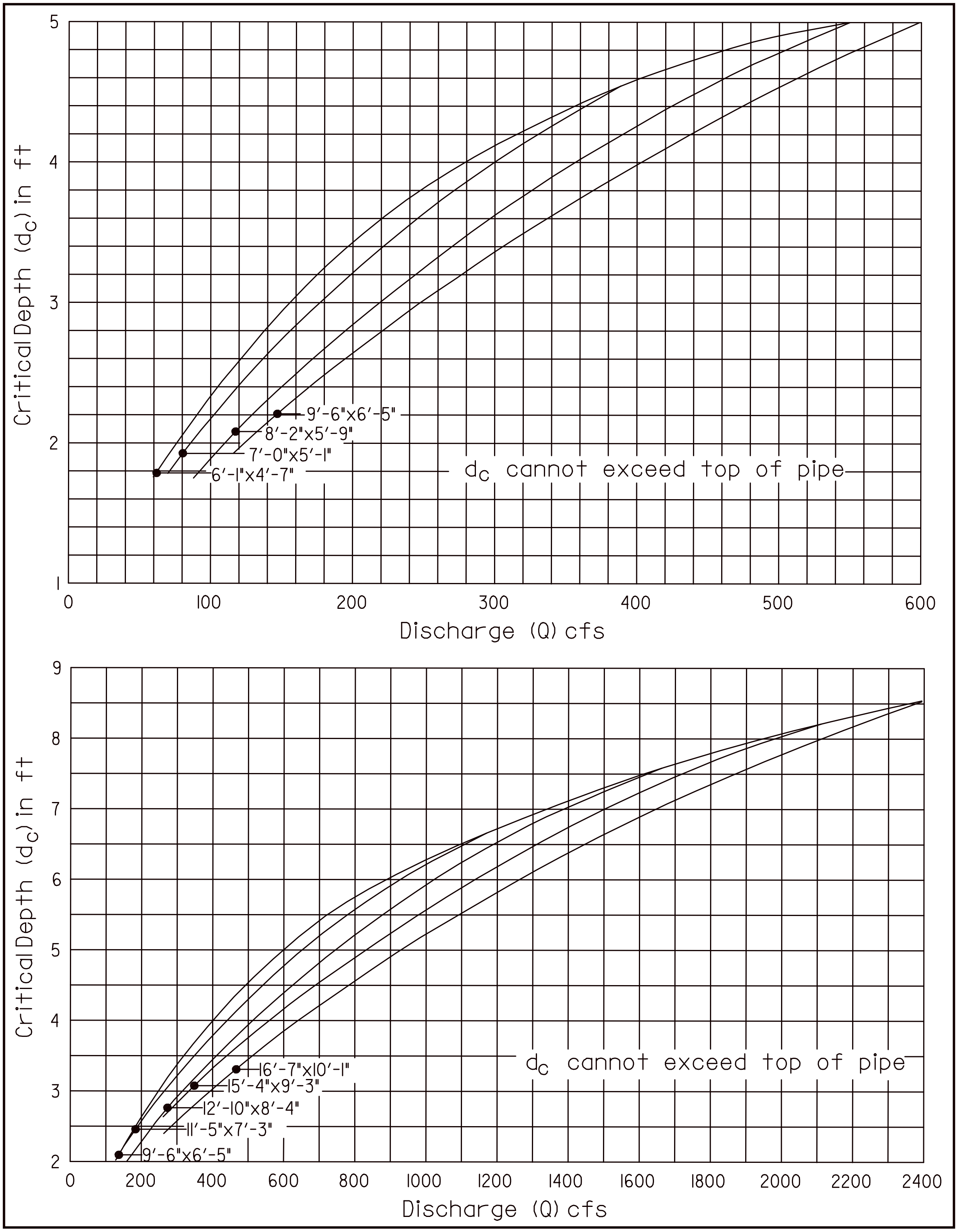


Exhibit F.23 Critical Depth for Structural Plate CMP-Arch Culverts with 18 in. Corner Radius Plate (Source: Reference F.1)

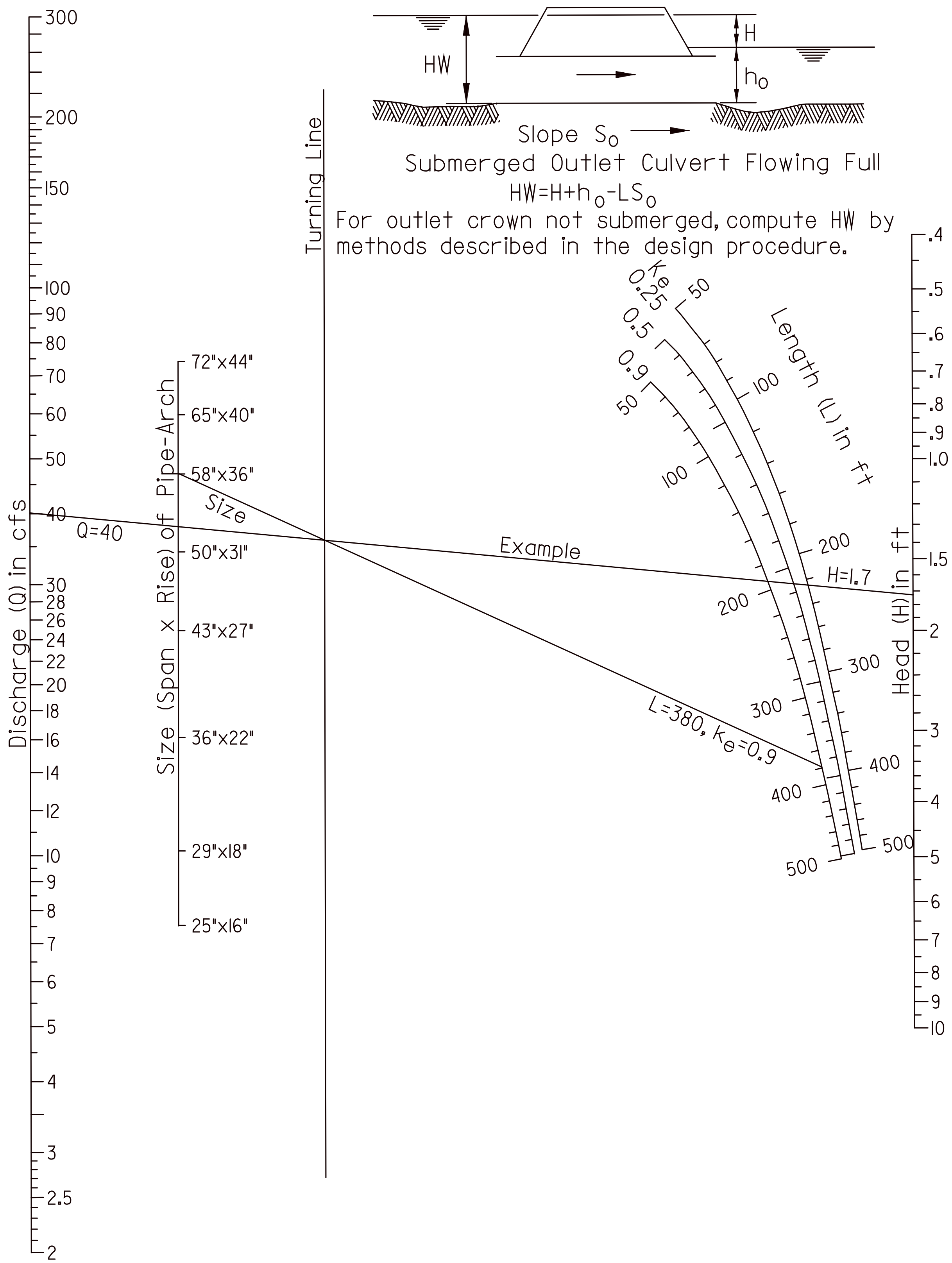


Exhibit F.24 Head for Standard CMP-Arch Culverts Flowing Full (n=0.024)
(Source: Reference F.1)

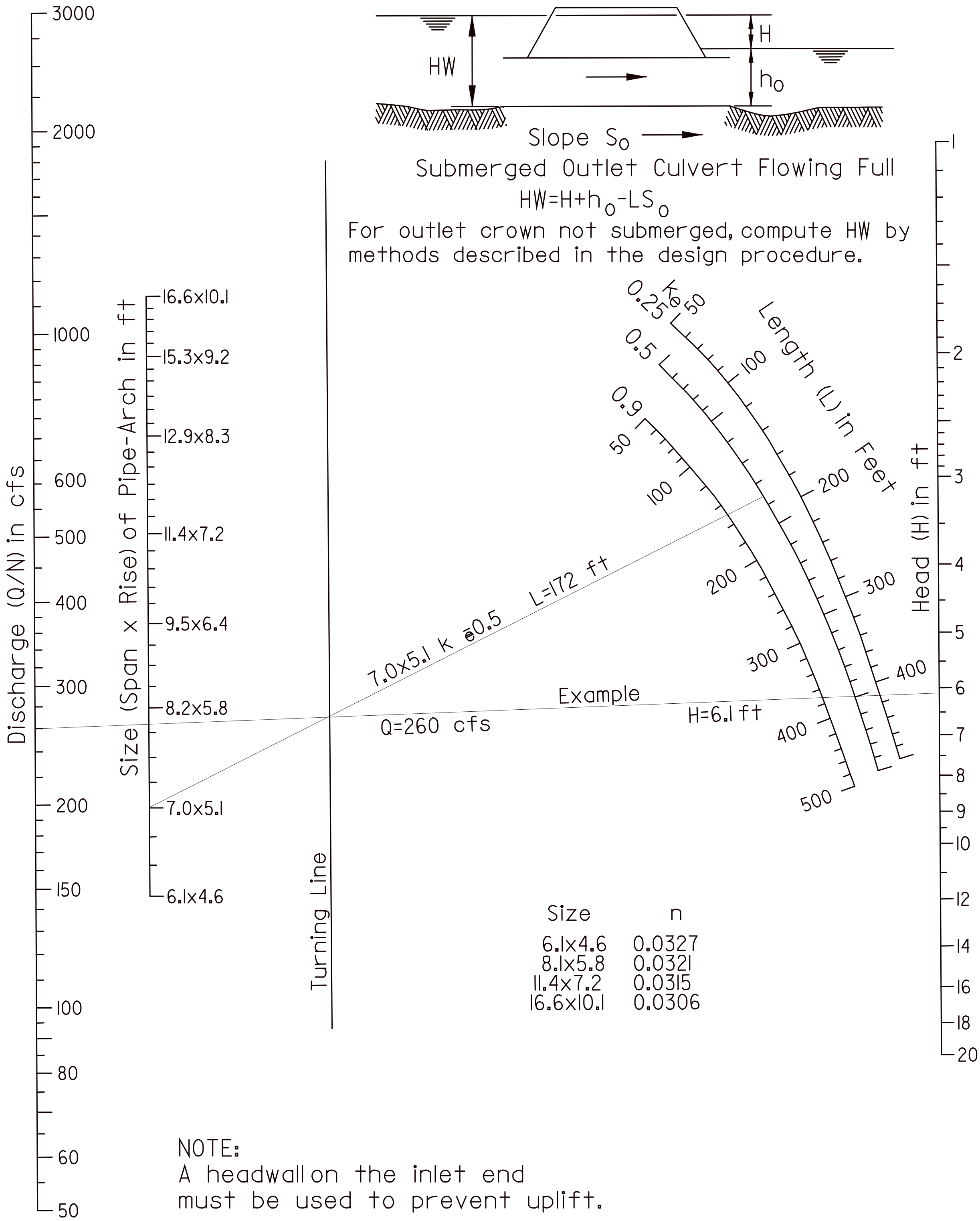


**Exhibit F.25 Head for Structural Plate CMP-Arch Culverts with 18 in. Corner Radius Plate Flowing Full (n=0.0327 to 0.0306)
(Source: Reference F.1)**

REFERENCES

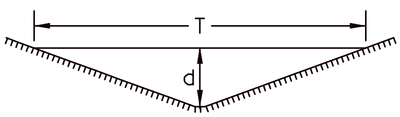
F.1 U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September, 1985. https://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7&id=13

F.2 American Concrete Pipe Association. (http://www.concrete-pipe.org/index.php?cp_Session=805edca166f308d21f57c53735e572af)

APPENDIX G
NOMOGRAPHS AND CHARTS FOR GUTTER FLOW & INLET DESIGN

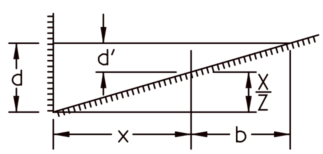
Instructions for Use

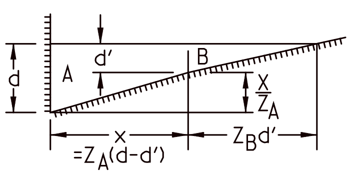
1. Connect $\frac{Z}{n}$ ratio with slope, s. Connect discharge Q with point where line crosses turning line. Read depth at curb d. Q can be found from d by connecting d with crossing of turning line.

2. 

For shallow V-shaped channel, use instruction 1, but with $Z = \frac{T}{d}$

3. To determine discharge Q_X in portion of channel having width x, determine depth for the entire section as in instruction 1. Then use nomograph to determine Q in section of width b for depth, $d' = d - \frac{x}{Z}$. Then, $Q_X = Q - Q_b$.



4. 

To determine discharge Q_T in composite section, follow instruction 3 to obtain discharge Q_A in section A at assumed depth d based on an extension of slope ratio Z_A to intersect water surface. Obtain Q_B for slope ratio Z_B and depth d' where $d' = d - \frac{x}{Z_A}$. Then $Q_T = Q_A + Q_B$.

Exhibit G.1 Use of Nomograph for Flow in Triangular Channels

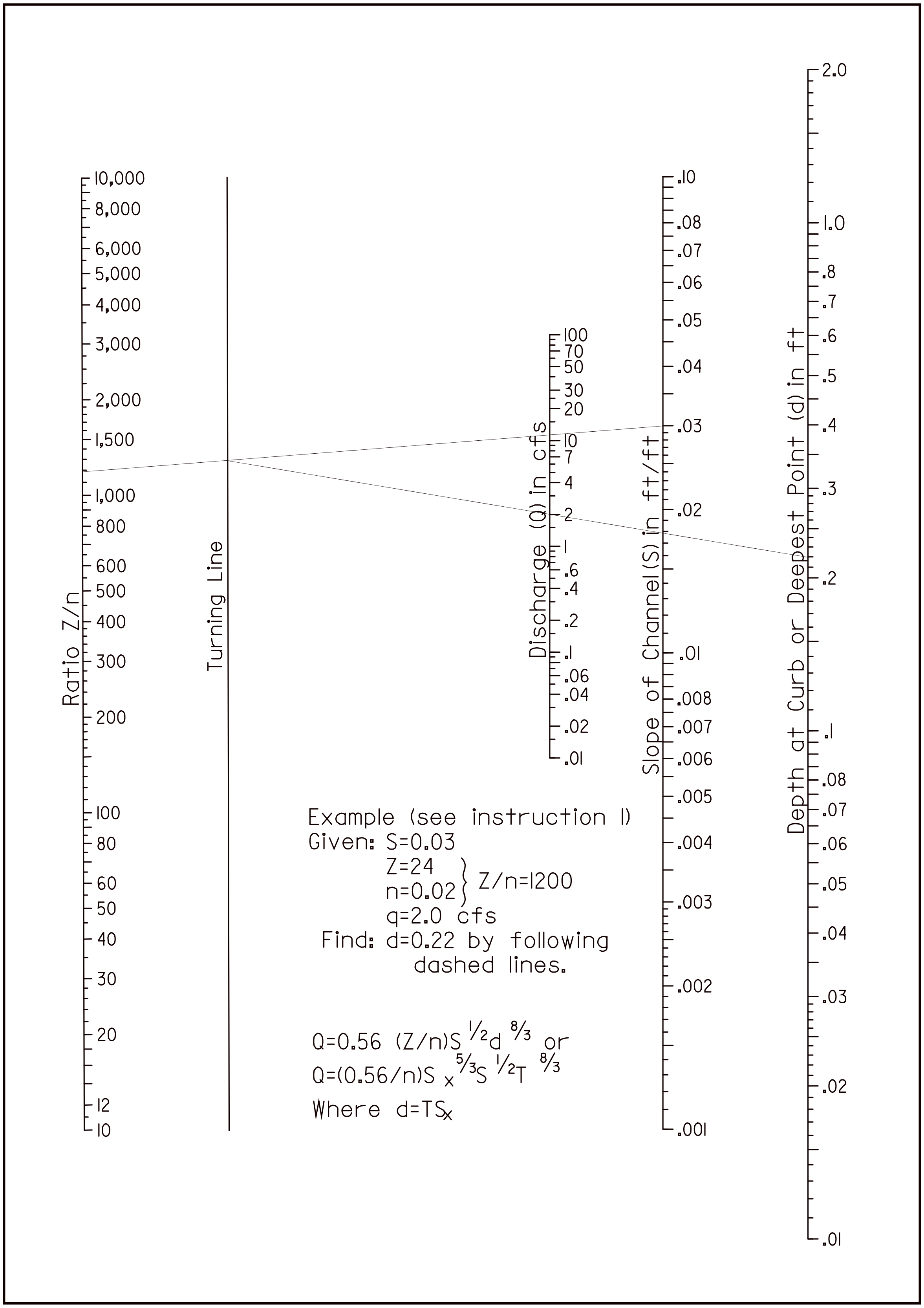


Exhibit G.2 Nomograph for Flow, Q, in Triangular Channels

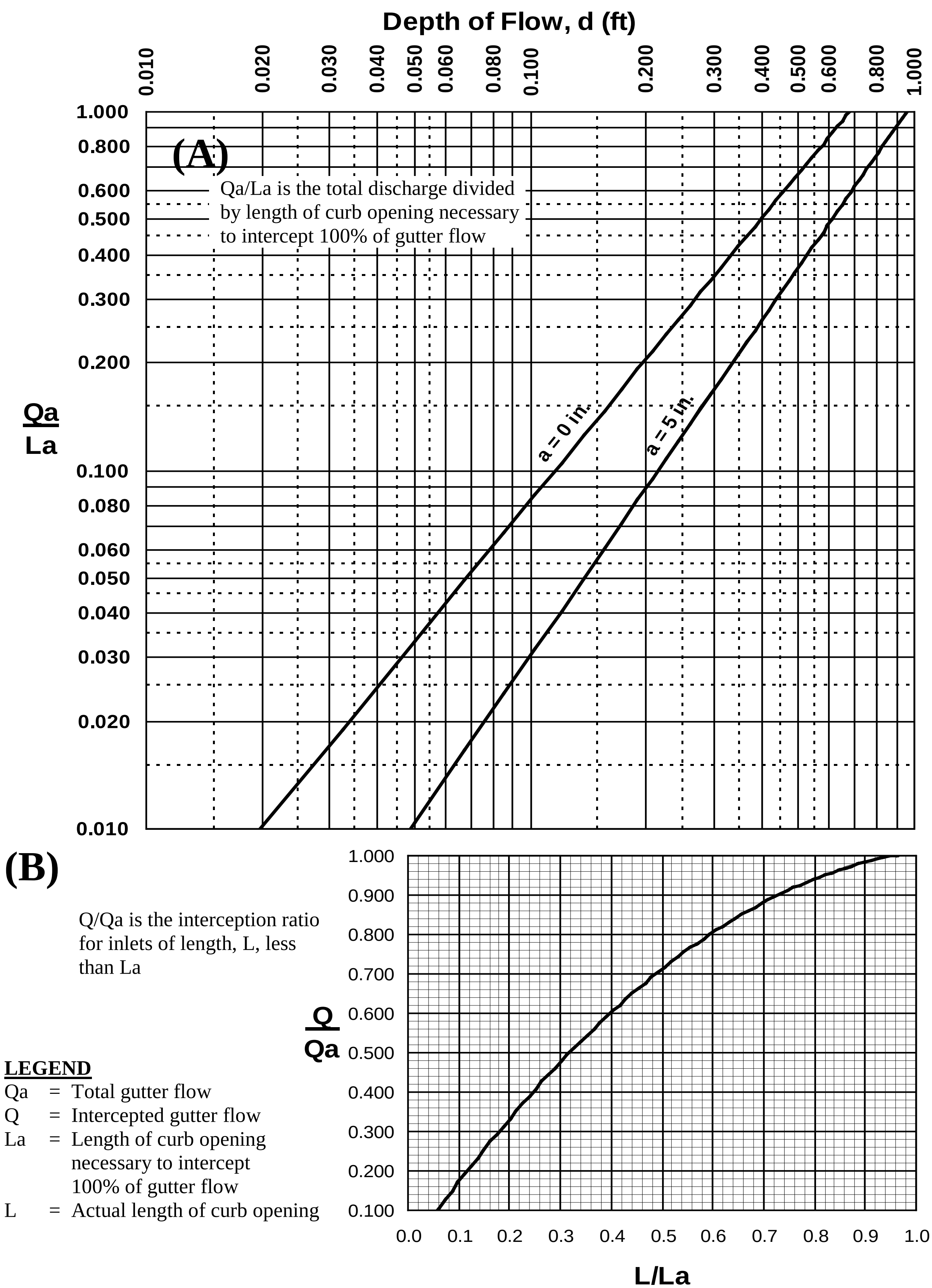


Exhibit G.3 Capacity Nomograph for Curb Opening Inlets on Continuous Grade

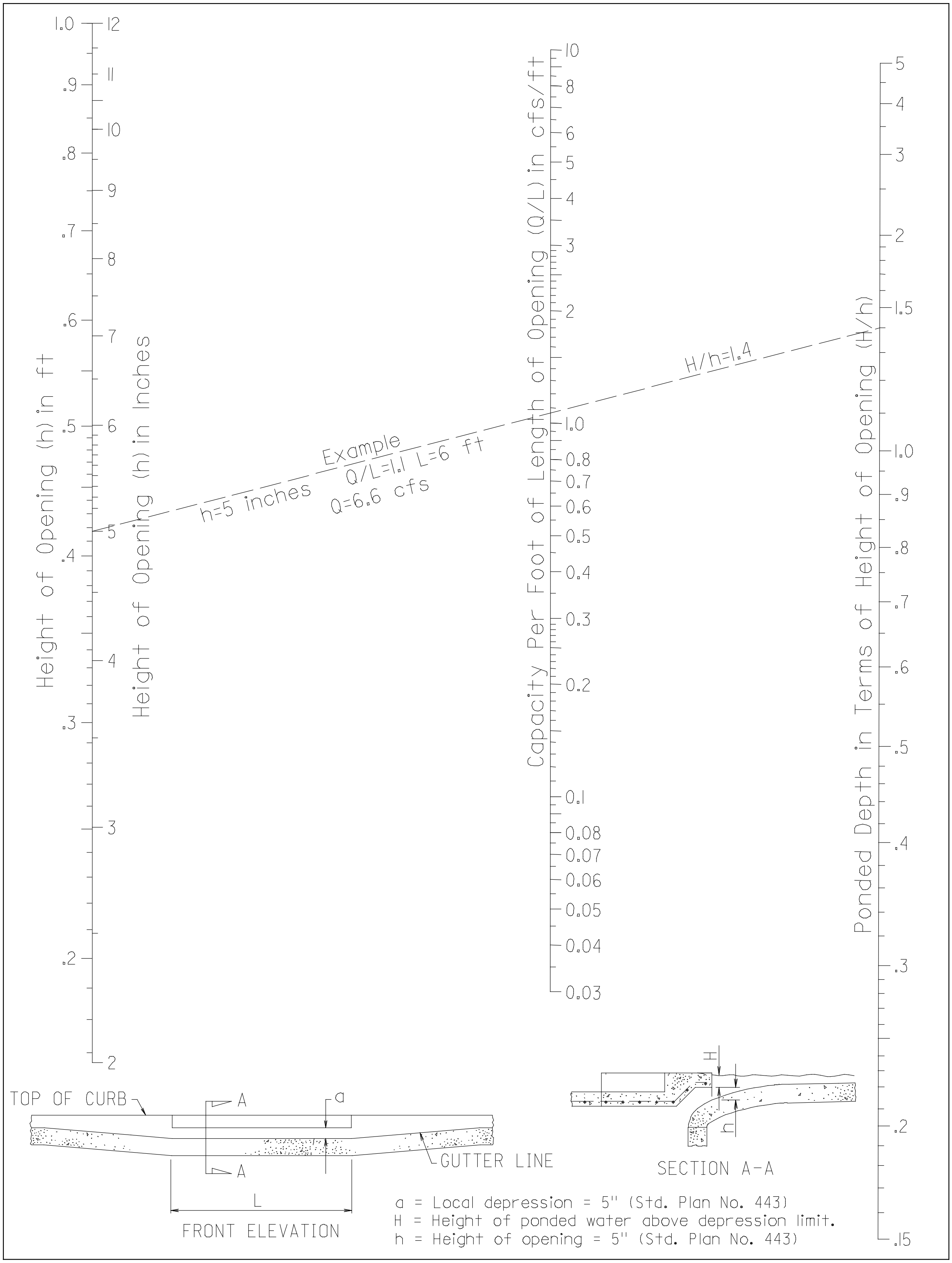


Exhibit G.4 Capacity Nomograph for Curb Opening Inlets in a Low Point or Sump

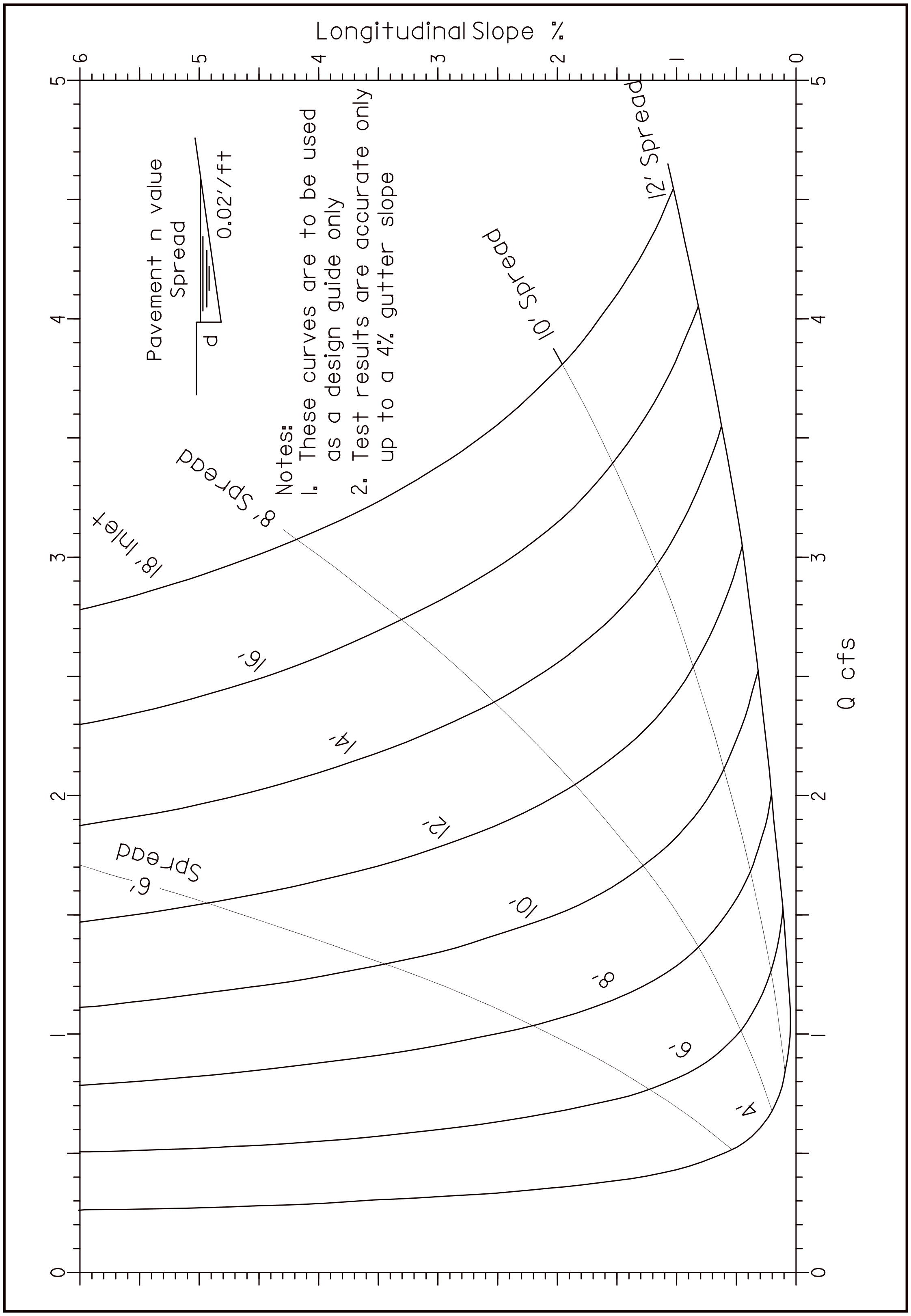


Exhibit G.5 Performance Curves for Curb Inlets Standard Plan
(For a cross-slope of 0.02 ft/ft)

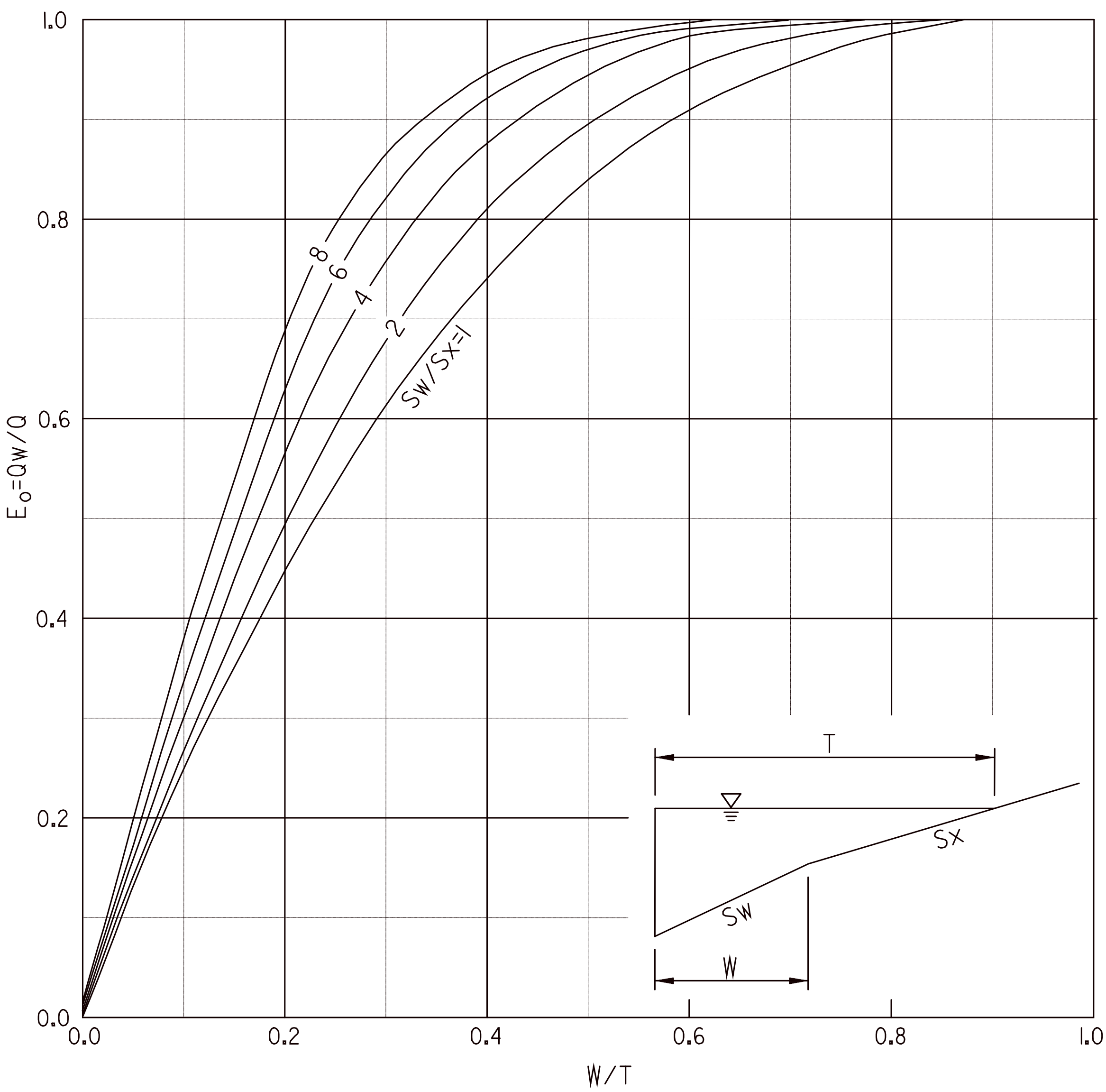


Exhibit G.6 Ratio of Frontal Flow to Total Gutter Flow
(Source: Reference G.1)

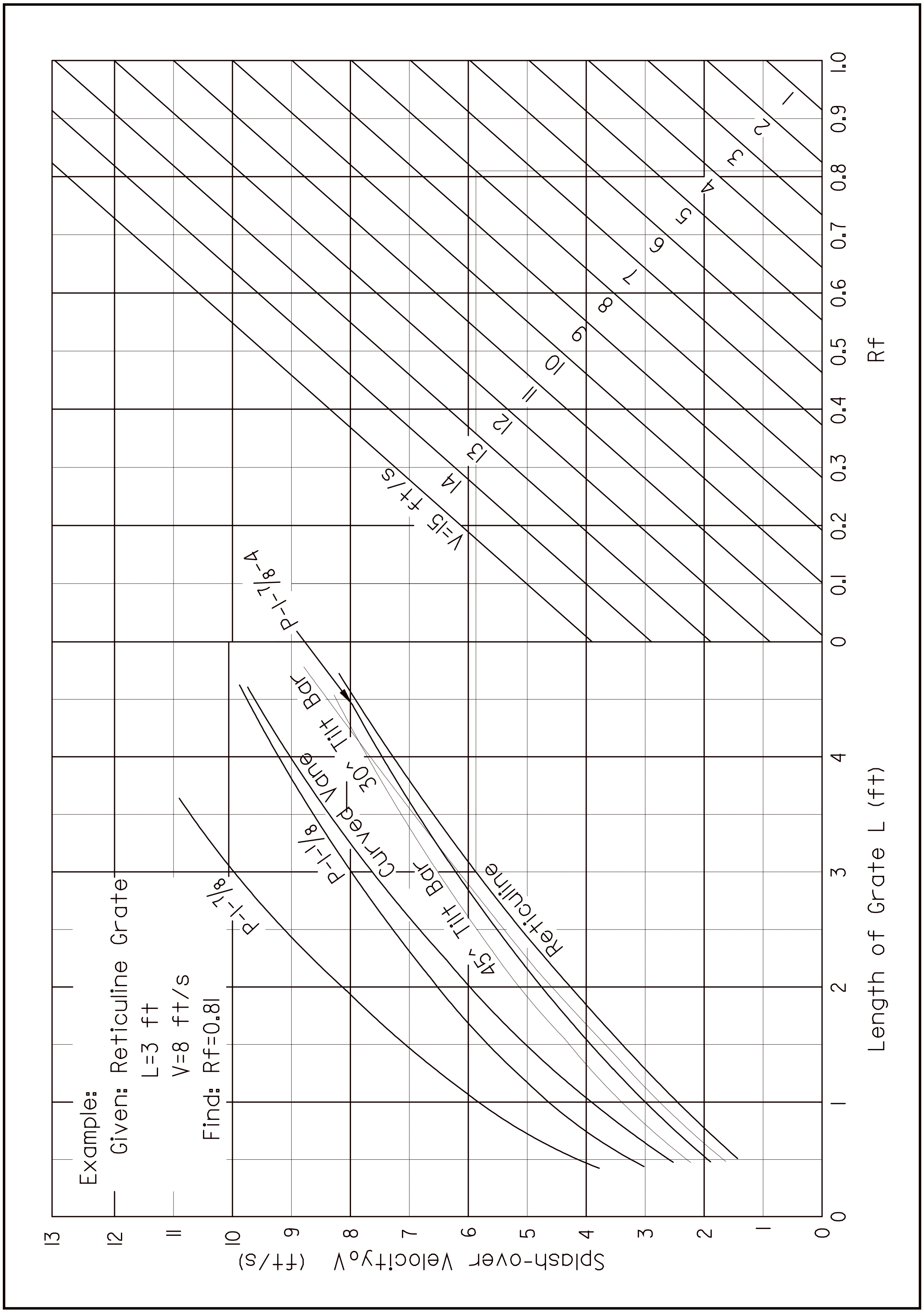


Exhibit G.7 Grate Inlet Frontal Flow Interception Efficiency
(Source: Reference G.1)

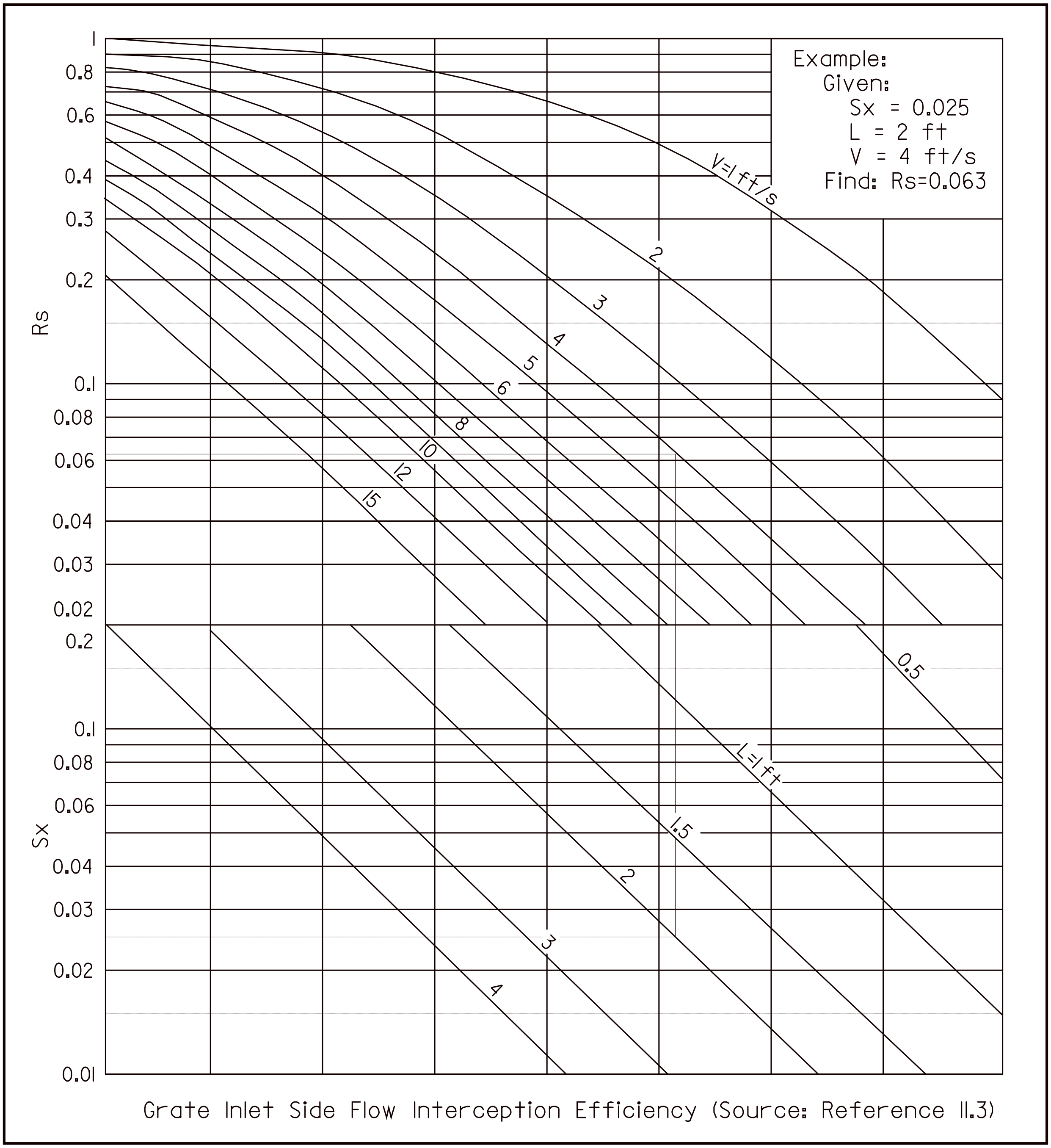


Exhibit G.8 Grate Inlet Side Flow Interception Efficiency
(Source: Reference G.1)

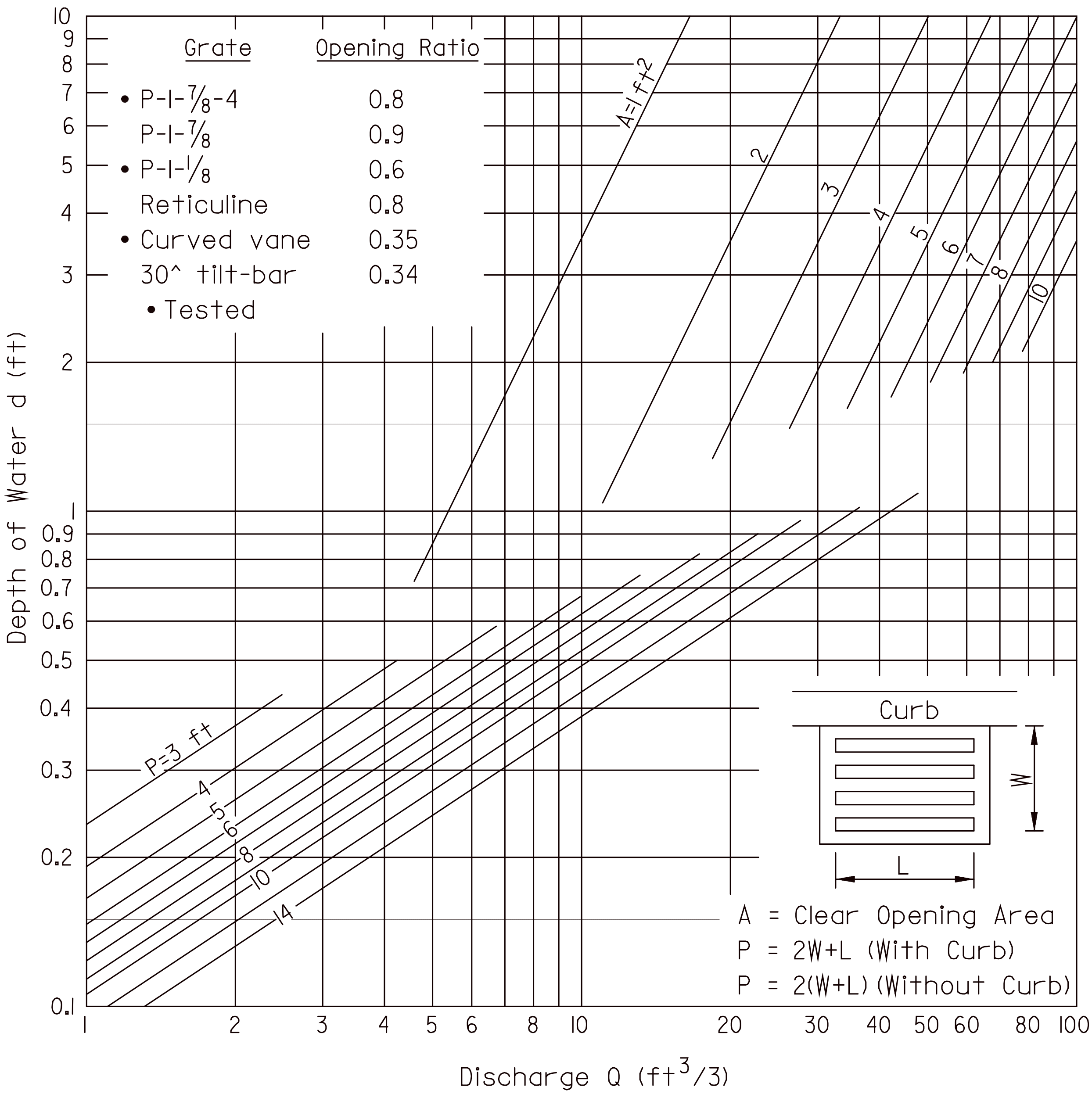


Exhibit G.9 Grate Inlet Capacity in Sump Conditions
(Source: Reference G.1)

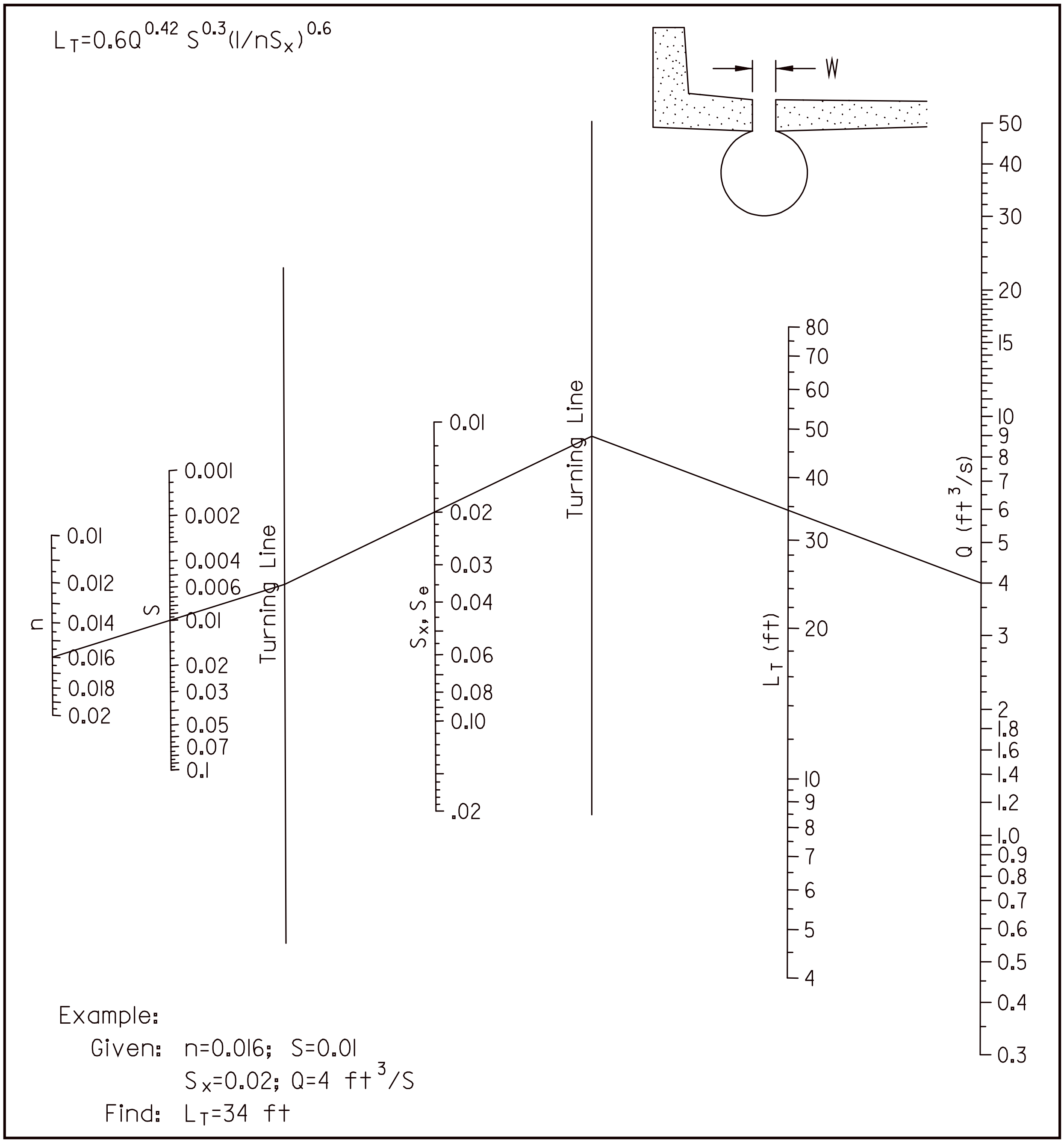


Exhibit G.10 Slotted Inlet Length for Total Interception
(Source: Reference G.1)

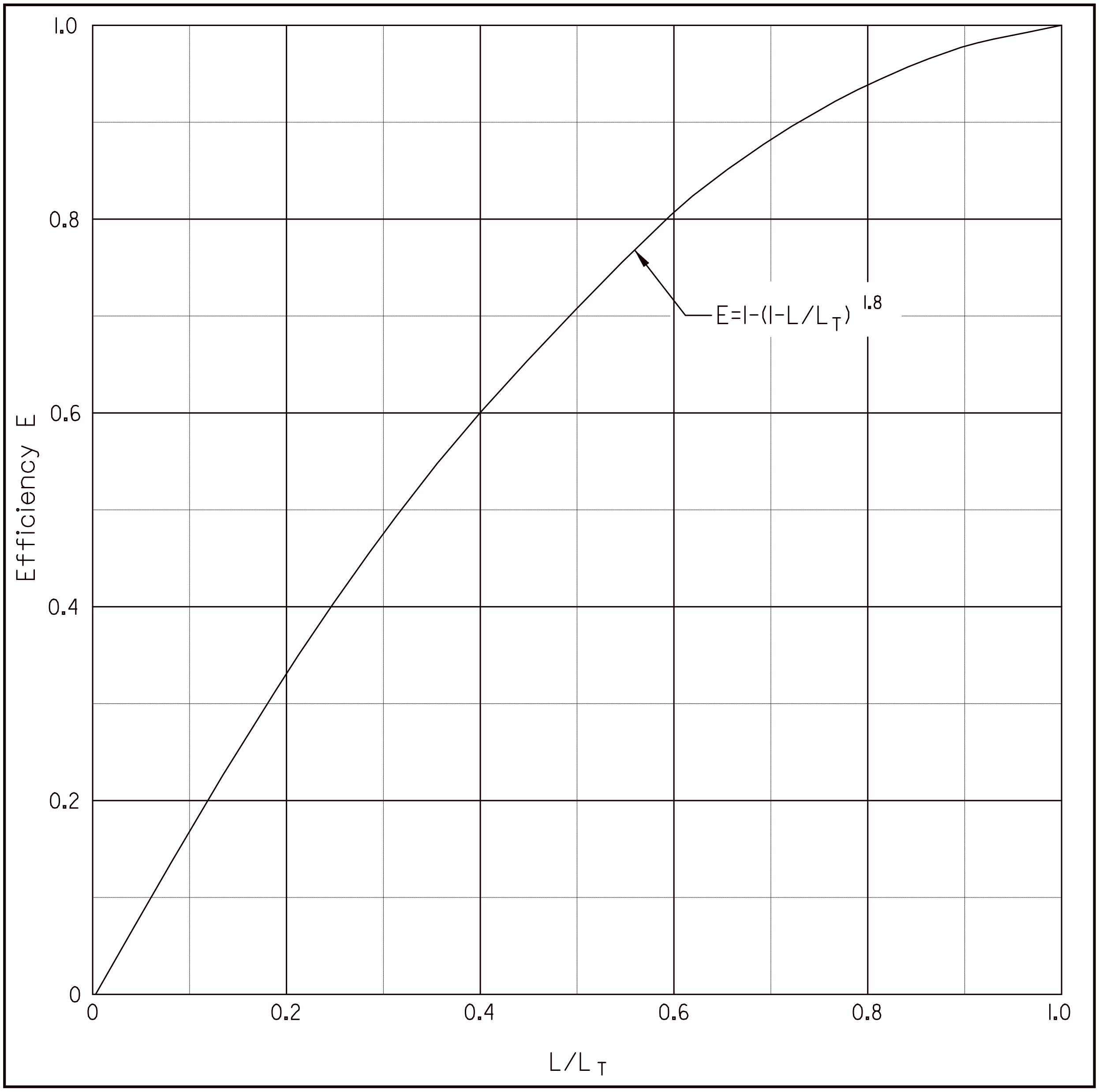


Exhibit G.11 Slotted Inlet Interception Efficiency
(Source: Reference G.1)

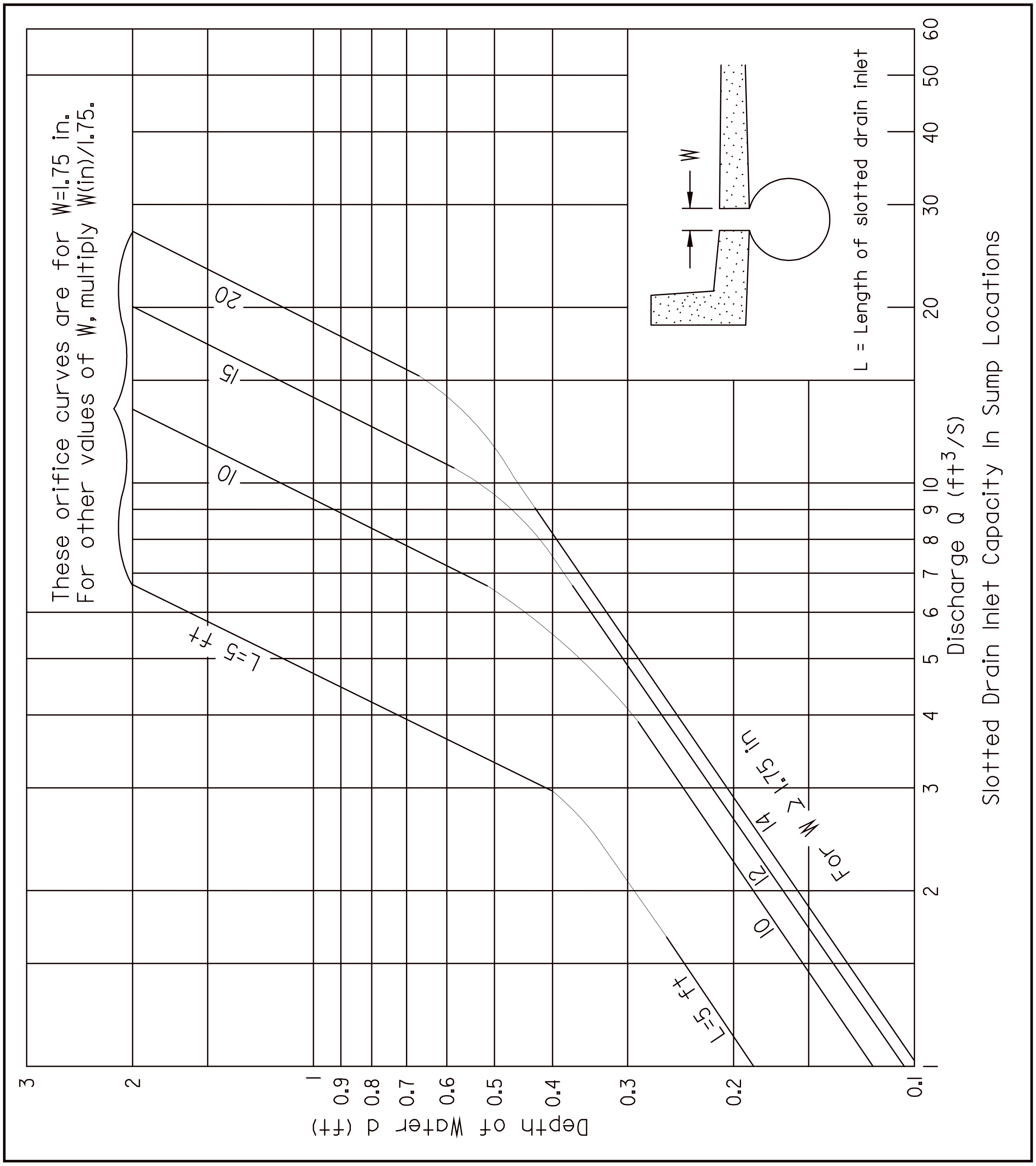


Exhibit G.12 Slotted Drain Inlet Capacity in Sump Locations
(Source: Reference G.1)

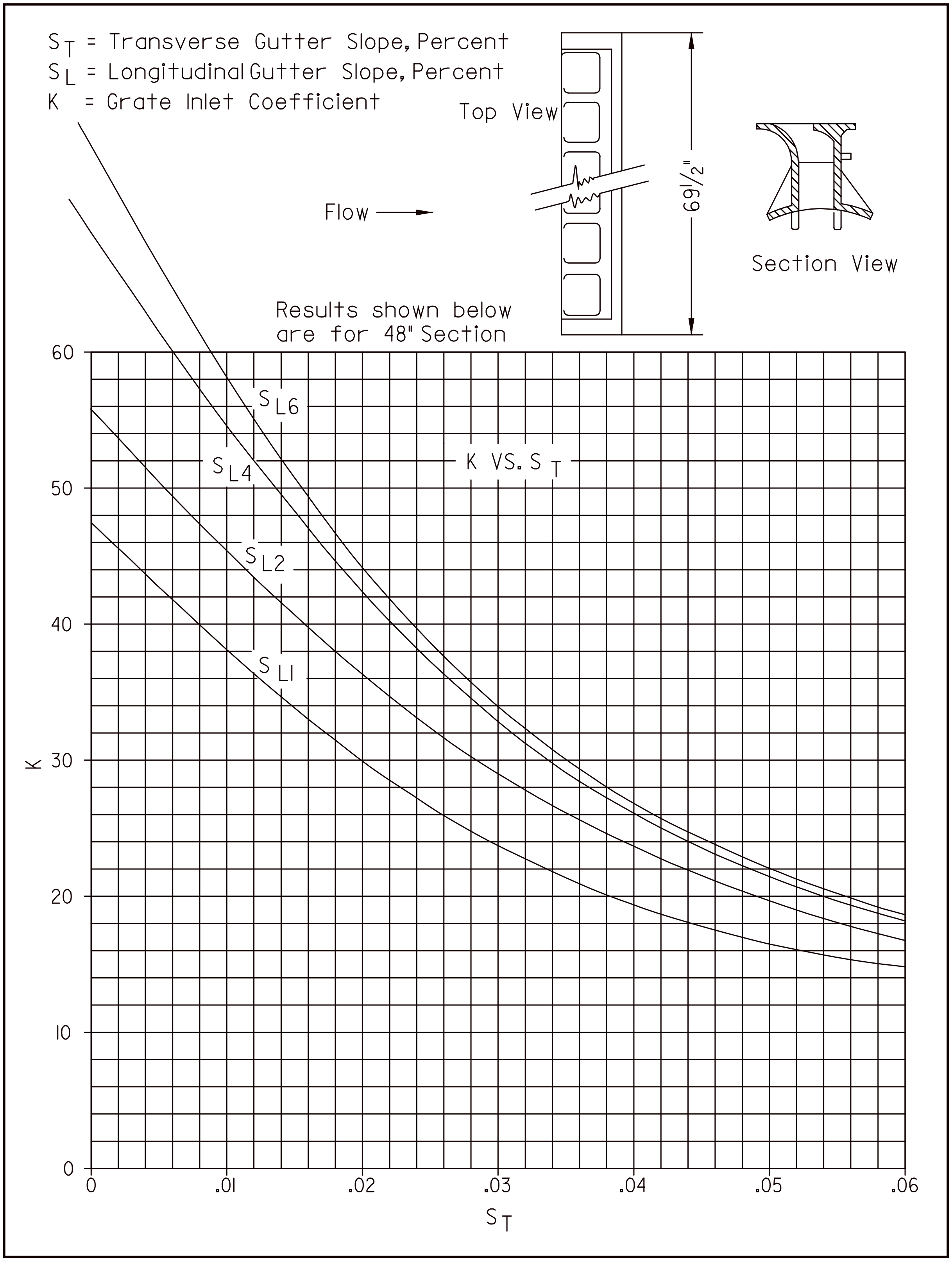


Exhibit G.13 Value of K for Slotted Vane Drain:
Applicable to Neenah Slotted Vane Drain R-3599 *Only*
(Source: Neenah Foundry Company)

REFERENCES

G.1 U.S. Department of Transportation, Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular (HEC) 12, FHWA-TS-84-202, 1984. (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec12.pdf)

APPENDIX H
NOMOGRAPHS AND CHARTS FOR STORM SEWER DESIGN

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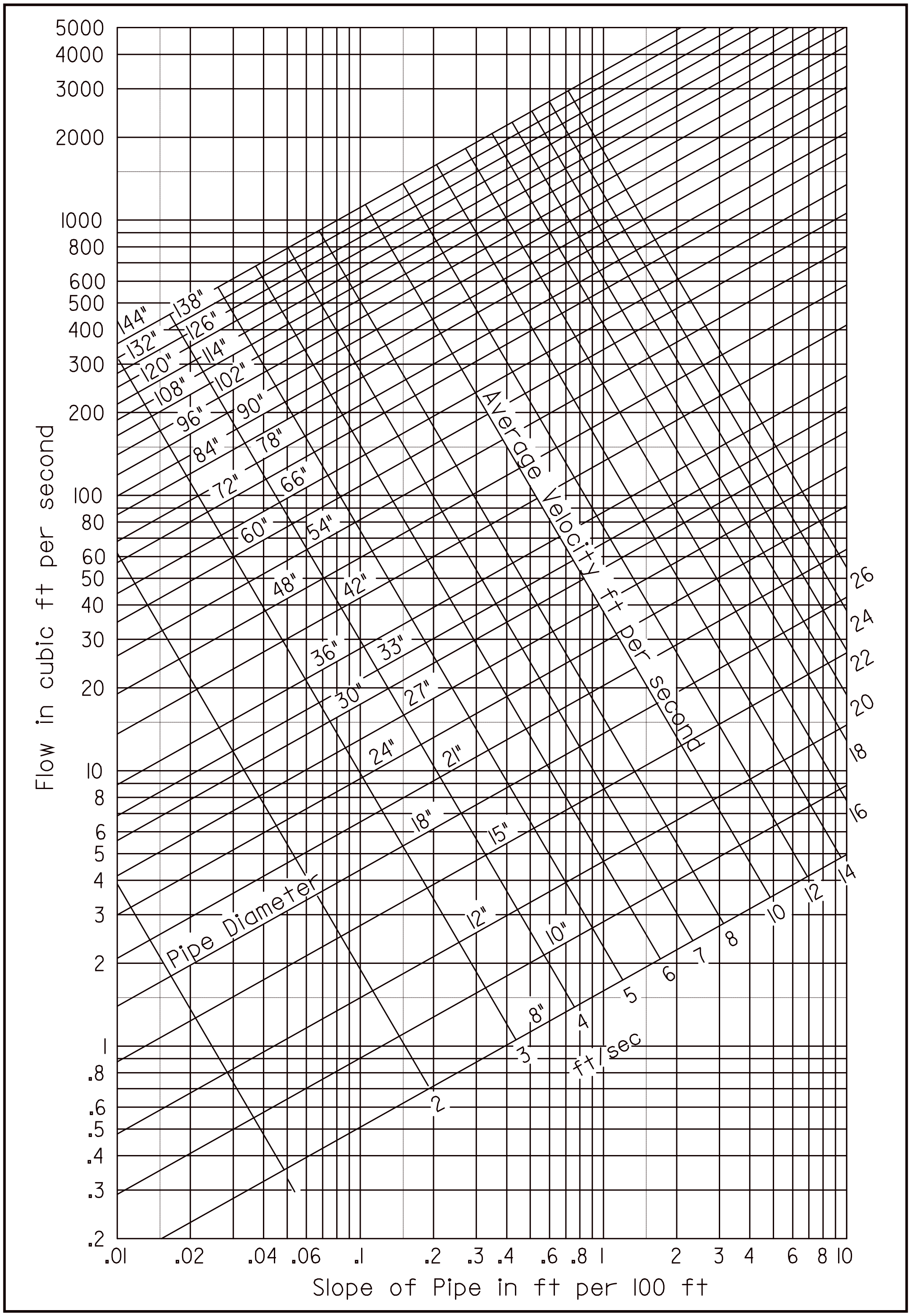

Exhibit H.1 Flow for Circular Pipe Flowing Full
Based on Manning's Equation (n=0.012)
(Source: Reference H.1)

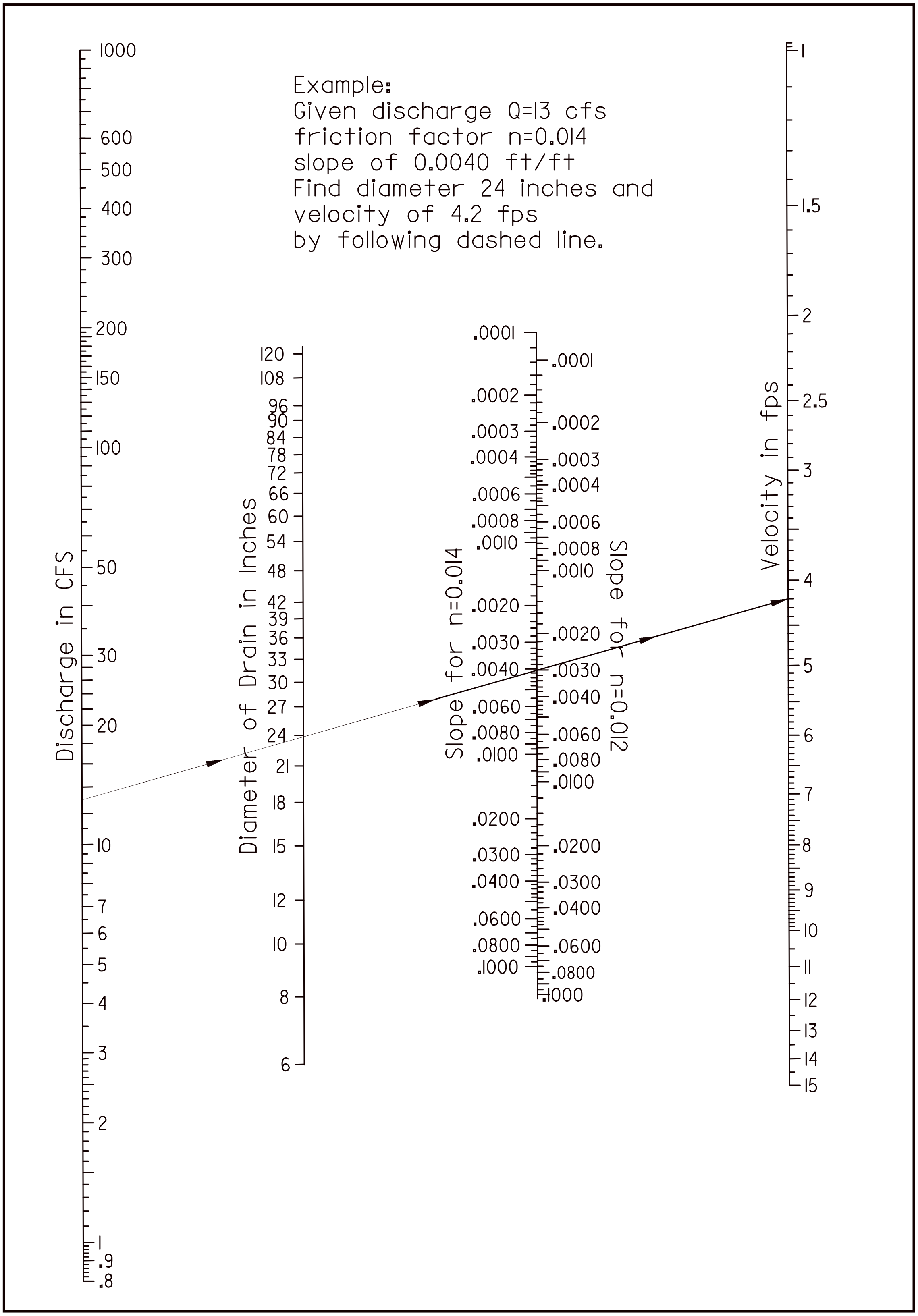

Exhibit H.2 Nomograph for Computing Required Size of Circular Pipe Flowing Full n=0.012 (Concrete) or n=0.014 (Clay)

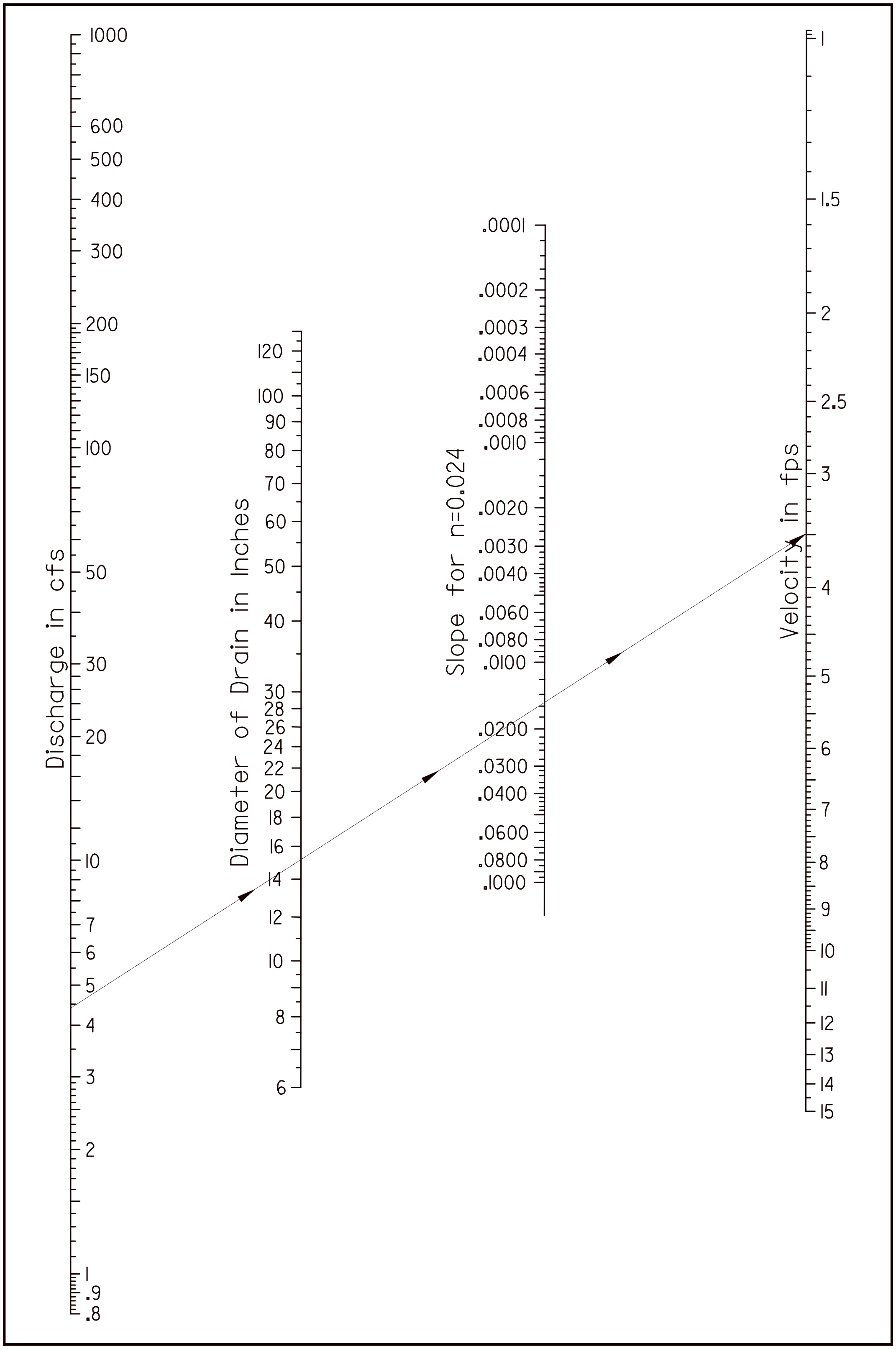

Exhibit H.3 Nomograph for Computing Required Size of Circular Pipe Flowing Full N=0.024 (CMP)

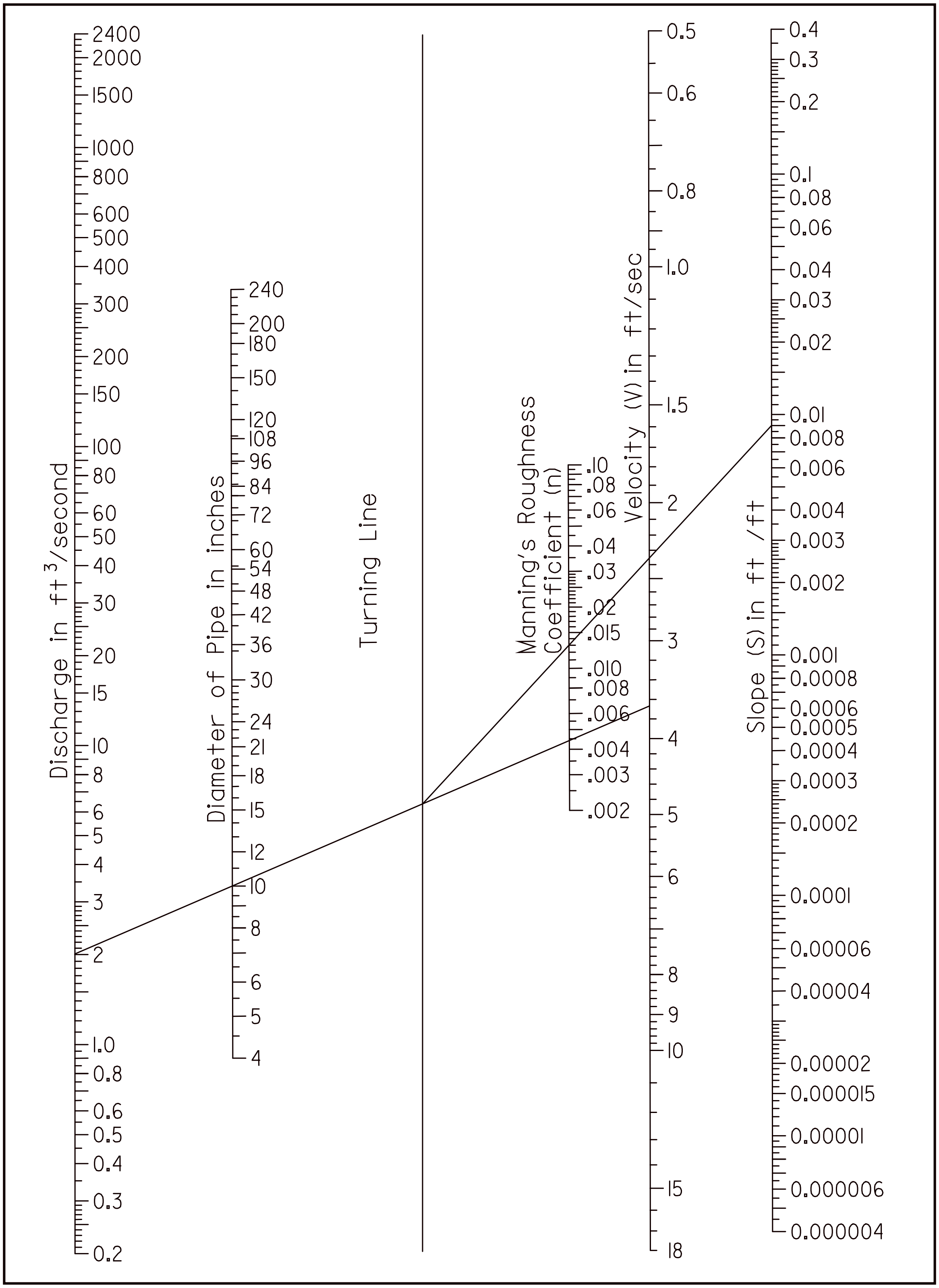

Exhibit H.4 Manning's Formula for Flow in Circular Pipe Flowing Full
(Source: Reference H.2)

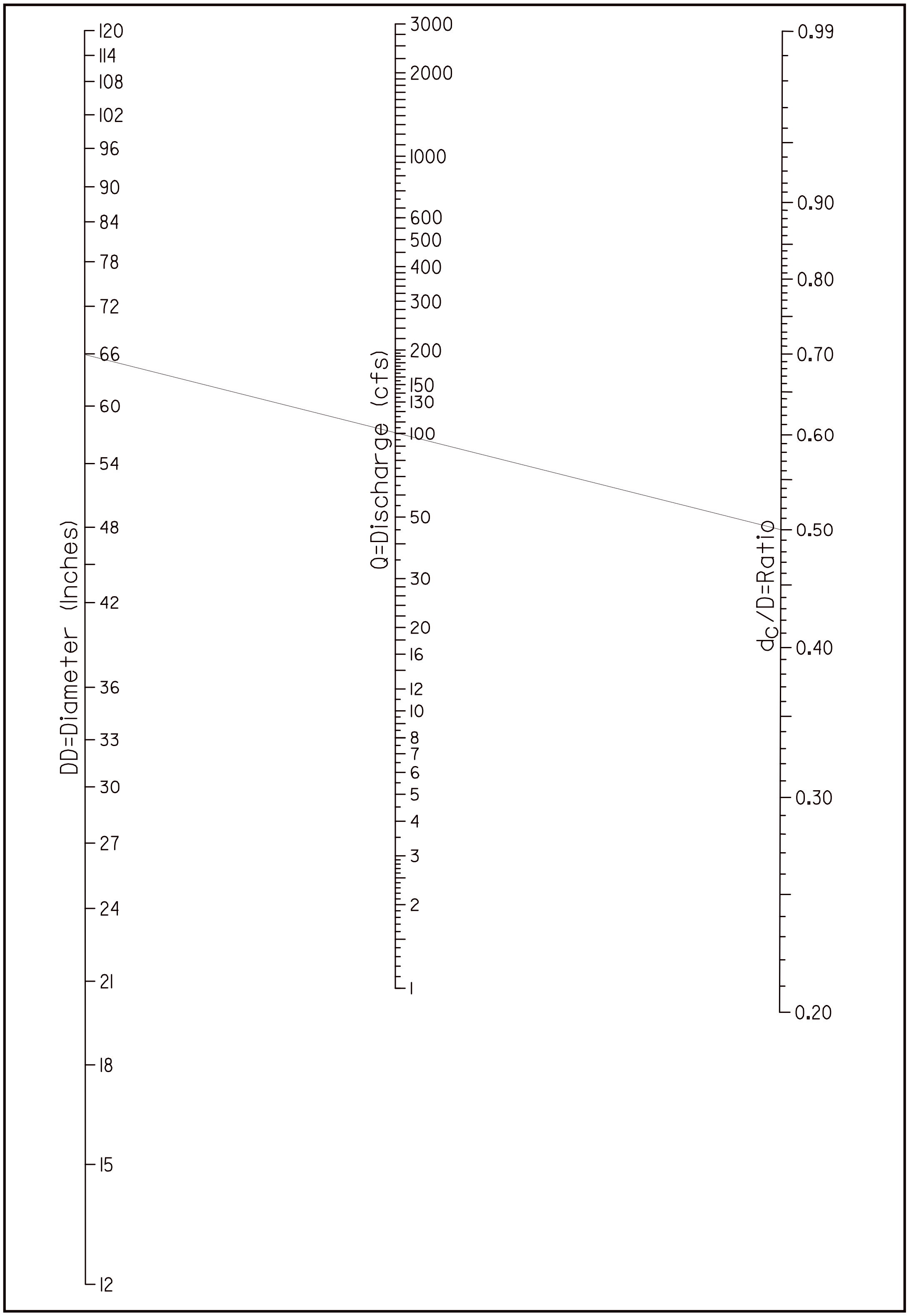

Exhibit H.5 Critical Depth of Flow for Circular Conduits
N=0.012 (Concrete) or n=0.024 (Corrugated Metal)

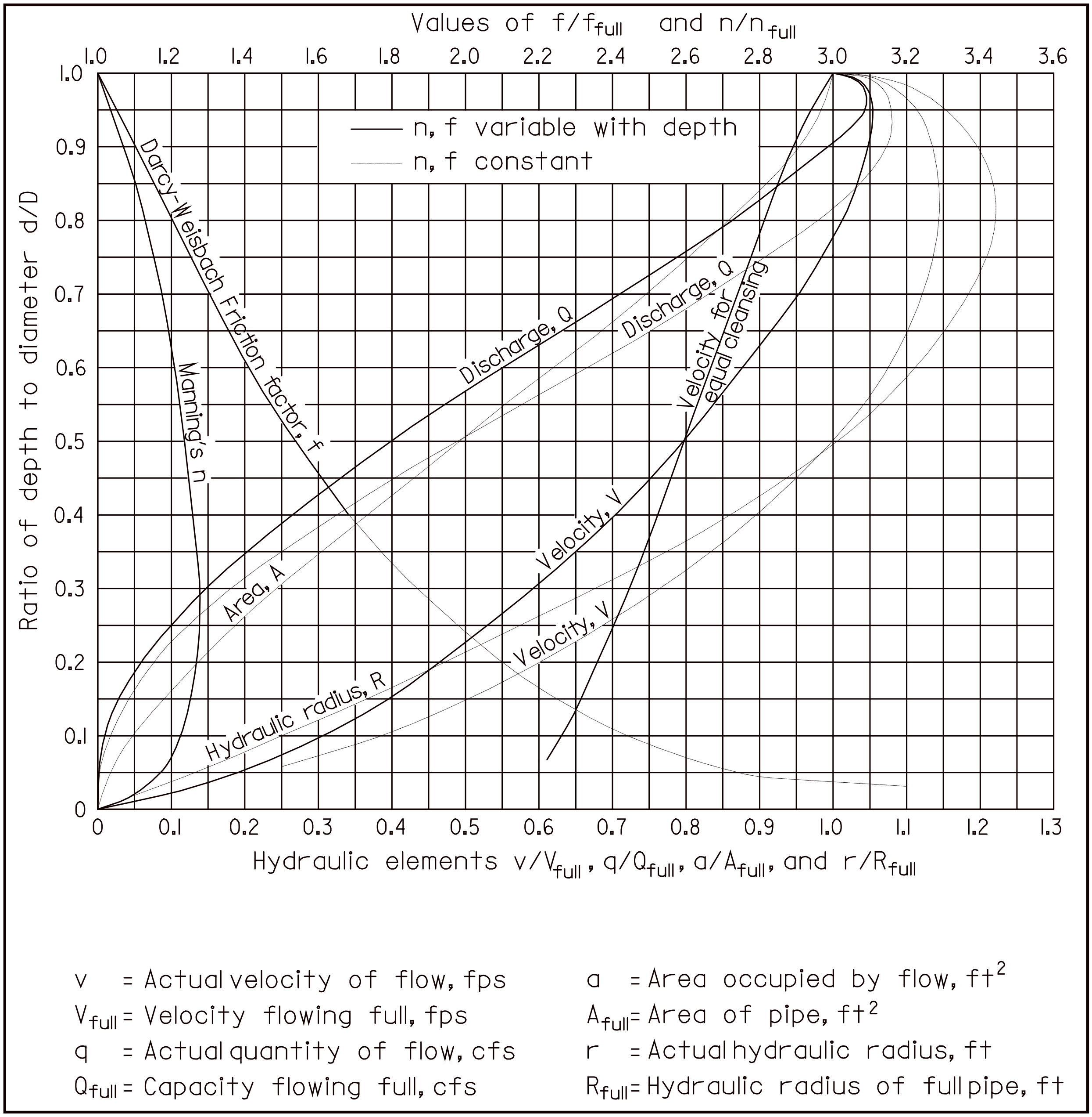

Exhibit H.6 Hydraulic Elements Chart
(Source: Reference H.3)

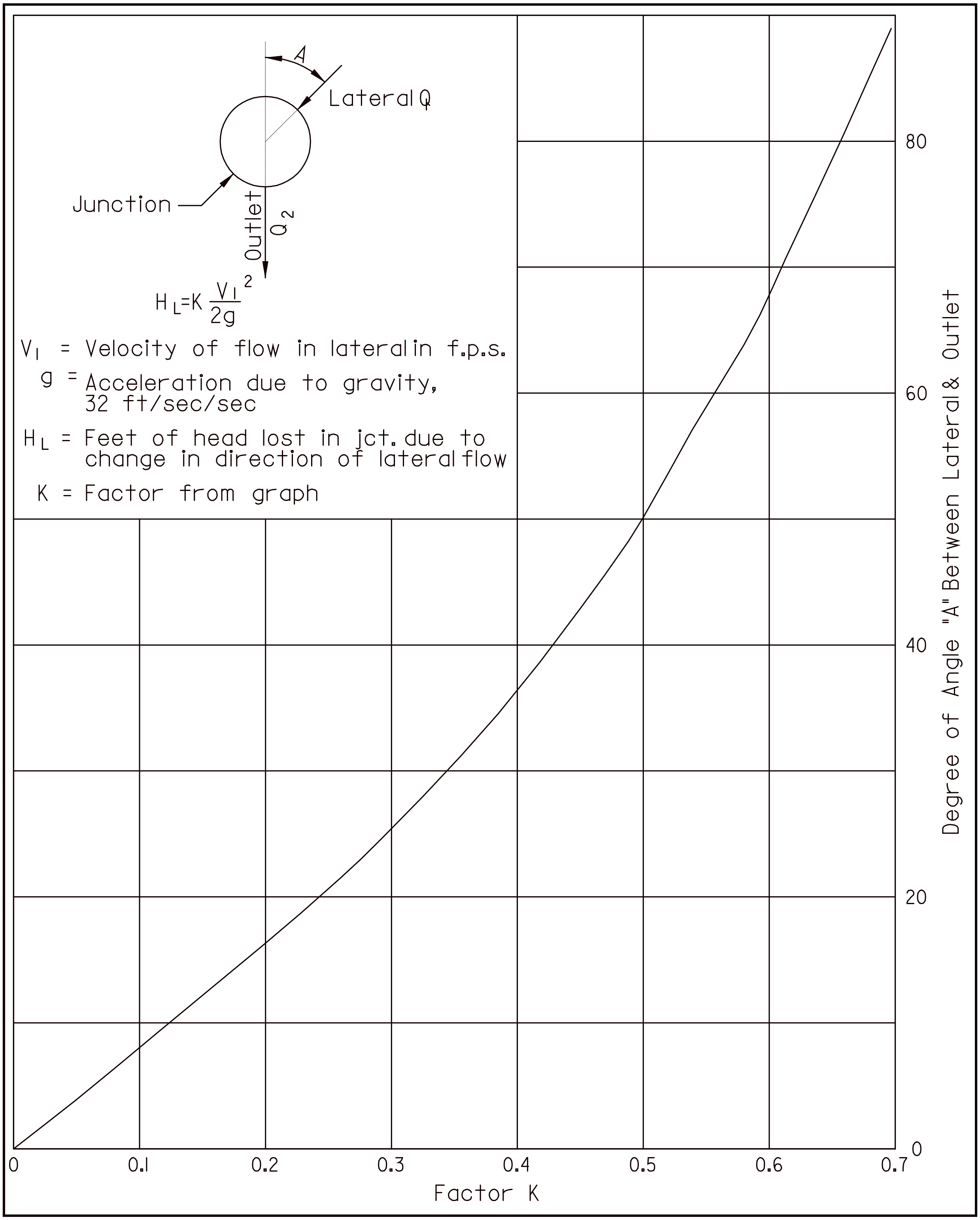

Exhibit H.7 Loss in Junction Due to Change in Direction of Flow in Lateral
(Source: Reference H.2)

REFERENCES

H.1 American Concrete Pipe Association. (http://www.concrete-pipe.org/index.php?cp_Session=805edca166f308d21f57c53735e572af)

H.2 U.S. Department of Transportation, Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular (HEC) 12, FHWA-TS-84-202, 1984. (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec12.pdf)

H.3 American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, Manuals and Reports on Engineering Practice - No. 37, 1979 Edition.

1. STRUCTURAL DESIGN OF RIGID AND FLEXIBLE CONDUITS

Designers should refer to the special plans for bedding and backfill requirements for culvert installation with up to 40 ft of fill. For culverts with over 40 ft of fill, designers should contact the **Highway Materials and Tests Manager** of the **Material and Research Division**.

1.A General

This section discusses the structural design of rigid and flexible conduits used for storm sewers, culverts, and sanitary sewers. The principles for calculating the external dead and live loads on underground conduits are based on studies conducted at Iowa State University by Marston, Schlick and Spangler. Soil Engineering, (Reference 1), provides additional discussion.

1.B Classification of Conduits

ASTM establishes standards for all types of materials.

1.B.1 Degree of Rigidity

Types of conduits available on the market today are as follows (classed by rigidity):

Rigid Conduits:

- Vitrified clay pipe (ASTM C 700, AASHTO M 65)
- Non-reinforced concrete pipe (ASTM C 985)
- Reinforced concrete pipe (ASTM C 76)
- Reinforced concrete arch pipe (ASTM C 506, AASHTO M 206)
- Reinforced concrete elliptical pipe (ASTM C 507, AASHTO M 207)
- Reinforced concrete culvert pipe (ASTM C76, AASHTO M 170)

Semirigid Conduits:

- Ductile iron pipe (ASTM A 746)

Flexible Conduits:

- Steel pipe (ASTM A 761)
- Corrugated metal pipe (ASTM A 806, AASHTO M 218)
- High-density polyethylene, HDPE (ASTM F 894, AASHTO M 294)
- Polyvinyl chloride, PVC (ASTM F 679)
- Acrylonitrile butadiene styrene, ABS (ASTM D 2680)
- Corrugated metal pipe, aluminized (ASTM C 76, AASHTO M 36)
- Aluminum corrugated pipe (ASTM B 790, AASHTO M 196)
- Corrugated metal pipe-arch (ASTM A 806, AASHTO M 218)
- Corrugated metal pipe-arch, aluminized (ASTM A 760, AASHTO M 36).

Semirigid conduits are generally considered rigid conduits for determining earth loads. Pipe material selection is a matter of policy (See Appendix C, "Pipe Material Policy").

1.B.2 Method of Installation

In order to determine the required design strength of a buried conduit, it is necessary to determine the total load that will be imposed upon the conduit. Calculation of the imposed loads is influenced by the installation condition. Underground conduits are classified into several groups and subgroups based upon the installation condition.

The two major classes are trench conduits and embankment conduits. Trench conduits are placed in natural ground in relatively narrow trenches. Embankment conduits are usually placed on natural ground and are overlaid by a constructed embankment.

The embankment group is further divided into positive projecting, negative projecting and induced trench subgroups, based upon the extent the pipe is exposed to direct embankment loading. Further subdivisions of the positive and negative projecting embankment groups are related to whether or not differential settlements may occur throughout the entire depth of backfill. EXHIBIT I.1 depicts the different features of the various types of installation conditions.

This appendix does not discuss loads on conduits installed in tunnel conditions.

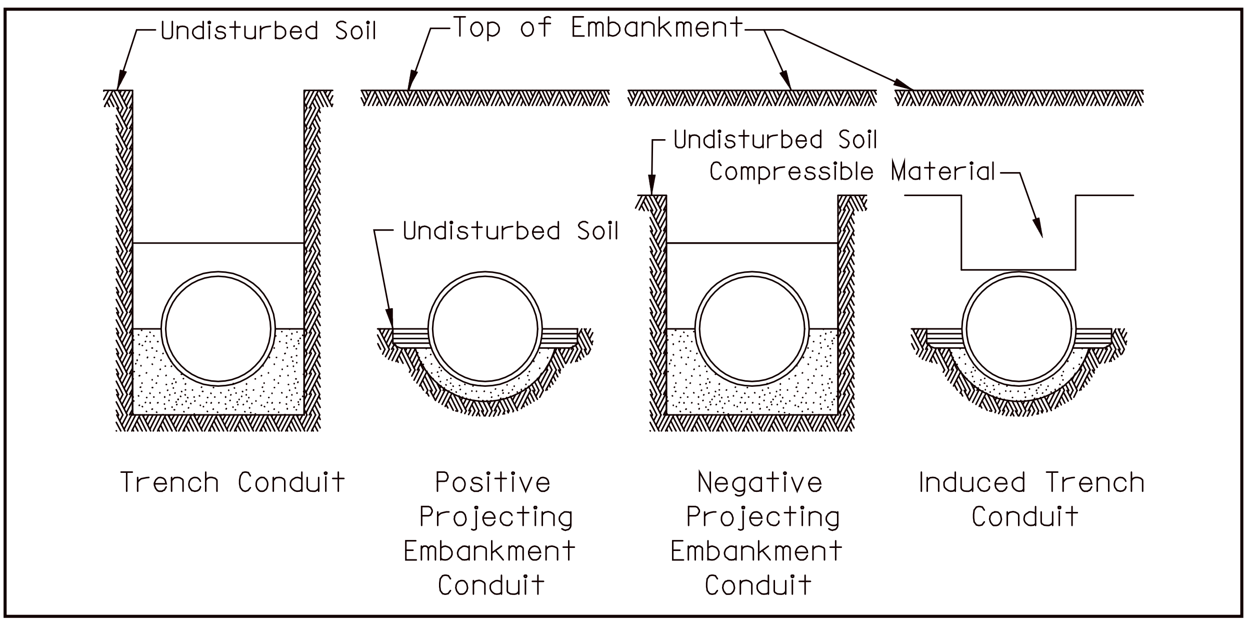

Exhibit I.1 Features of the Various Types of Underground Conduit Installations

1.C <u>Design of Rigid Conduits</u>

The governing standards most commonly used in the design of rigid conduits are those issued by ASTM. The designer should become familiar with these standards. Projects under design may have site-specific features that require modification of the standard.

1.C.1 Earth Loads on Trench Conduits

Trench conduits are installed in relatively narrow trenches excavated in undisturbed soil and then covered with earth backfill that extends to the original ground surface. The backfill load on the conduit is equal to the weight of the backfill material less the summation of the frictional load transfers, and is expressed by the equation:

$$W_c = C_d \, w \, B_d^2 \qquad \text{Eq. I.1}$$

where:
- W_c = Backfill load on conduit, lbs/lin ft.;
- C_d = Load coefficient for trench condition;
- w = Unit weight of backfill material, lbs/cu ft.;
- B_d = Width of trench at top of conduit, ft.

Equation I.1 indicates the width of the trench at the top of the pipe is the controlling factor in determining the backfill load on the conduit. At any given depth and for any given conduit size, however, there is a certain limiting value to the width of trench beyond which no additional load is transmitted to the conduit. This limiting value is called the "transition width." There are sufficient experimental data to show that it is safe to calculate the imposed load by means of the trench conduit formula (Eq. I.1) for all widths of trench less than that which gives a load equal to the load calculated by the positive projecting embankment conduit formula (Eq. I.2). In other words, as the width of the trench increases, other factors remaining constant, the load on a rigid conduit increases in accordance with the theory for a trench conduit until it equals the load determined by the theory for a positive projecting conduit. EXHIBIT J.1 in Appendix J, "Nomographs & Charts for Designing Earth Loads on Conduits", provides load coefficient (C_d) for trench condition.

1.C.2 Earth Loads on Positive Projecting Embankment Conduits

Positive projecting conduits are installed in shallow bedding with the top of the conduit projecting above the surface of the natural ground, or compacted fill at the time of installation, and then covered with earth fill. The load on a positive projecting conduit is computed by the equation:

$$W_c = C_c \, w \, B_c^2 \qquad \text{Eq. I.2}$$

where:
- W_c = Fill load on the conduit, lbs/lin ft.;
- C_c = Load coefficient for positive projecting embankment condition;
- w = Unit weight of fill material, lbs/cu ft.;
- B_c = Outside diameter of the conduit, ft.

EXHIBIT J.2 in Appendix J, "Nomographs & Charts for Designing Earth Loads on Conduits", provides values for the load coefficient (C_c) for positive projecting embankment condition. Values to be used for w and C_c are discussed further in Section 1.C.6. The r_{sd} values can be obtained from EXHIBIT I.2.

Installation and Foundation Condition	Settlement Ratio R_{sd}	
	Usual Range	Design Value
Positive Projecting............................	0.0 to +1.0	
Rock or Unyielding Soil.................	+1.0	+1.0
*Ordinary Soil...............................	+0.5 to +0.8	+0.7
Yielding Soil..................................	0.0 to +0.5	+0.3
Zero Projecting................................		0.0
Negative Projecting...........................	-1.0 to 0.0	
p'=0.5..		-0.1
p'=1.0..		-0.3
p'=1.5..		-0.5
p'=2.0..		-1.0
Induced Trench................................	-2.0 to 0.0	
p'=0.5..		-0.5
p'=1.0..		-0.7
p'=1.5..		-1.0
p'=2.0..		-2.0

* The value of the settlement ratio depends on the degree of compaction of the fill material adjacent to the sides of the pipe. With good construction methods resulting in proper compaction of bedding and sidefill materials, a settlement ratio design value of +0.5 is recommended.

Exhibit I.2 Design Values of Settlement Ratio (r_{sd})

1.C.3 Earth Loads on Negative Projecting Embankment Conduits

Negative projecting conduits are those which are installed in shallow trenches of such depth that the top of the conduit is below the surface of the natural ground or compacted fill and then covered with an embankment, which extends above this ground level. The procedure for computing the load on a negative projecting conduit is similar to that used in computing the load on a positive projecting conduit:

$$W_c = C_n \, w \, B_d \, B'_d \qquad \text{Eq. I.3}$$

where:
- W_c = Fill load on the conduit, lbs/lin ft;
- C_n = Load coefficient for negative projecting embankment condition;
- w = Unit weight of fill material, lbs/cu ft;
- B_d = Width of trench at top of the conduit, ft;
- B'_d = Average of the width of trench at the top of the conduit in ft, and the outside diameter of the conduit in ft: $(B_d + B_c)/2$

EXHIBIT J.3 in Appendix J, "Nomographs & Charts for Designing Earth Loads on Conduits", provides values for the load coefficient (C_n) for negative projecting embankment conditions and induced trench embankment conditions.

1.C.4 Earth Loads on Induced Trench Embankment Conduits

The induced trench method of construction is a practical method for relieving the load on conduits placed under very high fills. The essential features of this method of construction are described as follows:

1. Install the pipe in the same manner as a positive projecting embankment conduit with the desired class of bedding.
2. Compact fill material at each side of the pipe for a lateral distance equal to twice the outside diameter of the pipe or 12 ft, whichever is less. This fill is constructed up to an elevation of at least one pipe diameter over the top of the pipe.
3. Excavate a trench in the compacted fill directly over the pipe. The depth of the trench should be at least one pipe diameter and the width should coincide as nearly as possible with the outside diameter of the pipe.
4. Refill the induced trench with loose compressible material such as straw, sawdust or organic soil.
5. Complete the balance of the fill by normal methods up to the finished grade of the embankment.

The induced trench method of construction can also be obtained by constructing the embankment, before the pipe is installed, to an elevation above the top of the pipe equal to the external diameter of the pipe after which the trench is excavated and the pipe installed. Loose backfill is then placed directly above the pipe and the embankment completed.

The load on a conduit installed by the induced trench method is computed by the equation:

$$W_c = C_n\, w\, B_c^2 \qquad \text{Eq. I.4}$$

where:
- W_c = Fill load on the conduit, lbs/lin ft.;
- C_n = Load coefficient for induced trench condition;
- w = Unit weight of fill material, lbs/cu ft.;
- B_c = Outside diameter of the conduit, ft.

The induced trench method produces a less severe loading condition than either the positive projecting or negative projecting embankment conditions. The load that the pipe must support is reduced because the fill over the pipe will settle downward relative to the adjacent fill, thus generating shearing forces in an upward direction. This method of construction has another advantage since the sidefill material can be readily compacted adjacent to the sides of the pipe. This high degree of compaction enables development of effective lateral pressure against the sides of the pipe, which increases the in-place supporting strength of the pipe.

1.C.5 Live Loads on Buried Conduits

In the structural design of rigid conduits, it is necessary to evaluate the effect of live loads when the conduit is installed with shallow cover (0.5 to 6 ft) under an unsurfaced roadway and subjected to heavy truck traffic. If pavement is designed for heavy truck traffic, the intensity of the live load is usually reduced sufficiently so that the live load transmitted to the conduit is negligible. In the case of flexible pavements designed for light traffic and subjected to heavy traffic, the distribution of the live load through the pavement structure to the subgrade must first be evaluated. The intensity of the distributed load at the subgrade surface should then be used to determine the live load on the conduit.

The distribution of a wheel load applied at the surface to any horizontal plane in the subsoil results in the intensity of the load on any plane in the soil mass being greatest at the vertical axis directly beneath the point of application and decreases in all directions outward from the center of application. As the distance between the plane and the surface increases, the intensity of the load at any point on the plane decreases.

The total live load transmitted to an underground conduit can be determined by calculating the volume of the pressure intensity diagram. This volume is closely approximated by an elliptical cylinder and ellipsoid.

Based on this loading configuration, the total live load per ft of conduit is obtained from the equation:

$$W_L = \pi WL\ (2P_1 + P_2)\ /\ L + 24 \qquad \text{Eq. I.5}$$

where:
- W_L = Total live load transmitted to the conduit, lbs/lin ft.;
- π = 3.1416;
- W = Width of loaded area, inches;
- L = Length of loaded area, inches;
- P_1 = Vertical unit pressure at the center of the conduit due to applied live load, psi.;
- P_2 = Vertical unit pressure at the outside diameter of the conduit due to applied live load, psi.

1.C.6 Design Criteria for Calculating Earth Loads and Live Loads on Buried Rigid Conduits

The following section provides general design criteria for the designer when calculating earth loads and live loads on buried conduits.

1.C.6.a Sidewall Clearances and Minimum Design Trench Widths

Designers, in some instances, use trenches that are too narrow in order to reduce the earth load on the conduit. Sidewall clearances need to be established to allow a worker sufficient room to properly place and compact bedding material to avoid pipe failure. Sidewall clearances should include an allowance for construction tolerances since the conduit is not always in the center of the trench.

After sidewall clearances are determined, minimum design trench widths should be established and used for calculation of earth loads on the conduit.

Construction specifications must include the maximum allowable trench width when installing conduits in a trench condition. If the maximum trench width specified is not maintained by the contractor in the field, pipe failure may occur.

1.C.6.b Unit Weight of Soil

Unit weight of soil should be obtained from soil borings for the project (See the Roadway Design Manual, Chapter Seven: Earthwork, Section 6, (http://www.roads.nebraska.gov/business-center/design-consultant/rd-manuals/), Reference 2). A unit weight of 120 lb/cu ft is generally sufficient if job specific data are not available.

1.C.6.c Load Coefficient (C)

The value for the load coefficient varies with the type of soil (cohesive or non-cohesive) and is also dependent on the product of K and u'. K is the ratio of active lateral unit soil pressure to vertical unit soil pressure, and u' is the coefficient of friction between the backfill material and trench walls.

Values to be used for the load coefficient can be found in EXHIBITS J.1, J.2 AND J.3 in Appendix J, "Nomographs & Charts for Designing Earth Loads on Conduits".

1.C.7 Design Strength of Rigid Conduits

The strength of a buried rigid conduit, or the load (dead and live load) that a buried rigid conduit is capable of supporting, is primarily dependent upon the class of pipe bedding utilized and the structural strength of the pipe itself.

1.C.7.a Pipe Bedding

The load that a pipe will support depends in part on the width of the bedding contact area and the quality of the contact between the pipe and the bedding. To develop the full load-bearing capacity of a pipe, a segment of the bottom of the pipe equal to or greater than one-fourth of the outside diameter (B_c) must be firmly supported. Bell holes must be carefully excavated at proper intervals to ensure that no part of the load is supported solely by the bells.

Pipe bedding should be continuous between manholes or in the case of a roadway cross section, continuous throughout the fill. In addition, the same class of conduit should be maintained throughout the installation. Three classes of bedding for both circular and arch conduits are shown in EXHIBITS I.3 AND I.4 for trench and embankment bedding, respectively. Normally, the installation will be Class C, shaped subgrades; however, in cases where heavy loading is anticipated, such as at railroad crossings or in high fills, two solutions are available:

- Increase the supporting strength of the conduit by specifying a higher pipe class.
- Increase the support capacity by using Class A or Class B bedding.

Load factors (L_f) for each class of pipe bedding are indicated on each figure.

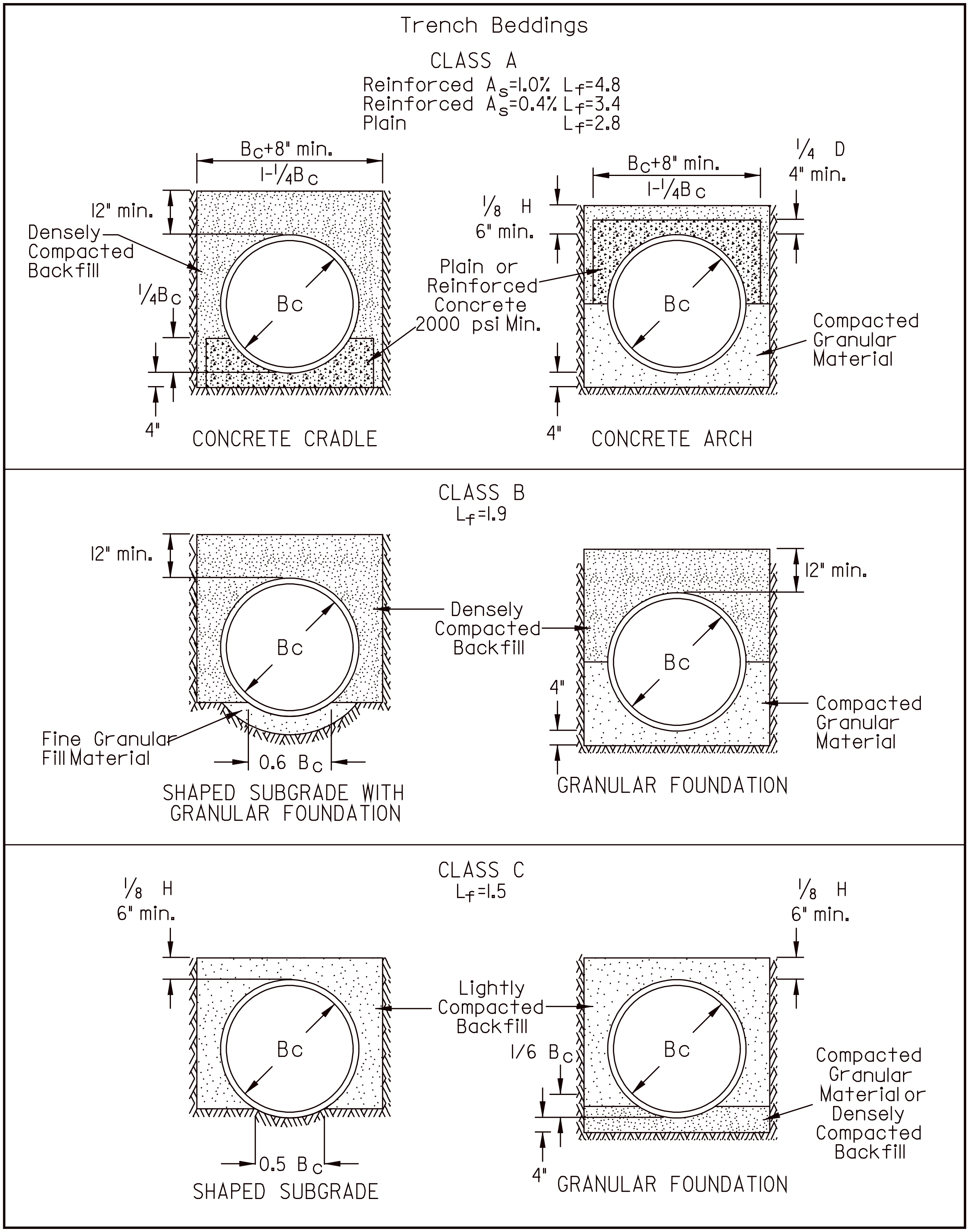

Exhibit I.3 Trench Beddings

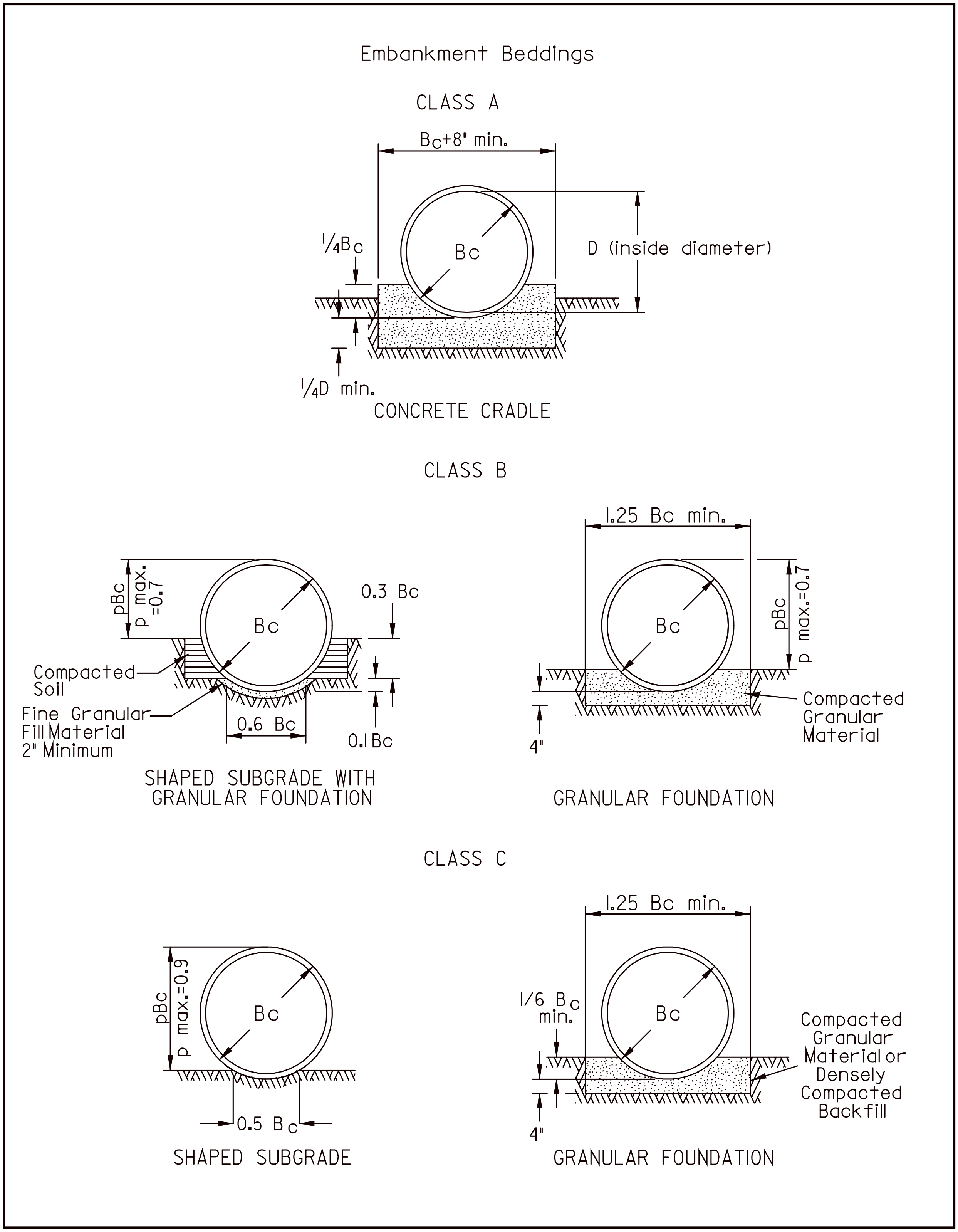

Exhibit I.4 Embankment Beddings

1.C.7.b Pipe Strength

Rigid conduits are tested for strength by the three-edge bearing test. The testing procedures are given in ASTM C 301 for vitrified clay pipe and ASTM C 497 for non-reinforced concrete pipe and reinforced concrete pipe. The three-edge bearing test is the most severe loading to which a rigid conduit will be subjected since the applied loads are essentially point loads.

Vitrified clay pipe and non-reinforced concrete pipe are tested to the ultimate load the pipe will withstand. Reinforced concrete pipe is tested to a load that will produce a 0.01 inch crack and the ultimate load the pipe will withstand. These loads are used to calculate the three-edge bearing strength.

Pipe classes are designated in the ASTM standards by the D-load where:

$$\text{D-Load} = \frac{TEB}{D} \qquad \text{Eq. I.6}$$

where:
- D-Load = lbs/lin ft/ft of diameter;
- TEB = three-edge bearing strength, lbs/lin ft;
- D = inside diameter of circular pipe (or inside horizontal span of elliptical and arch pipe), in ft.

D-load strength for reinforced concrete pipe can be classified as the 0.01 inch crack strength $D_{0.01}$, and the ultimate strength, D_{ult}. D-load strength for vitrified clay pipe and non-reinforced concrete pipe are designated only as ultimate strength, D_{ult}.

1.C.7.c Determination of Pipe Design Strength

Design strength can be determined from the following:

$$\text{design strength} = \frac{(\text{D-Load} \times D) \times L_f}{FS} \qquad \text{Eq. I.7}$$

where:
- design strength = lbs/lin ft.;
- D-Load = lbs/lin ft/ft diameter;
- D = Inside diameter of circular pipe (or inside horizontal span of elliptical and arch pipe), ft.;
- L_f = Load factor for pipe bedding;
- FS = Factor of safety.

For reinforced concrete pipe, decide if the design strength should be based upon 0.01 inch crack strength ($D_{0.01}$) or ultimate strength (D_{ult}). Also determine the factor of safety to be used. Safety factors for rigid conduit design generally range from 1.1 to 1.5.

Solutions for the various equations utilized for the design of rigid conduits are provided in Sections 2.A and 2.B.

1.D Design of Flexible Conduits

The following section provides general guidance for the designer regarding the structural design of flexible conduits. Consult the Handbook of PVC Pipe: Design and Construction, (Reference 3), for detailed discussion and design procedures.

The Handbook of PVC Pipe: Design and Construction, (Reference 3), provides theory and procedures for design of PVC pipe. The flexible conduit theory in Reference 3 also applies to PE pipe, however, parameters for PE pipe such as modules of elasticity will be different.

1.D.1 Flexible Conduit Theory

A flexible conduit is generally defined as a pipe that will deflect at least 2% without any sign of rupture or cracking. A flexible pipe derives its soil carrying capability from its flexibility. The buried pipe under load tends to deflect. The deflection mechanism transfers the vertical force derived from the load into approximately horizontal forces that are, in turn, opposed by opposite and equal reactions derived from the side support soil.

The amount of deflection that will occur in any buried flexible pipe depends on three factors:

- Pipe stiffness.
- Soil stiffness.
- Earth load and dead load on the pipe.

For further information see Deflection: The Pipe/Soil Mechanism, (Reference 4), (https://www.uni-bell.org/resources/applications/storm-sewer/engineering-design).

1.D.2 Pipe Stiffness

Pipe stiffness, as a value for a specific flexible pipe, is the unit load required to produce deflection in parallel plate loading to an arbitrary value, usually 5%.

Three-edge loading to failure is an appropriate measure of load bearing strength for rigid pipe but not for flexible pipe. When considering the load-carrying capability of flexible pipe, the soil stiffness must be considered as well as the pipe stiffness.

Pipe stiffness can be defined by the following equation:

$$PS = F/\Delta Y \geq \frac{EI}{0.149r^3} = 0.559\ E\ (t/r)^3 \qquad \text{Eq. I.8}$$

where:
- PS = Pipe stiffness, lbs/lin inch/inch or psi.;
- F = Force, lbs/lin inch;
- ΔY = Vertical deflection, inch;
- E = Modulus of tensile elasticity, psi.;
- I = Moment of inertia of the wall cross-section per unit length of pipe, in^4/linear inch = in^3.;
- r = Mean radius of pipe, inch;
- t = Wall thickness, inch.

Theoretical pipe stiffness may be calculated using the following equation:

$$PS = 4.47 \left[E/(DR-1)^3 \right] \qquad \text{Eq. I.9}$$

where:
- E = Modulus of tensile elasticity, psi.;
- DR = Dimension ratio = $\frac{OD}{t}$
- OD = Outside diameter, inch.
- t = Minimum wall thickness, inch.

An arbitrary data point of 5% deflection has been selected to permit meaningful comparison of pipe stiffness in different flexible pipes. Pipe stiffness values for all flexible pipes will vary with deflection. Pipe stiffness values can be obtained from the pipe manufacturer.

1.D.3 Soil Stiffness

Soil stiffness is defined as the soil's ability to resist deflection. Because of the ability of flexible pipe to interact with the surrounding soil in supporting a given load, soil stiffness is very important. The measurement of soil stiffness is termed the modulus of passive resistance (e). The modulus of passive resistance of side support soil is considered to be the unit pressure developed as the side of a flexible pipe moves outward against the side fill. The E' value is commonly termed the modulus of soil reaction and is defined as the product of the modulus of passive soil resistance, e, and the mean pipe radius.

EXHIBIT I.5 provides E' values for different types of soils, embedment materials, and densities.

Soil Type-Pipe Bedding Material (Unified Classification System[a]) (1)	**E' for Degree of Compaction of Pipe Zone Backfill, psi**			
	Loose (2)	**Slight < 85% Proctor. < 40% relative density (3)**	**Moderate 85%-95% Proctor. 40%-70% relative density (4)**	**High >95% Proctor. > 70% relative density (5)**
Fine-grained soils (LL > 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available: consult a competent soils engineer: Otherwise use E' = 0			
Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained soils with fines GM, GC, SM, SC[c] contains more than 12% fines	100	400	1,000	2,000
Coarse-grained soils with little or no fines GW, GP, SW, SP[c] contains less than 12% fines	200	1,000	2,000	3,000
Crushed rock	1,000	3,000	3,000	3,000
Accuracy in terms of percentage deflection[d]	$\pm$ 2	$\pm$ 2	$\pm$ 1	$\pm$ 0.5

[a]ASTM Designation D 2487, USBR Designation E-3.
[br]LL = Liquid Limit.
[c]Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).
[d]For $\pm$1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m) . Table does not include any safety factor. For use in predicting initial deflections only, appropriate deflection lag factor must be applied for long-term deflections. If bedding fails on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598.000 J/m^3) (ASTM D 698, AASHO -T99, USBR Designation E-11), 1 psi = 6.9 kN/m^2.

Exhibit I.5 Average Values of Modulus of Soil Reaction, E'
(For Initial Flexible Pipe Deflection) (Source: Reference 5)

1.D.4 Earth Loads on Flexible Conduits

The embankment load is used to calculate earth load on flexible pipe installed in trench conditions and is derived by determining the load imposed by the weight of the vertical prism of soil over the buried pipe, and is calculated as follows:

$$W_c = HwB_c \qquad \text{Eq. I.10}$$

where:
- W_c = Load on the conduit, lbs/lin ft.;
- H = Height of cover over the conduit, ft.;
- w = Unit weight of backfill, lbs/ft^3.;
- B_c = Outside diameter of conduit, ft.

When the prism load is expressed in psi, Eq. I.10 becomes:

$$P = \frac{wH}{144} \qquad \text{Eq. I.11}$$

where:
- P = Prism load on conduit, psi.;
- w = Unit weight of backfill, lbs/ft^3.;
- H = Height of cover over the conduit, ft.

Earth loads on buried flexible pipe when calculated using the prism load equation (Eq. I.11) provide a conservative design.

1.D.5 Live Loads on Flexible Conduits

The effects of live loads on flexible conduits are shown in EXHIBIT I.6. Generally, live loads are only significant at shallow depths.

Height Of Cover (ft)	Live Load Transferred To Pipe, lb/in^2 Highway H20[(A)]	Railway E80[(B)]	Airport [(C)]	Height Of Cover (ft)	Live Load Transferred To Pipe, lb/in^2 Highway H20[(A)]	Railway E80[(B)]	Airport [(C)]
1	12.50	N.R.	N.R.	16	N.S.	3.47	2.29
2	5.56	26.39	13.14	18	N.S.	2.78	1.91
3	4.17	23.61	12.28	20	N.S.	2.08	1.53
4	2.78	18.40	11.27	22	N.S.	1.91	1.14
5	1.74	16.67	10.09	24	N.S.	1.74	1.05
6	1.39	15.63	8.79	26	N.S.	1.39	N.S.
7	1.22	12.15	7.85	28	N.S.	1.04	N.S.
8	0.69	11.11	6.93	30	N.S.	0.69	N.S.
10	N.S.	7.64	6.09	35	N.S.	N.S.	N.S.
12	N.S.	5.56	4.76	40	N.S.	N.S.	N.S.
14	N.S.	4.17	3.06	------------	------------	------------	-------------

1 lb/in^2 = 144 lb/ft^2

(A) Simulates 20 ton truck traffic + impact.
(B) Simulates 80,000 lb/ft railway load + impact.
(C) 180,000 lb dual tandem gear assembly. 26 inch spacing between tires and 66 inch center-to-center spacing between fore and aft tires under a rigid pavements 12 inches thick + impact.
N.R. Not recommended.
N.S. Live load not significant.

Exhibit I.6 Live Loads on PVC Pipe

1.D.6 Deflection

The modified Iowa equation is used to determine deflection of buried flexible conduits and is expressed as:

$$\Delta X = D_L \frac{K w_c r^3}{EI + 0.061\, E' r^3} \qquad \text{Eq. I.12}$$

where:

- ΔX = Horizontal deflection, inch;
- D_L = Deflection lag factor;
- K = Bedding constant;
- w_c = Load per unit length of pipe, lbs/lin inch;
- r = Mean pipe radius, inch;
- E = Modulus of tensile elasticity of the pipe material, psi.;
- I = Moment of inertia per unit length, in^3.;
- E' = Modulus of soil reaction, psi.

The relationship between horizontal deflection (ΔX) and vertical deflection (ΔY) in buried flexible conduits can be determined from the following equation:

$$\Delta X = 0.913\, \Delta Y \qquad \text{Eq. I.13}$$

where:

- ΔX = Horizontal deflection, inch;
- ΔY = Vertical deflection, inch.

Under most soil conditions, flexible pipe tends to deflect into a nearly elliptical shape and the horizontal and vertical deflections may be considered equal for small deflections (Δ). Since most flexible pipe is described by either pipe stiffness ($F/\Delta Y$) or outside diameter to thickness ratio (DR), the modified Iowa equation (Eq. I.12) can be rewritten as follows:

$$\%\ \text{Deflection} = \%\ \frac{\Delta Y}{D} = \frac{D_L K P\ (100)}{0.149\ \frac{F}{\Delta Y} + 0.061 E'} \qquad \text{Eq. I.14}$$

or,

$$\%\ \text{Deflection} = \%\ \frac{\Delta Y}{D} = \frac{D_L K P\ (100)}{[2E/3(DR - 1)^3] + 0.061 E'} \qquad \text{Eq. I.15}$$

Where

- P = Prism load, psi.;
- w = Unit weight of soil, lbs/ft^3.;
- H = Height of cover over the pipe, ft and the rest as defined before.

When live loads must be considered, the following equation should be used:

$$\% \text{ Deflection} = \% \frac{\Delta Y}{D} = \frac{(D_L KP + K\,W')\,(100)}{[2E/3(DR - 1)^3] + 0.061E'} \qquad \text{Eq. I.16}$$

where:
- P = Prism load, psi.;
- K = Bedding constant;
- W' = Live load, psi.;
- DR = Dimension ratio;
- E = Modulus of tensile elasticity of the pipe material, psi.;
- E' = Modulus of soil reaction, psi.;
- D_L = Deflection lag factor.

The deflection lag factor (D_L) accounts for the fact that, in pipe/soil systems, the soil consolidation at the sides of the pipe continues at an ever decreasing rate with time, after the maximum load reaches the buried pipe. Experience demonstrates that deflection of buried flexible pipe will continue for a period of time after completion of pipe installation before final equilibrium is achieved.

The full load on any buried pipe is not reached immediately after installation unless the final backfill is compacted to a high density. For a pipe with good flexibility, the long term load will not exceed the prism load. The increase in load with time is the largest contribution to increasing deflection. Therefore, for design, the prism load can be used to compensate for the increased trench consolidation load with time and resulting increased deflection. When deflection calculations are based on prism loads, the deflection lag factor, D_L, should be 1.0.

The bedding constant (K) accommodates the response of the buried flexible pipe to the opposite and equal reaction to the load force derived from the bedding under the pipe. The bedding constant varies with the width and angle of the bedding achieved in the installation. The bedding angle (θ) is shown in EXHIBIT I.7.

EXHIBIT I.8 presents a list of bedding constant values dependent on the bedding angle. As a general rule, when using the modified Iowa equation, a value of K = 0.1 is assumed.

Equations I.14, I.15, and I.16 do not apply to profile wall flexible pipe, which is pipe similar to corrugated metal pipe in that it has corrugations on the inside or outside or both.

1.E Manufacturer Literature

Pipe manufacturers of rigid and flexible conduits often publish tables and other literature that include maximum recommended heights of cover for various classes and combinations of pipe and bedding. Before utilizing the above information, the designer should satisfy himself/herself that the tables are based upon acceptable design criteria and procedures.

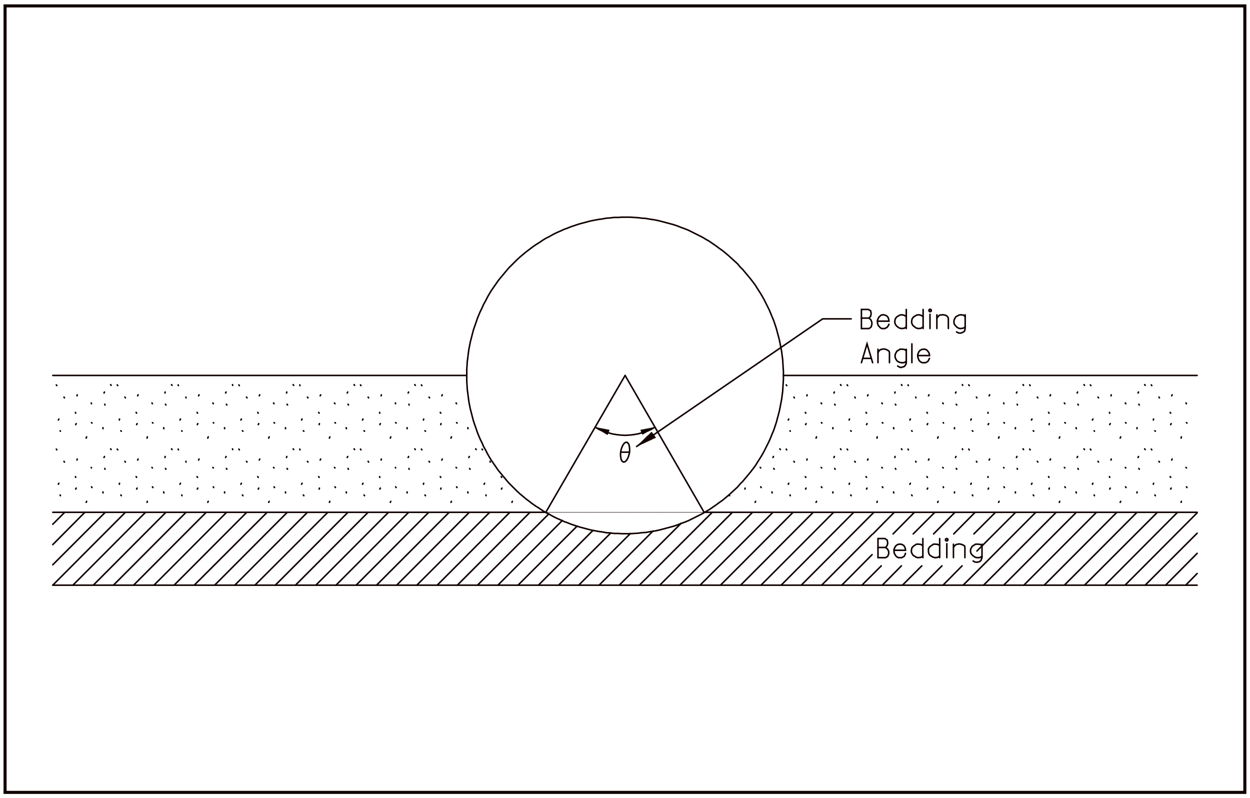

Exhibit I.7 Bedding Angle (Source: UNI-BELL PVC Pipe Association)

Bedding Angle (Degrees)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

Exhibit I.8 Values of Bedding Constant, K

2. EXAMPLE PROBLEMS

2.A Earth Loads on Rigid Trench Conduits

Determine the backfill load on a 24-inch diameter reinforced concrete pipe that is beneath 14 ft of cover installed in a trench. Backfill material is saturated topsoil.

Step 1: Determine the outside diameter of pipe (B_c).

Assume wall thickness is 2 inches. Refer to appropriate ASTM for actual wall thickness.

B_c = 24 in + 2 (2 in) = 28 in = 2.33 ft.

Step 2: Calculate the width of the trench at the top of the pipe (B_d).

Allow 12-inch sidewall clearance on each side of the pipe.

B_d = 2.33 ft + 2 ft = 4.33 ft.

Step 3: Find the load coefficient for trench conditions (C_d) using EXHIBIT J.1 in Appendix J, "Nomographs & Charts for Designing Earth Loads on Conduits".

H/B_d = 14 ft/4.33 ft = 3.23. Therefore, C_d = 2.1.

Step 4: Determine the backfill load on the pipe (W_c) using Eq. I.1.

Assume unit weight of backfill material (w) is 120 lbs/cu ft.

$W_c = C_d w B_d^2$

$= 2.1 \times 120 \times (4.33)^2$

= 4,725 lbs/lin ft.

2.B Rigid Conduit Design

A 24-inch diameter reinforced concrete pipe will be placed in a trench with Class C shaped subgrade bedding. Determine the class of pipe required to support a backfill load of 4,725 lbs/lin ft. Assume that live load is negligible.

Step 1: Select a trial class of pipe and determine the D-Load value from ASTM C-76.

For Class IV pipe, D-Load_{ult} = 3000 lbs/lin ft/ft diameter.

Step 2: Determine the load factor (L_f), for the specified trench bedding from EXHIBIT I.3.

$L_f = 1.5$.

Step 3: Calculate the design strength of the pipe using Eq. I.7.

$$\text{Design Strength} = \frac{(\text{D-Load} \times D) \times L_f}{FS}$$

$$= \frac{(3000 \text{ lbs/lin ft/ft diameter} \times 2 \text{ ft}) \times 1.5}{1.5}.$$

= 6000 lbs/lin ft

Step 4: Evaluate trial class of pipe.

6000 lbs/lin ft > 4,725 lbs/lin ft.

Therefore, Class IV pipe with Class C shaped subgrade bedding is adequate.

2.C Earth Loads on Flexible Trench Conduits

Determine the backfill load on an 18-inch PVC pipe beneath 11 ft of cover installed in a trench. Backfill material is saturated topsoil.

Step 1: Determine the outside diameter of the pipe (B_c).

From manufacturer's literature:

B_c = 18.70 in = 1.56 ft.

Step 2: Determine the unit weight of the backfill material (w).

Assume unit weight is 120 lbs/cu ft.

Step 3: Determine the backfill load on the pipe (W_c) using Eq. I.10.

$W_c = HwB_c = 11 \times 120 \times 1.56 = 2{,}059$ lbs/lin ft.

2.D Flexible Conduit Design

Determine the amount of deflection (percent) that will occur in an 18-inch PVC pipe installed under the following conditions. Evaluate if the amount of deflection is acceptable.

Pipe Size and Material:	18-inch (nominal) PVC
Outside Diameter:	18.70-inch (average)
Wall Thickness:	0.534-inch
Modulus of Tensile Elasticity, E':	400,000 psi
Installation Conditions:	trench condition
Cover Depth:	11 ft
Backfill Material:	saturated topsoil
Unit Weight of Backfill Material:	120 lbs/cu ft

Pipe Bedding Material: fine-grained soils (LL <50) with more than 25 coarse-grained particles.

90% compaction (Proctor)

Live Load: H20 (highway).

Step 1: Determine the modulus of soil reaction (E') from EXHIBIT I.5.

E' = 1000 psi.

Step 2: Calculate the dimension ratio (DR).

$$DR = \frac{OD}{T} = \frac{18.70}{0.534} = 35.02$$

Step 3: Find the prism load (P) using Eq. I.11.

$$P = \frac{wh}{144}$$

$$P = \frac{120 \times 11}{144} = 9.17 \text{ psi}$$

Step 4: Obtain bedding constant (K) from EXHIBIT I.8.

Bedding angle is assumed to be approximately zero, thus, K = 0.110.

Step 5: Calculate percentage of deflection using Eq. I.15.

Deflection lag factor (D_L) is 1.0 since calculations are based on prism load.

$$\% \text{ Deflection} = \frac{D_L KP(100)}{[2E/3(DR - 1)^3] + 0.061E'}$$

$$= \frac{1.0 \times 0.110 \times 9.17 \times 100}{[2(400{,}000)/3(35\text{-}1)^3] + 0.061(1000)}$$

$$= 1.48\%.$$

Allowable deflection is generally 5%, therefore 1.48% is acceptable.

3. REFERENCES

1. Spangler, M.G. and R.L. Handy, Soil Engineering, 4th Edition, Harper and Row, 1982.

2. Nebraska Department of Transportation, Roadway Design Manual, Current Edition (http://www.roads.nebraska.gov/business-center/design-consultant/rd-manuals/).

3. Handbook of PVC Pipe: Design and Construction; Uni-Bell PVC Pipe Association; Dallas, Texas; 1986.

4. Deflection: The Pipe/Soil System Mechanism; Uni-Bell PVC Pipe Association; Dallas, Texas; 1990. (https://www.uni-bell.org/resources/applications/storm-sewer/engineering-design).

5. Amster Howard, Soil Reaction for Buried Flexible Pipe, ASCE Journal of Geotechnical Engineering Division, January, 1977.

APPENDIX J
NOMOGRAPHS & CHARTS FOR DESIGNING EARTH LOADS ON CONDUITS

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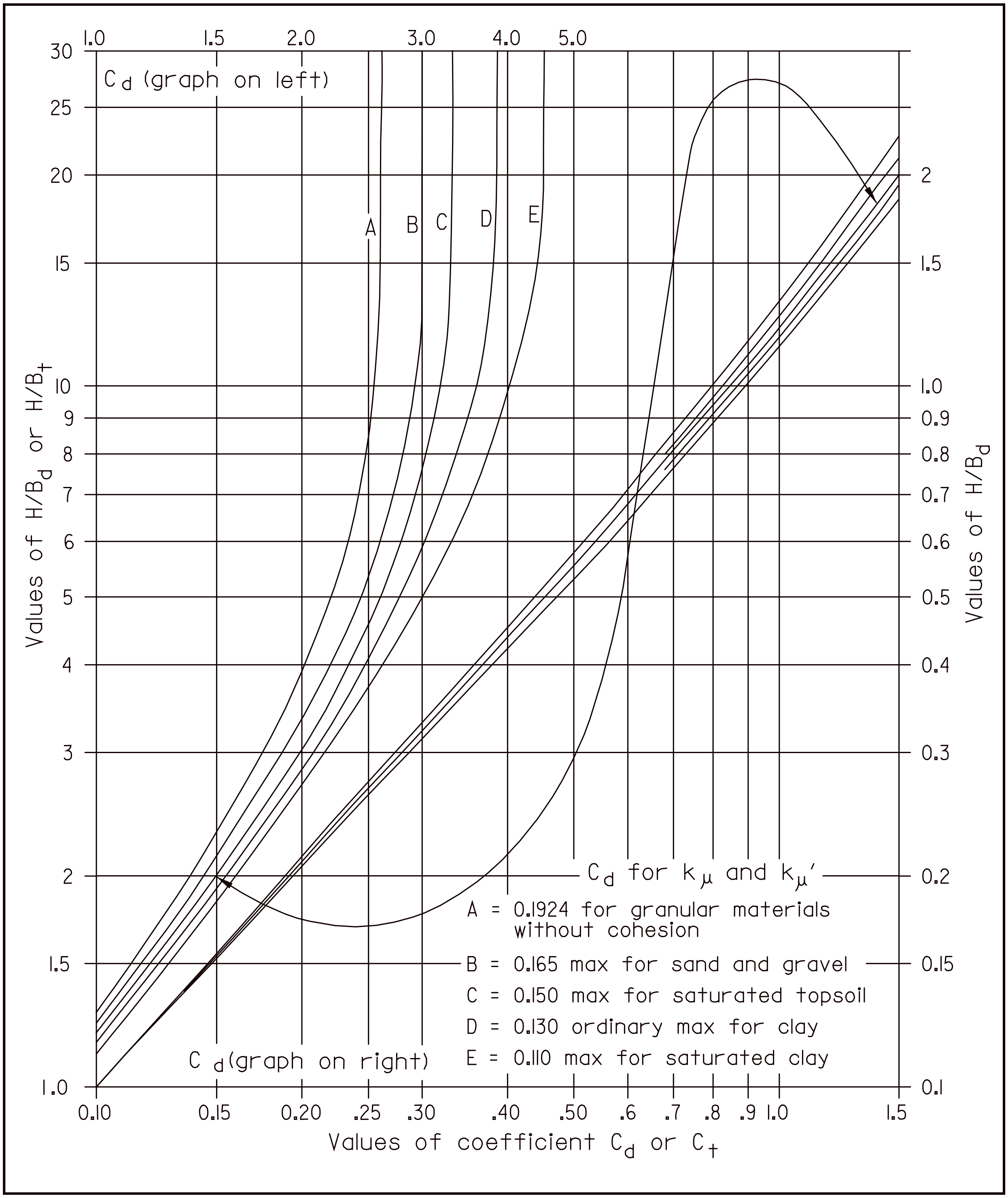


Exhibit J.1 Load Coefficient (C_d) for Trench Condition
(Source: Reference 1)

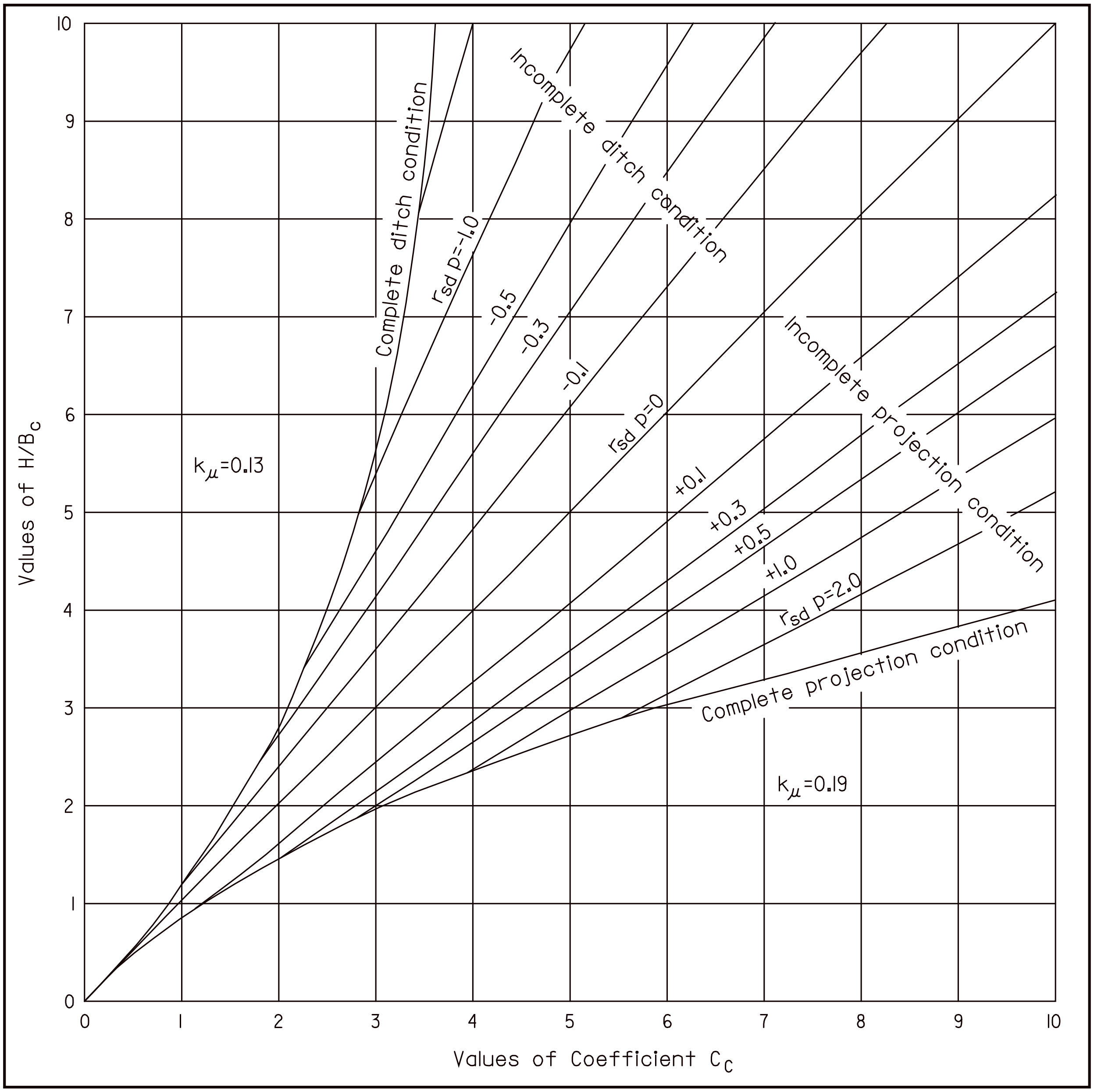


Exhibit J.2 Load Coefficient (C_c) for Positive Projecting Embankment Condition (Source: Reference 1)

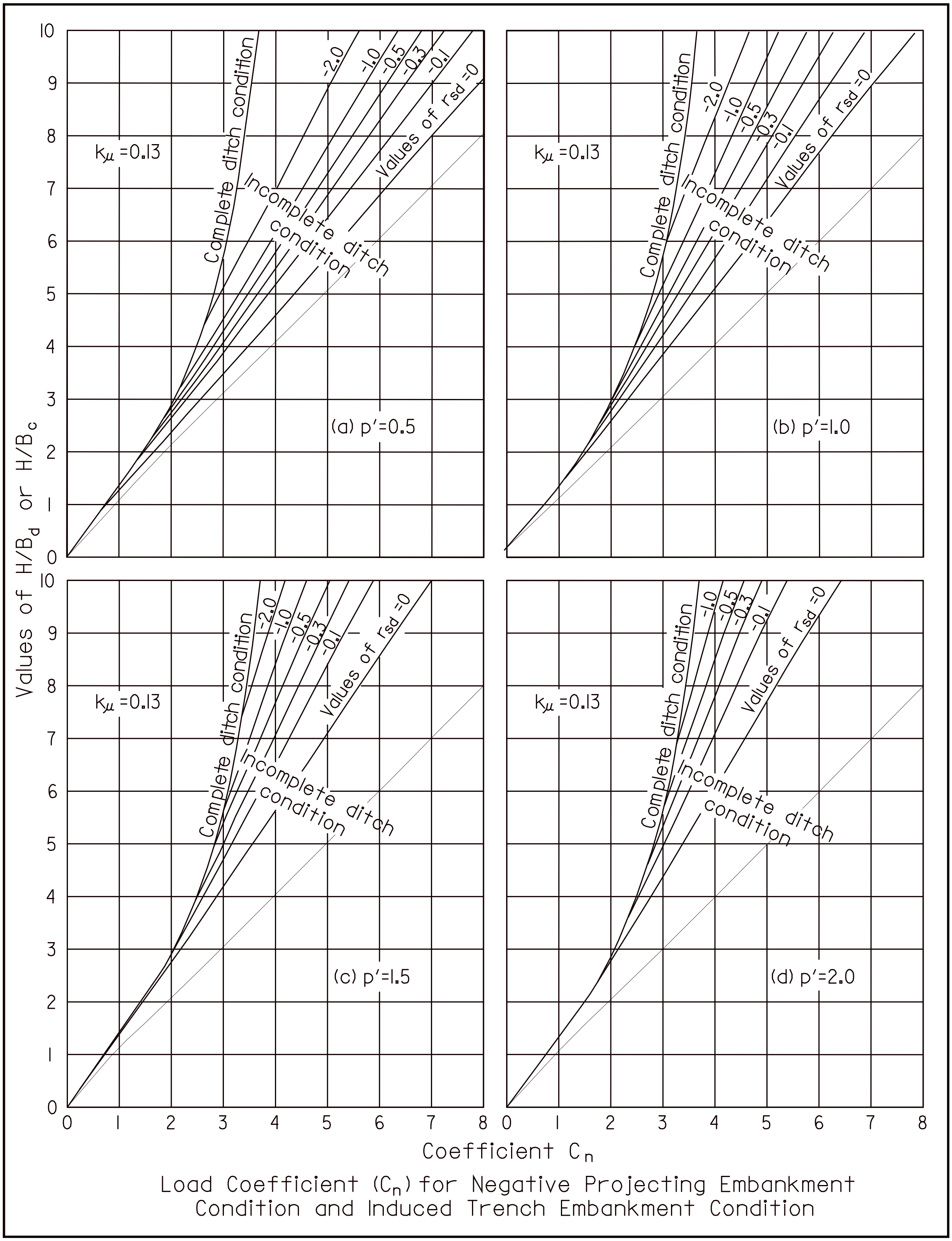


Exhibit J.3 Load Coefficient (C_n) for Negative Projecting Embankment Condition and Induced Trench Embankment Condition
(Source: Reference 1)

REFERENCES

1. American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, Manuals and Reports on Engineering Practice – No. 37, 1979 Edition.

Equa. No.	Description	Equation	Page No.
1.1	Peak Runoff (Rational Method)	$Q = CiA$	1-13
1.2	Time of Concentration (Kirpich Equation)	$T_c = 0.0078\ L^{0.77}\ S^{-0.385}\ C_F$	1-21
1.3	Critical Depth in Open Channel Flow	$A^3/T = Q^2/g$	1-31
1.4	Froude Number	$Fr = V/(gD)^{0.5}$	1-32
1.5	Manning's Equation for evaluating uniform flow in open channels	$V = \frac{1.486}{n} R^{2/3}\ S^{1/2}$	1-33
1.6	Compound Bend Angle	$\gamma = 180° - Cos^{-1}(-\cos(\alpha) \times \cos(\beta))$	1-54
1.7a	Modified Manning's Equation for evaluating gutter flow hydraulics (depth known)	$Q = 0.56(z/n)\ S^{1/2} d^{8/3}$	1-66
1.7b	Modified Manning's Equation for evaluating gutter flow hydraulics (spread known)	$Q = (0.56/n)\ S_x^{5/3} S^{1/2} T^{8/3}$	1-66
1.8	Determine pavement cross slope for a V-shaped gutter	$S_X = S_{X1} S_{X2} / (S_{X1} + S_{X2})$	1-66
1.9	Length of inlet required to intercept 100% of gutter flow	$L_a = Q_a/(Q_a/L_a)$	1-72
1.10	Ratio of frontal flow to total gutter flow for grate inlets	$E_o = Q_w/Q = 1 - (1 - W/T)^{2.7}$	1-74
1.11	Ratio of side flow to total gutter flow for grate inlets	$Q_s/Q = 1 - Q_w/Q = 1 - E_o$	1-74
1.12	Ratio of frontal flow intercepted to total frontal flow for grate inlets	$R_f = 1 - 0.09\ (V - V_o)$	1-74
1.13	Ratio of side flow intercepted to total side flow for grate inlets	$R_s = 1 / [1 + (0.15V^{1.8}/S_x L^{2.3})]$	1-74
1.14	Efficiency of a grate	$E = R_f E_o + R_s(1-E_o)$	1-75
1.15	Interception capacity of a grate on grade	$Q_i = EQ = Q[R_f E_o + R_s(1-E_o)]$	1-75
1.16	Capacity for a grate inlet in a low point or sump, inlet acting as a weir	$Q_i = CPd^{1.5}$	1-76
1.17	Capacity for a grate inlet in a low point or sump, inlet operating as an orifice	$Q_i = CA(2gd)^{0.5}$	1-76
1.18	Length of slotted pipe inlet required to intercept 100% of gutter flow on continuous grade	$L_T = 0.6Q^{0.42} S^{0.3} (1/nS_x)^{0.6}$	1-77
1.19	Interception efficiency of a slotted pipe inlet shorter than the length required for total interception of gutter flow on continuous grade	$E = 1 - (1 - L/L_T)^{1.8}$	1-77
1.20	Actual gutter flow intercepted by a slotted pipe inlet on a continuous grade	$Q_i = EQ$	1-77
1.21	Capacity for a slotted pipe inlet in a low point or sump, inlet operating as an orifice	$Q_i = 0.8LW(2gd)^{0.5}$	1-78
1.22	Capacity for a slotted pipe inlet (with a slot width of 1.75 in) in a low point or sump, inlet operating as an orifice	$Q_i = 0.94Ld^{0.5}$	1-78
1.23	Capacity of a slotted vane drain	$Q = Kd^{5/3}$	1-78

Equa. No.	Description	Equation	Page No.
1.24	Discharge rate of flow of a storm sewer	$Q = \dfrac{1.486}{n} A R^{2/3} S^{1/2}$	1-81
1.25	Velocity of flow in a storm sewer flowing full	$V_{full} = \dfrac{0.590}{N} D^{2/3} S^{1/2}$	1-81
1.26	Discharge rate of flow in a storm sewer flowing full	$Q_{full} = \dfrac{0.463}{n} D^{8/3} S^{1/2}$	1-81
1.27	Energy losses from pipe friction in storm sewers	$S_f = [Qn/1.486\ AR^{2/3}]^2$	1-82
1.28	Head losses due to friction in storm sewers	$H_f = S_f L$	1-82
1.29	Velocity head losses in storm sewers	$H = K(V^2)/2g$	1-83
1.30	Terminal losses in storm sewers	$H_{tm} = (V^2)/2g$	1-83
1.31	Entrance losses in storm sewers	$H_e = K(V^2)/2g$	1-83
1.32	Head loss at a storm sewer junction	$H_{j1} = (V^2)\ (outflow)/2g$	1-83
1.33	Energy loss in a storm sewer junction due to a change in direction of flow	$H_b = K(V^2)\ (outlet)/2g$	1-84
1.34	Energy loss in a storm sewer junction with several entering flows	$H_{j2} = [(Q_4V_4^2)-(Q_1V_1^2)-(Q_2V_2^2)+(KQ_1V_1^2)]/(2gQ_4)$	1-85
2.1	Shear stress on a channel	$\tau = \gamma RS$	2-27
2.2	Permissible shear stress for non-cohesive soils	$\tau_p = (4.0\ lbs/cu\ ft) \times (D_{50} \div SF)$	2-28
2.3	Channel bend shear stress	$\tau_b = (K_b) \times (\tau_{max})$	2-30
2.4	Distance increased shear stress travels down a channel from a bend	$L_p = (0.604) \times (R^{7/6} \div n_b)$	2-30
2.5	Brink depth at a pipe culvert outlet	$Y_e = (A \div 2)^{0.5}$	2-55
2.6	Froude Number	$V_0 \div (g\ Y_e)^{0.5}$	2-55
2.7a – 2.7f	Variable used in calculating riprap basin depression	$F \div Y_e$ = (variable x Fr) – variable	2-55
2.8	Riprap basin depression	$F = (F \div Y_e) \times Y_e$	2-55
I.1	Earth loads on trench conduits	$W_c = C_d\ w\ B_d^2$	I-3
I.2	Earth loads on positive projecting embankment conduits	$W_c = C_c\ w\ B_c^2$	I-3
I.3	Earth loads on negative projecting embankment conduits	$W_c = C_n\ w\ B_d\ B'_d$	I-4
I.4	Earth loads on induced trench embankment conduits	$W_c = C_n\ w\ B_c^2$	I-5
I.5	Live loads on buried conduits	$W_L = \pi W\ L\ (2P_1 + P_2)/\ L + 24$	I-6
I.6	Pipe strength	D-Load = TEB / D	I-10
I.7	Pipe design strength	$\dfrac{(\text{D-Load} \times D) \times L_f}{FS}$	I-10
I.8	Pipe stiffness	$PS = F/\Delta Y \geq EI / 0.149r^3 = 0.559\ E\ (t/r)^3$	I-11
I.9	Theoretical pipe stiffness	$PS = 4.47\ [\ E/(DR-1)^3\ ]$	I-12
I.10	Earth loads on flexible culverts (lbs/lin ft)	$W_c = HwB_c$	I-14

Equa. No.	Description	Equation	Page No.
I.11	Earth loads on flexible culverts (psi)	$P = wH / 144$	I-14
I.12	Deflection of buried flexible conduits	$\Delta X = D_L \dfrac{K w_c r^3}{EI + 0.061\,E' r^3}$	I-15
I.13	Relationship between the horizontal and vertical deflection of buried flexible conduits	$\Delta X = 0.913\,\Delta Y$	I-15
I.14	Percent of deflection of buried flexible conduits, ($\Delta X = \Delta Y$)	$\% \dfrac{\Delta Y}{D} = \dfrac{D_L KP\,(100)}{0.149 \dfrac{F}{\Delta Y} + 0.061E'}$	I-15
I.15	Percent of deflection of buried flexible conduits, ($\Delta X = \Delta Y$)	$\% \dfrac{\Delta Y}{D} = \dfrac{D_L KP\,(100)}{[2E/3(DR-1)^3] + 0.061E'}$	I-15
I.16	Percent of deflection of buried flexible conduits with live loads considered, ($\Delta X = \Delta Y$)	$\% \dfrac{\Delta Y}{D} = \dfrac{(D_L KP + K W')\,(100)}{[2E/3(DR-1)^3]+0.061E'}$	I-16

The information contained in Appendix L: Stormwater Treatment Form A – Project Evaluation, dated September 2013 has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

Form A - Project Evaluation

Stormwater Treatment within MS4 Communities

Project Name: Project Number: Control No:

Preliminary Project Evaluation (To be completed by RSU)

Project MS4 city/county:

This project is:

Define any Water Quality Issues below (Include Stream Name):

DETERMINATION

- ○ Consideration of Treatment is Required: Form A is complete. Advance to and complete Form B
- ○ Consideration of Treatment is Unknown: Complete second half of this form for determination.
- ○ Consideration of Treatment is NOT Required. No further action needed.

RSU Staff Signature & Date

Additional Project Evaluation (To be completed by designer if indicated above. Provide a copy of this form to RSU via E-Mail and on OnBase)

Does the project disturb 1 acre or more of soil?

Is there at least 5,000 square feet of New Pavement, including bridges?

DESIGNER DETERMINATION

- ○ Consideration of Treatment is Required (Both Questions Answered YES): Advance to and complete Form B (If bridge work includes substructure modification, forward to bridge designer for further coordination).
- ○ Consideration of Treatment is NOT Required. No further action needed.

Project Designer Signature & Date

Design Unit Head Signature & Date

If there is any scope change that alters any determination on this form notify RSU so that the project may be reviewed based on the new conditions.

Instructions for Completing Form A – Project Evaluation (Taken from Chapter 3, Section 3).

3.B Preliminary Project Evaluation

Completed by **Roadside Development & Compliance Unit** (**RDC**)

3.C Final Project Evaluation

Completed by Roadway Designer and **Unit Head**

If the Preliminary Project Evaluation determines that Stormwater Treatment Facilities STFs are to be determined, the designer will perform a Final Project Evaluation of the project and shall complete the corresponding section of Form A. This should be completed early enough in the "Plan-In-Hand Phase" (See the DPO, Ref. 3.1) that conceptual STF designs can be completed and placed within the Plan-In-Hand plans. Upon completion of the Final Project Evaluation, the designer will forward Form A to his/her **Unit Head** with a recommendation for or against STFs in the project. The **Unit Head** will be responsible for reviewing the recommendation and signing off on the form. The Final Project Evaluation will consider the following items:

- **Is this project classified as 3R and has ≥ 5000 sq. ft. of New Pavement?**
  3R projects that result in the net increase of at least 5,000 square feet of New Pavement require assessment for stormwater treatment needs. Projects redeveloping non-linear facilities such as maintenance yards and rest areas which result in the net increase of at least 5,000 square feet of New Pavement or building(s) also require assessment.
- **Does the project disturb ≥ 1 acre of soil.**
  Any project which results in a land disturbance of equal to or greater than 1 acre. Land disturbance includes any areas where the bare soil will be exposed to weather for any period of time. The 1 acre value is compared to the cumulative total of exposed soil.
  - **Is the project part of a Common Plan of Development?**
    Projects that disturb less than 1 acre and are part of a larger Common Plan of Development whose total land disturbance activities are 1 acre or more are considered to meet the ≥ 1 acre disturbance criteria. In addition, the **DEQ** can designate projects as part of a common plan of development.

Upon completion of his/her review, the **Unit Head** will forward a copy of the signed Form A to the **Roadside Development & Compliance Unit** (**RDC**) and return the original to the designer. If STFs are not required to be considered for the project, the form will be closed out and placed in the project file. If STFs need to be considered for the project, the designer shall complete the "Stormwater Treatment within MS4 Communities / Form B - STFs" (Form B), included in Appendix M. Form B will be used to document the design decisions made under Section 4, "Stormwater Treatment Facility Design Process", in Chapter 3 of this manual.

3.E Change in Project Scope

Completed by Roadway Designer, **Unit Head**, and **RDC**

A change in project scope that affects one or more of the criteria or considerations in Sections 3.A and 3.B in Chapter 3 of this manual requires a re-evaluation of the project for STFs. The designer must contact the **RDC Highway Environmental Program Manager** as soon as possible. A re-evaluation will be completed by the **RDC** and **Roadway Design** and Form A will be updated to reflect any changes.

The information contained in Appendix M: Stormwater Treatment Form B – STFs, dated September 2013, has been updated to reflect the __________ Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

NDOT Stormwater Treatment within MS4 Communities

Form B - Stormwater Treatment Facilities (STFs)

Project Name: ____________________
Project Number: ____________________
Control Number: ____________________

Designer: ____________________
Unit Head: ____________________
Section Head: ____________________

MS4 Communities: ____________________
____
____
____

Unit Head Approval: ____________________
Date: ____________________

(Signed following completion of Form B - Stormwater Treatment Facilities)

Instructions

1 Fill out the above information on the "Information & Instructions" tab and read the "Instructions" and "Read Me" sections.

2 On the "Form B" tab, gray shaded cells accept user input. Unshaded cells are automatically calculated or populated.

3 Identify and document every outfall or treatment point and select "Yes" or "No" (from the dropdown menu) whether it is a priority outfall. See comment in column heading for definition of priority outfalls.

4 For each priority outfall, identify and document all outfalls or treatment points contributing to the priority outfall. (Note: The priority outfall itself may be the treatment point, or it may have several treatment points contributing to it.)

5 Add new outfalls using the "Add New Outfall" button.

6 Enter the area of "New Pavement" that discharges runoff to the outfall/treatment point.

7 Enter the area of inseparable, untreated "Run-On" that intermingles with "New Pavement" runoff before it is treated, and provide a description.

8 Required design outputs are calculated for up to 5 acres of drainage area. For larger areas, see NDOT Drainage Manual Chapter 3 methods.

9 Design STF and document ("Yes" or "No", from dropdown menu) whether you achieved treatment for the calculated treatment drainage area (TDA).

10 Identify the STF designed, choosing from the drop-down menu.

11 After documenting every outfall and treatment point, check your current credit balance. If green, you have met treatment requirements; if red, provide more treatment and consult RSU or Roadway Design Hydraulics.

Read Me

1 This workbook requires macros to be enabled. If macros are not enabled, this worksheet will not function.

2 Many cells and sheets in this workbook are protected. To unprotect/protect a sheet, go to "Review" and "Protect/Unprotect Sheet". The password is "NDOR".

3 Any changes made to worksheets while unprotected may cause the workbook to malfunction. It is the user's responsibility to maintain a functional workbook.

Appendix N: Distance Required to be Exceeded by an Ephemeral Drainage to Exclude a Stormwater Outfall from Priority Status [1]

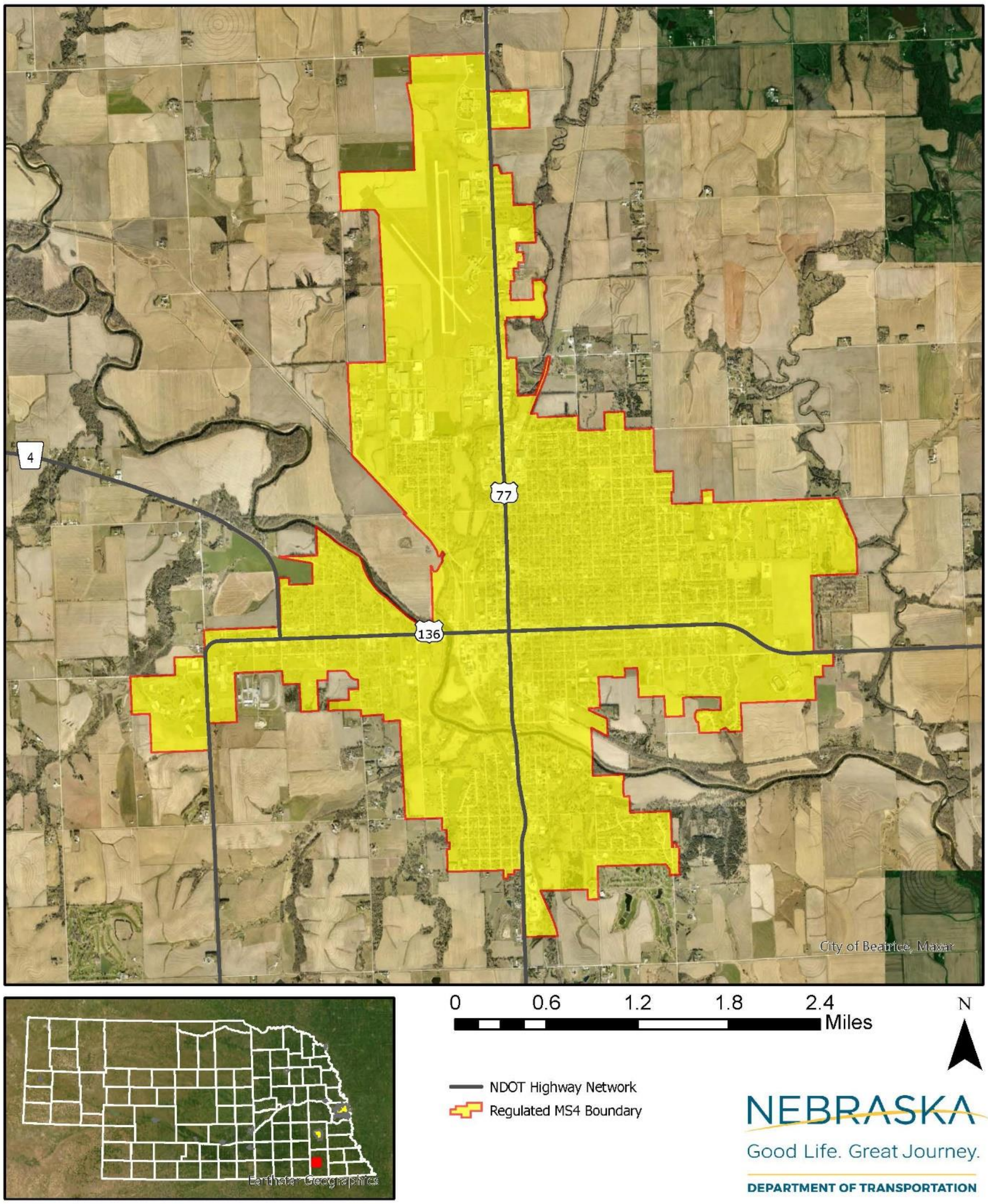
Vegetated Ephemeral Drainage												
Swale		Water Quality Volume Discharge Rate(cfs)										
Width (ft)	Grade (%)	0.2	0.45	0.9	1.3	1.8	2.2	2.7	3.1	3.6	4	4.4
15	0.5	150	150	150	150	150	175	200	225	250	250	275
	1	150	150	150	175	200	225	250	275	300	325	350
	1.5	150	150	150	175	225	250	300	325	350	-	-
	2	150	150	150	200	250	275	325	-	-	-	-
	2.5	150	150	175	225	275	300	-	-	-	-	-
	3	150	150	175	225	275	325	-	-	-	-	-
	4	150	150	200	250	325	-	-	-	-	-	-
	5	150	150	200	275	-	-	-	-	-	-	-
10	0.5	150	150	150	150	200	225	250	275	300	325	350
	1	150	150	175	200	250	275	325	350	-	-	-
	1.5	150	150	175	225	275	325	-	-	-	-	-
	2	150	150	200	250	325	-	-	-	-	-	-
	2.5	150	150	225	275	-	-	-	-	-	-	-
	3	150	150	225	300	-	-	-	-	-	-	-
	4	150	150	250	-	-	-	-	-	-	-	-
	5	150	175	275	-	-	-	-	-	-	-	-
5	0.5	150	150	175	225	275	325	375	450	500	550	-
	1	150	150	225	275	350	-	-	-	-	-	-
	1.5	150	175	275	325	-	-	-	-	-	-	-
	2	150	200	300	-	-	-	-	-	-	-	-
	2.5	150	200	325	-	-	-	-	-	-	-	-
	3	150	225	-	-	-	-	-	-	-	-	-
	4	150	250	-	-	-	-	-	-	-	-	-
	5	150	275	-	-	-	-	-	-	-	-	-
2	0.5	150	175	250	350	450	550	650	725	-	-	-
	1	150	225	300	375	-	-	-	-	-	-	-
	1.5	150	250	-	-	-	-	-	-	-	-	-
	2	175	275	-	-	-	-	-	-	-	-	-
	2.5	200	300	-	-	-	-	-	-	-	-	-
	3	200	-	-	-	-	-	-	-	-	-	-
	4	225	-	-	-	-	-	-	-	-	-	-
	5	250	-	-	-	-	-	-	-	-	-	-
0	0.5	150	250	450	550	725	800	950	-	-	-	-
	1	175	300	500	-	-	-	-	-	-	-	-
	1.5	225	325	-	-	-	-	-	-	-	-	-
	2	225	-	-	-	-	-	-	-	-	-	-
	2.5	275	-	-	-	-	-	-	-	-	-	-
	3	275	-	-	-	-	-	-	-	-	-	-
	4	-	-	-	-	-	-	-	-	-	-	-
	5	-	-	-	-	-	-	-	-	-	-	-
Non-Vegetated Emphemeral Drainage												
Swale		Water Quality Volume Discharge Rate(cfs)										
Width (ft)	Grade (%)	0.2	0.45	0.9	1.3	1.8	2.2	2.7	3.1	3.6	4	4.4
All	All	-	-	-	-	-	-	-	-	-	-	-

[1] Distances given in feet. A dash indicates no distance is acceptable for excluding the Stormwater Outfall as a priority.

REGULATED MS4 COMMUNITIES IN NEBRASKA

NDOT MS4 Regulated Boundary

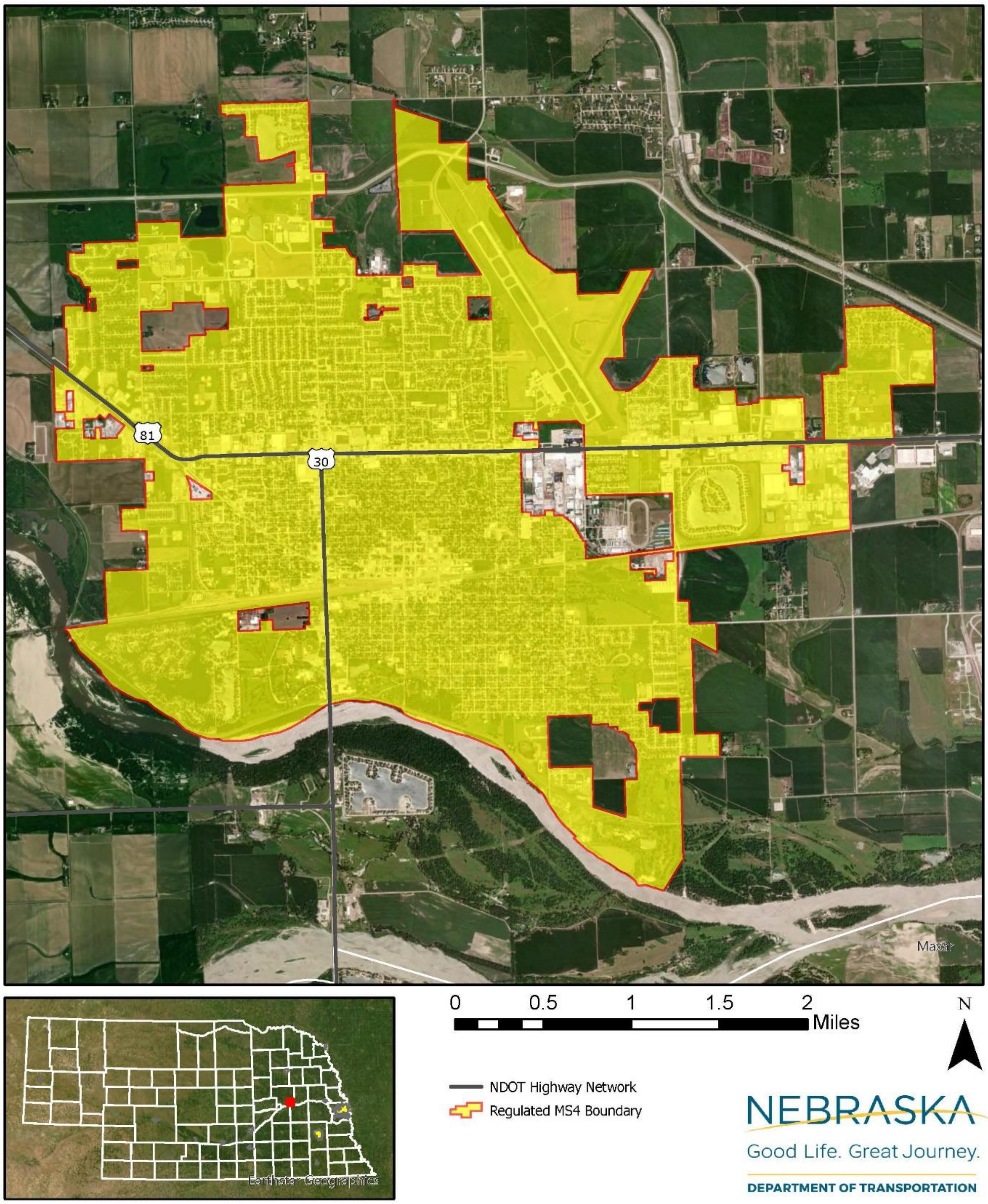
Beatrice, Nebraska - Gage County



NEBRASKA
Good Life. Great Journey.
DEPARTMENT OF TRANSPORTATION

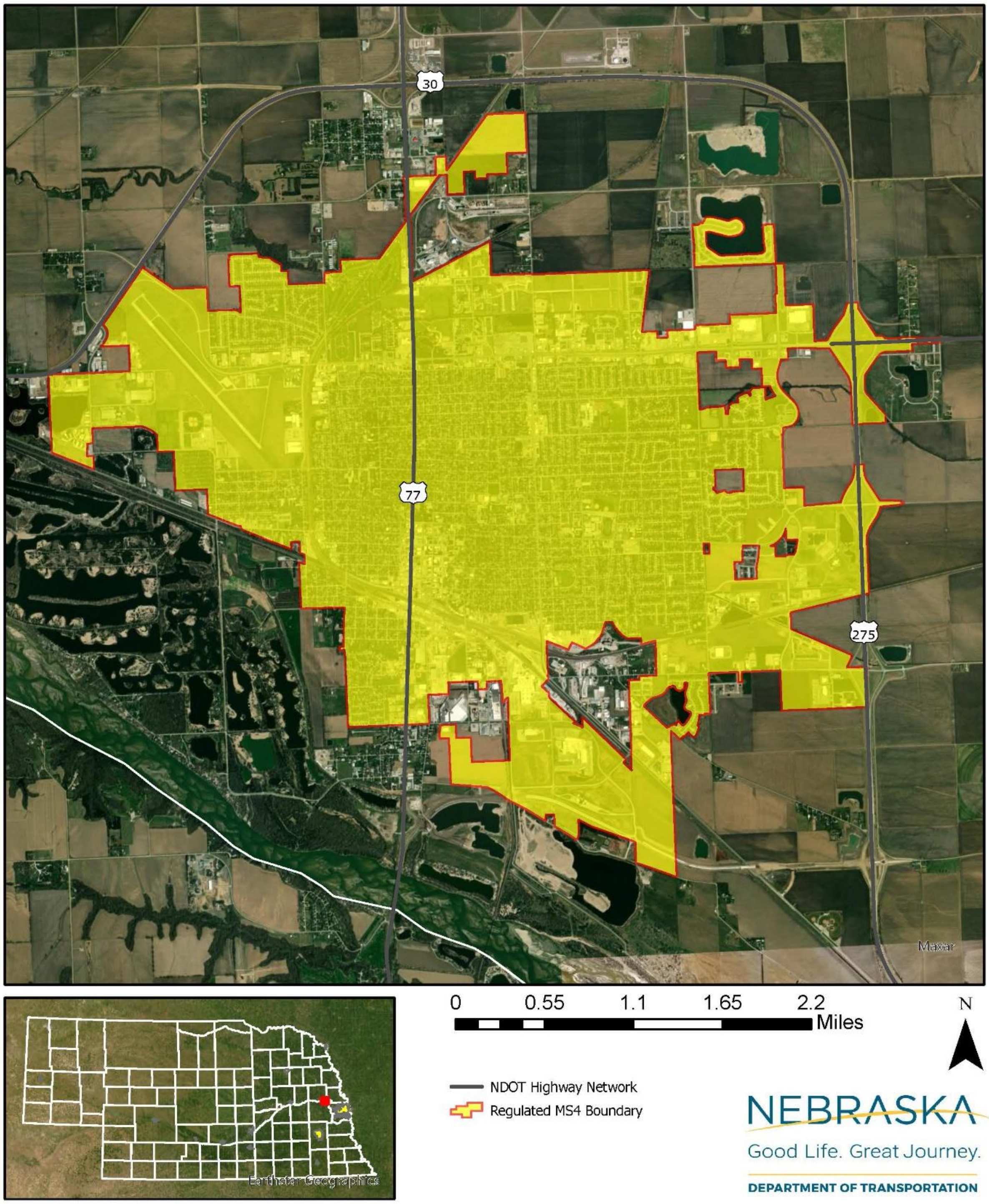
NDOT MS4 Regulated Boundary

Columbus, Nebraska - Platte County



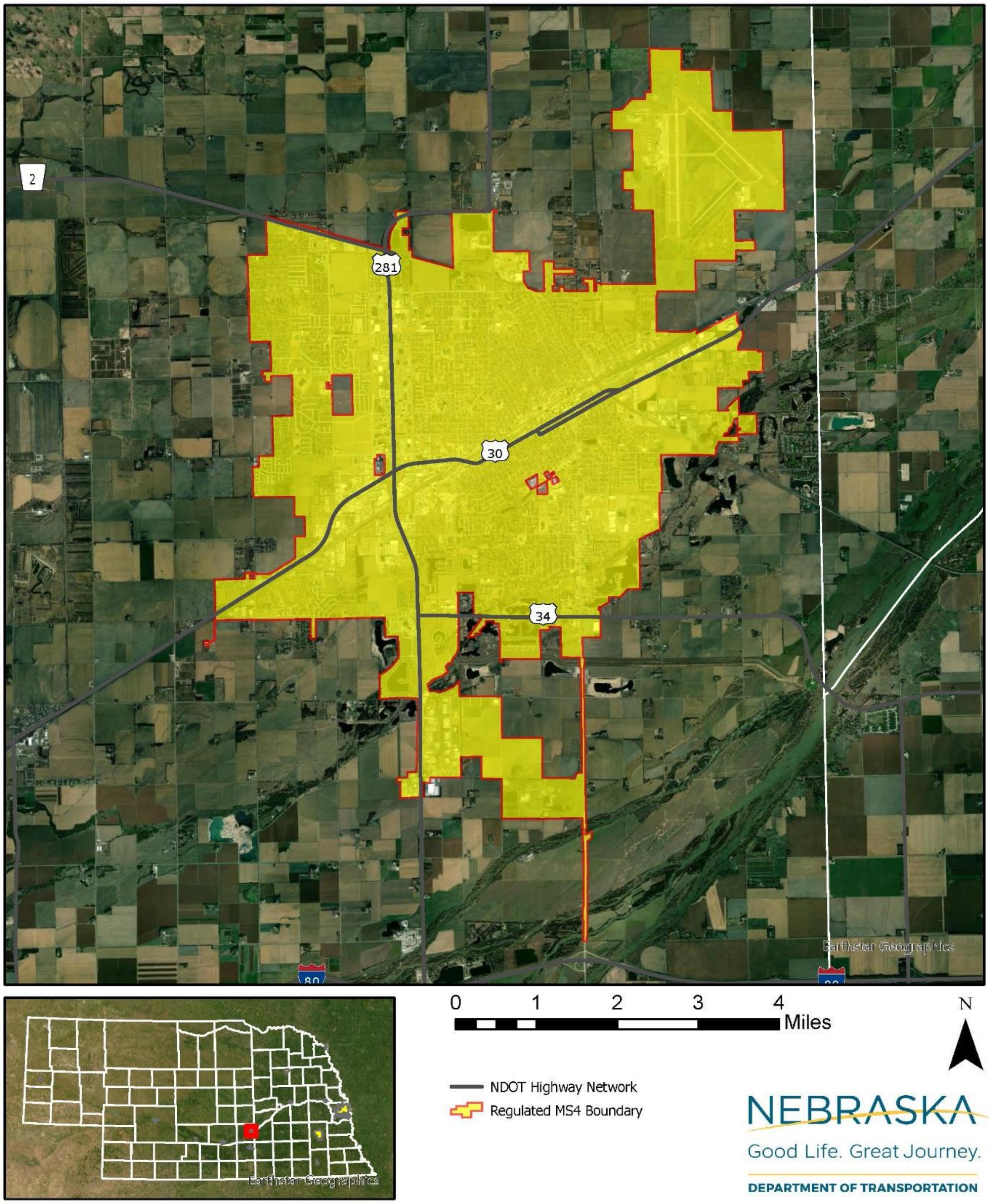
NDOT MS4 Regulated Boundary

Fremont, Nebraska - Dodge County



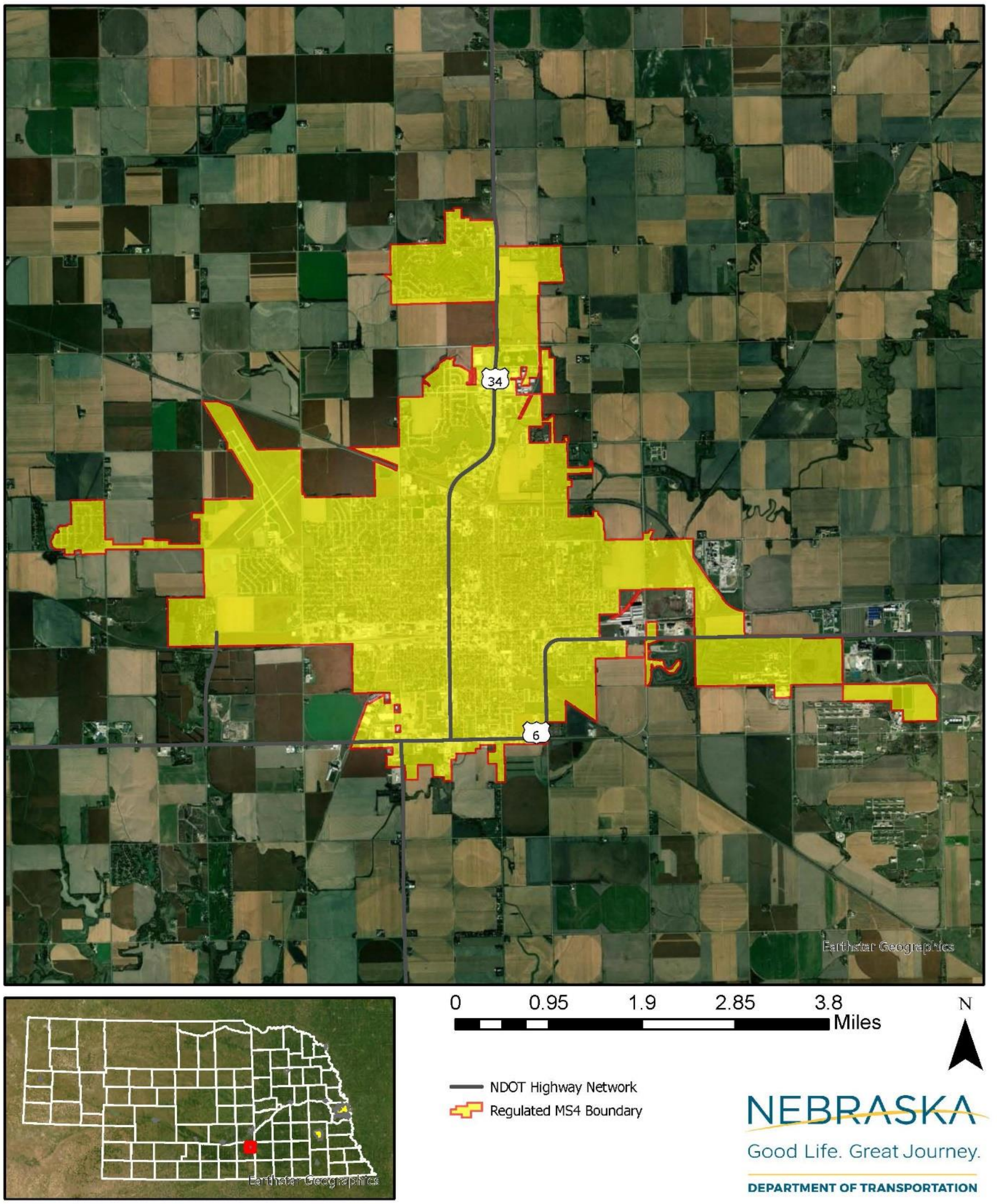
NDOT MS4 Regulated Boundary

Grand Island, Nebraska - Hall County



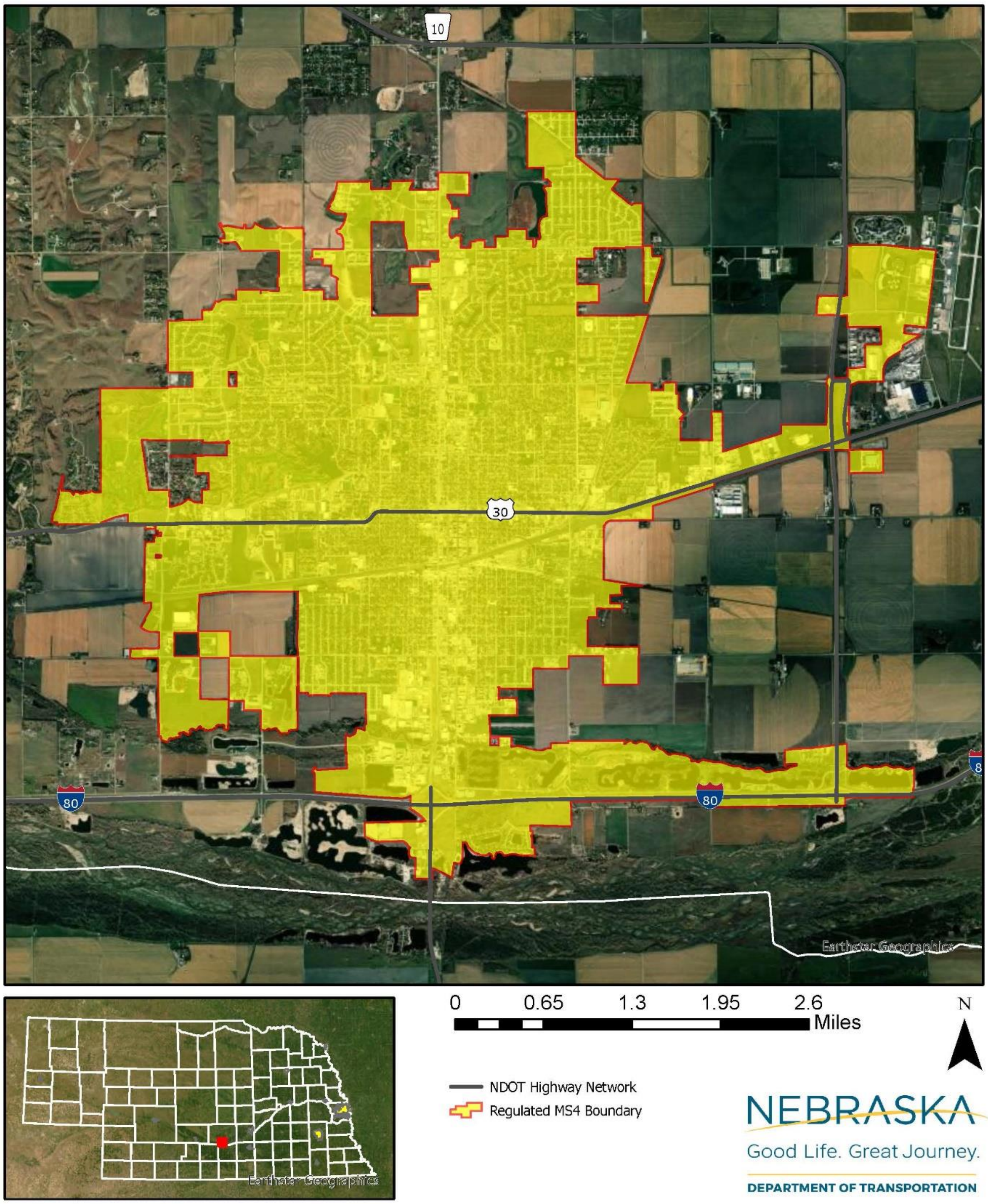
NDOT MS4 Regulated Boundary

Hastings, Nebraska - Adams County



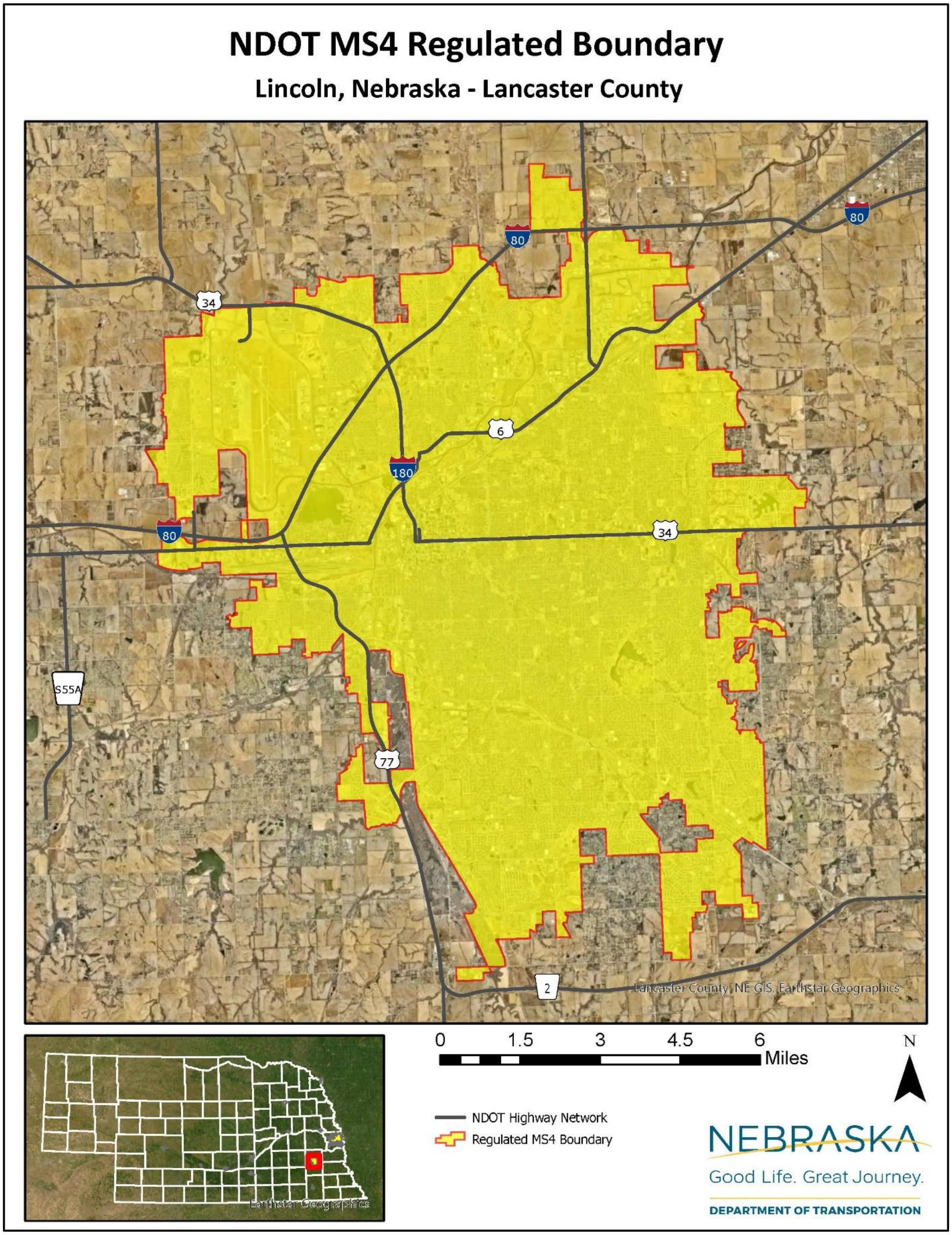
NDOT MS4 Regulated Boundary

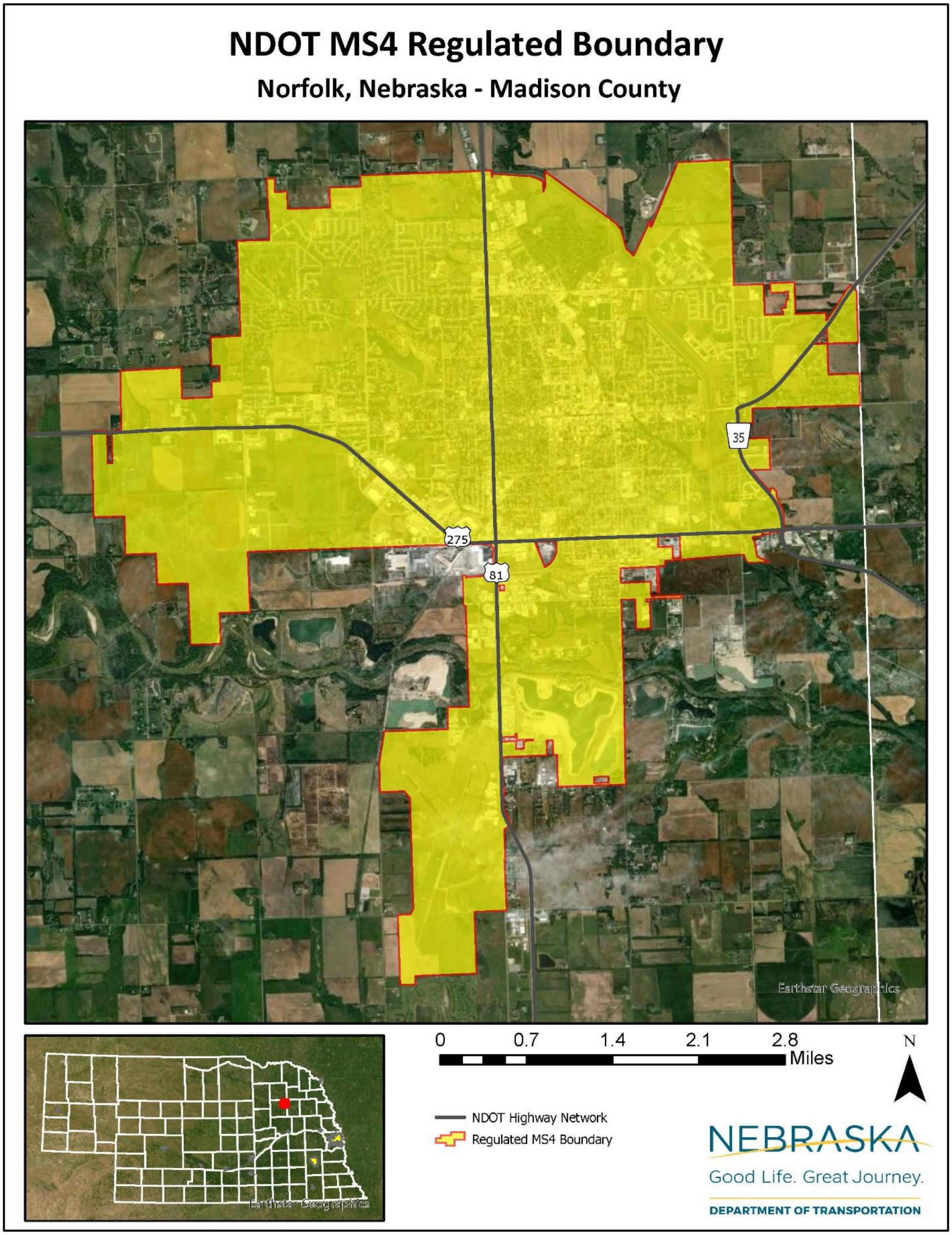
Kearney, Nebraska - Buffalo County

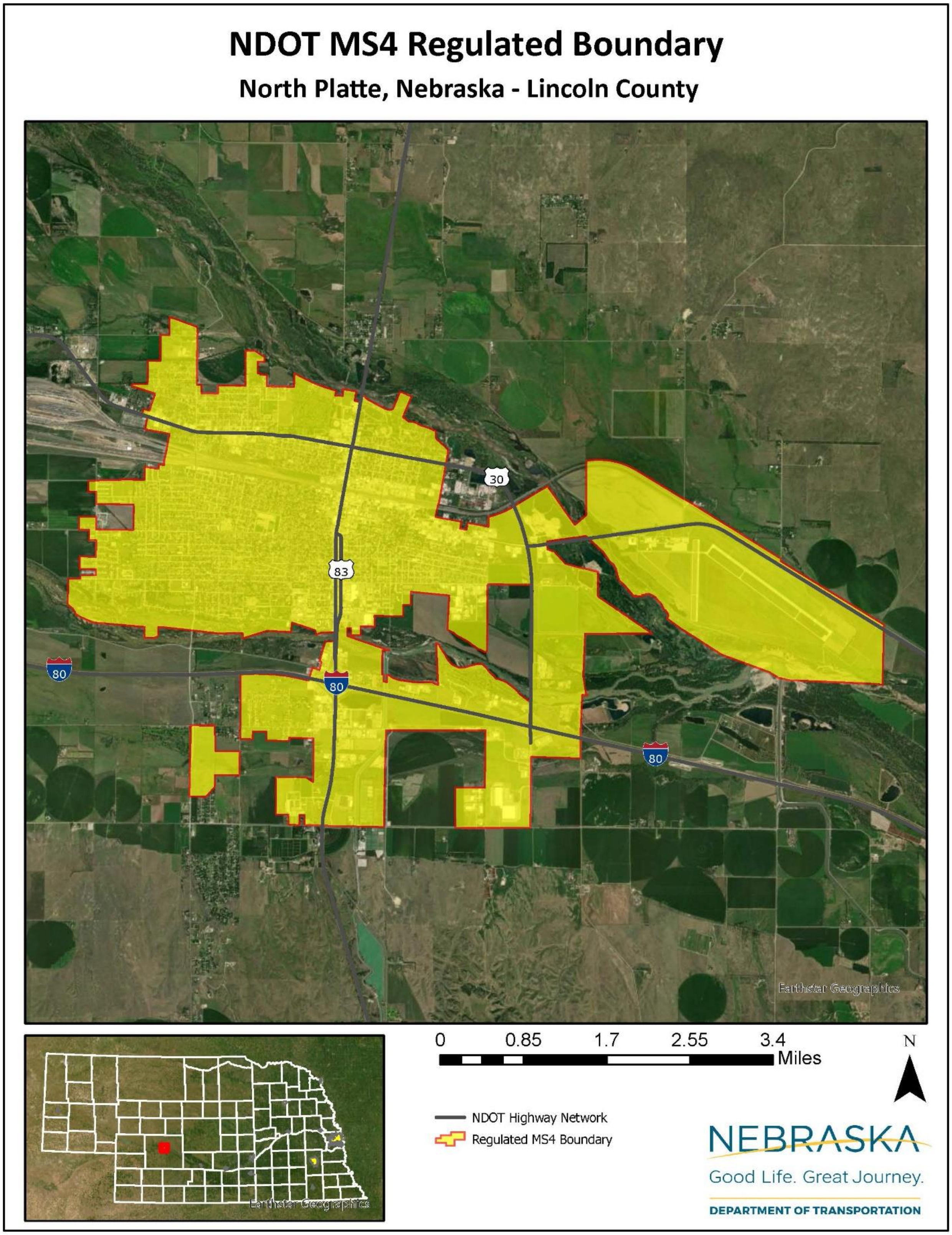


NDOT MS4 Regulated Boundary

Lexington, Nebraska - Dawson County

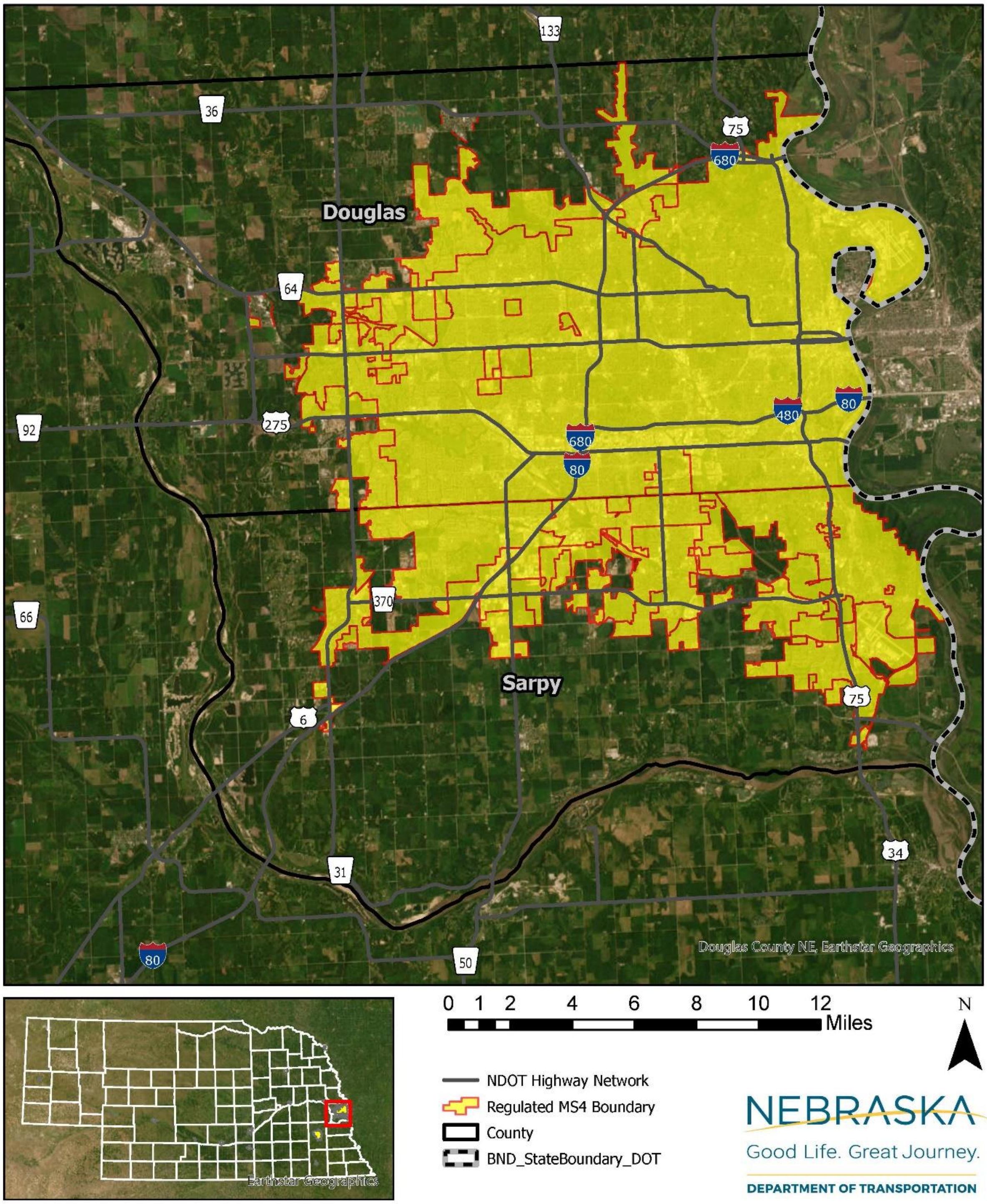

NDOT MS4 Regulated Boundary
Lincoln, Nebraska - Lancaster County
80
34
6
180
80
34
S55A
77
2
Lancaster County, NE GIS, Earthstar Geographics
Earthstar Geographics
0
1.5
3
4.5
6
Miles
N
NDOT Highway Network
Regulated MS4 Boundary
NEBRASKA
Good Life. Great Journey.
DEPARTMENT OF TRANSPORTATION

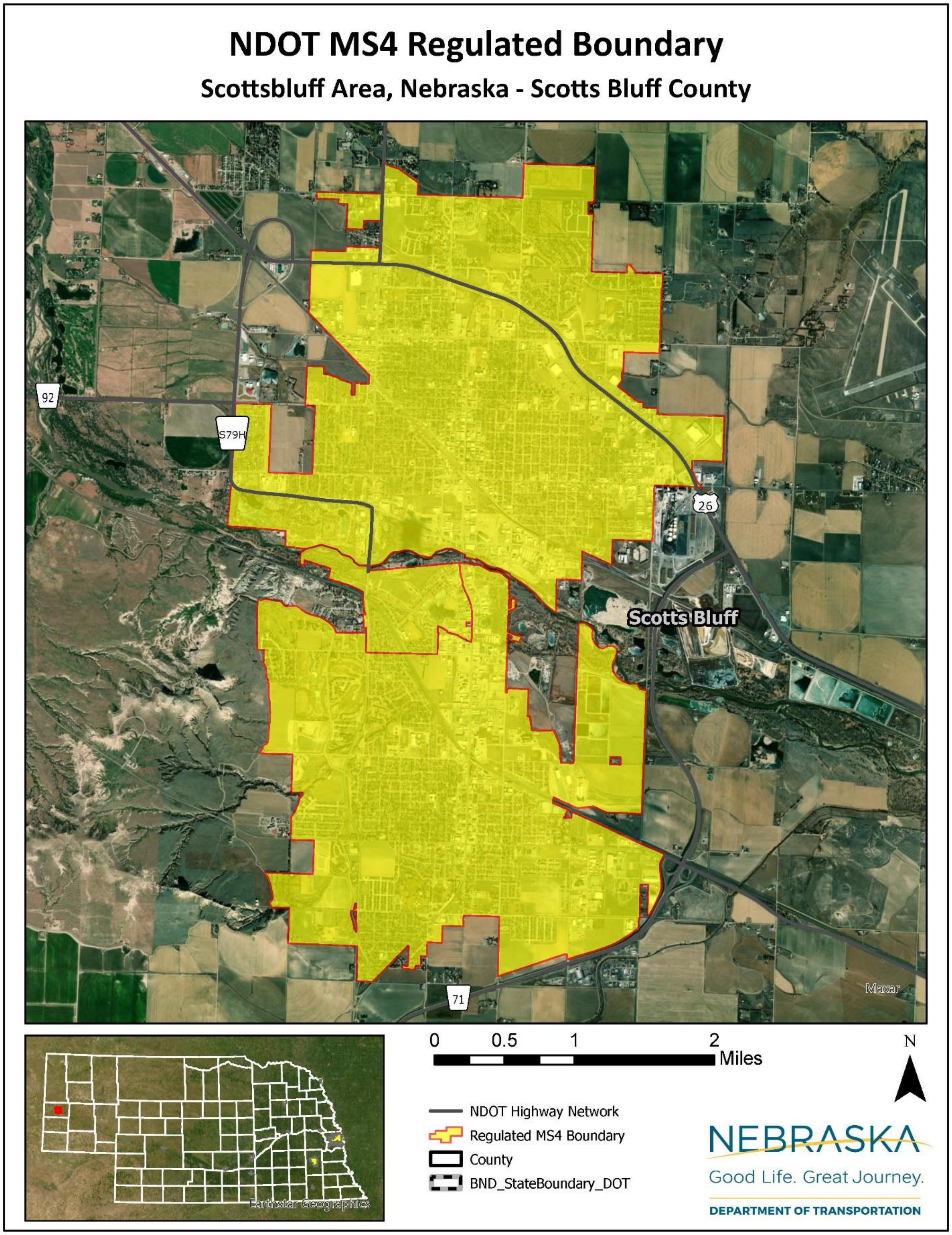

NDOT MS4 Regulated Boundary
Norfolk, Nebraska - Madison County
35
275
81
Earthstar Geographics
0
0.7
1.4
2.1
2.8
Miles
N
NDOT Highway Network
Regulated MS4 Boundary
NEBRASKA
Good Life. Great Journey.
DEPARTMENT OF TRANSPORTATION


NDOT MS4 Regulated Boundary
North Platte, Nebraska - Lincoln County
30
83
80
80
80
Earthstar Geographics
Earthstar Geographics
0
0.85
1.7
2.55
3.4
Miles
N
NDOT Highway Network
Regulated MS4 Boundary
NEBRASKA
Good Life. Great Journey.
DEPARTMENT OF TRANSPORTATION

NDOT MS4 Regulated Boundary

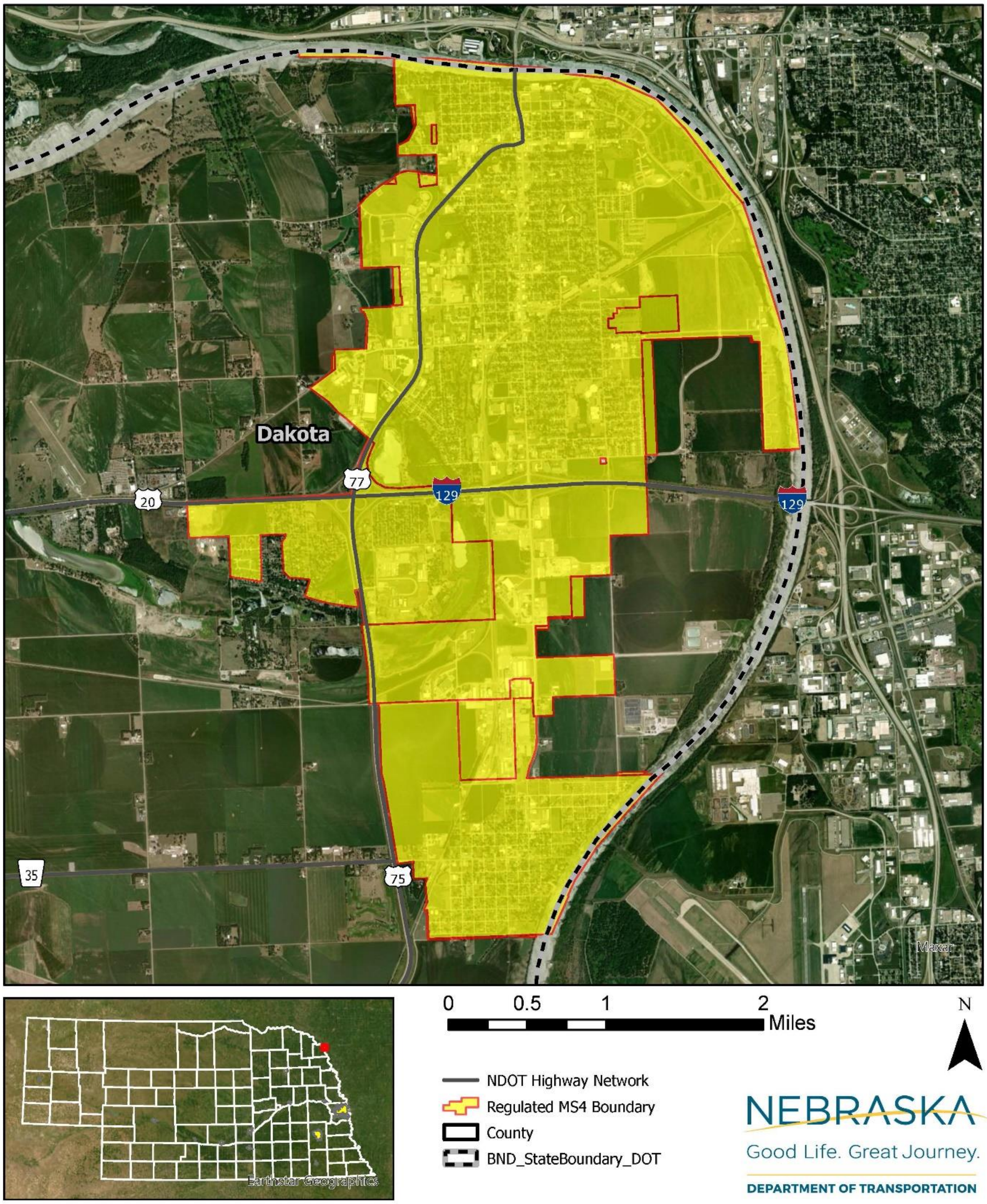
Omaha Metro Area, Nebraska - Douglas & Sarpy County




NDOT MS4 Regulated Boundary
Scottsbluff Area, Nebraska - Scotts Bluff County
92
S79H
26
Scotts Bluff
71
Maxar
Earthstar Geographics
0
0.5
1
2
Miles
N
NDOT Highway Network
Regulated MS4 Boundary
County
BND_StateBoundary_DOT
NEBRASKA
Good Life. Great Journey.
DEPARTMENT OF TRANSPORTATION

NDOT MS4 Regulated Boundary

South Sioux City Area, Nebraska - Dakota County



STF DESIGN GUIDELINES

Vegetated Filter Strip

OVERVIEW

Source: Nebraska Department of Transportation (NDOT)

Definition

Vegetated filter strips are strips of dense vegetation, typically grass, designed to filter and infiltrate sheet flow from upgradient development. Vegetated filter strips can be located between a pavement surface and a surface water collection system, such as a swale, pond, or river, or as pre-treatment to downstream stormwater STFs.

Benefits

- Filters sediment and trash.
- Provides green space available for snow storage.
- Has low design, installation, and maintenance costs.
- Can be used on roadside embankments, reducing additional right-of-way needs.

Overview Table

Associated Costs	L	M	H
Design	X		
Construction	X		
Maintenance	X		
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients		X	
Heavy Metals		X	
Hydrocarbons		X	

Limitations

- Dense vegetated cover is required to remove pollutants and protect from erosion.
- Space for filter strips is often limited in urban areas.
- There is limited treatment on steep slopes or large impervious areas with high velocity runoff.
- Vegetated filter strips are effective only where runoff entering and flowing through the strip remains as sheet flow.

Vegetated Filter Strip

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

A vegetated filter strip is a densely vegetated strip, usually consisting of grasses, designed to accept sheet flow from upgradient development and filter and infiltrate stormwater. Filter strips are different from grass swales in that they are designed to accommodate overland (sheet) flow rather than channelized or concentrated flow.

STF COMPONENTS

Soils – The soil(s) of concern for the Vegetated Filter Strip STF are from finished grade to a depth of three feet. Also of concern is the level of compaction that these soils have, or undergo, during construction. The type and condition of the soils impact the rate of infiltration and partially determine the density of vegetation. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics. Note any mixed and compacted soils for variability in soil characteristics.

Slope – Vegetated filter strips are designed to have enough slope to keep water moving across the strip and avoid ponding but not too much slope so that the water moves too quickly. The filter strip is uniformly graded to maintain sheet flow.

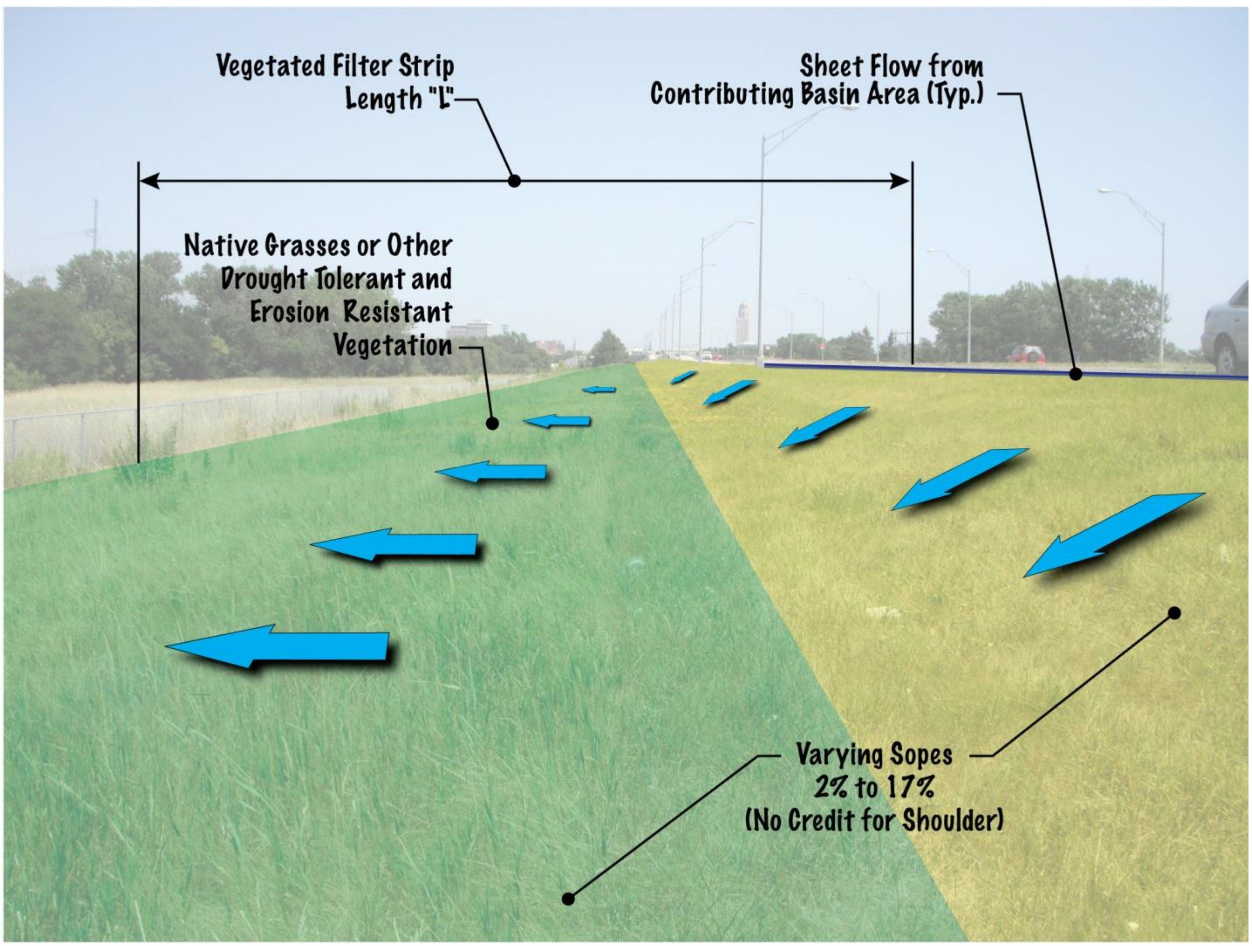
Vegetation – The purpose of the vegetation is to effectively bind the soil, reduce velocities, and uptake water and some nutrients. A dense stand of vegetation that is drought tolerant, durable, and erosion resistant is desired. Salt tolerance, water, and nutrient needs should also be considered when selecting vegetation. The selection of seed, sod, and plantings will generally fall under the responsibility of the Roadside Development & Compliance Unit although there may be exceptions. Refer to NDOT's *Plan for the Roadside Environment*, June 2008, for region-specific information on vegetation.

Length – The length of the vegetated filter strip is measured in the direction of flow and depends on the slope of the filter strip and Manning's coefficient.

Width – The width of the vegetated filter strip is measured perpendicular to flow and is determined by the upgradient drainage area and the ability to distribute the flow as uniform sheet flow to the filter strip.

Level Spreader – A level spreader is a stormwater feature that receives concentrated, potentially erosive runoff and releases it as sheet flow uniformly over a stabilized area.

Vegetated Filter Strip



Vegetated Filter Strip Example
Not to Scale

Vegetated Filter Strip

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Can I distribute flow evenly across the filter strip?

Q. Is a level spreader needed to help distribute flow?

Q. What is the slope of the proposed surface for the vegetated filter strip?

Q. What types of soils are on the site and are they compacted?

Site characteristics are very important when designing vegetated filter strips and should be taken into consideration early in the design process.

Sheet flow, distributed evenly across and through the filter strip, is required to effectively treat runoff. Because higher runoff rates and increased depths reduce the effectiveness of treatment, the drainage area upgradient of the vegetated filter strip is limited.

Vegetated filter strips should be used only where the topography of the site allows appropriate slope and sheet flow can be distributed evenly across the filter strip. Grading and level spreaders are often used to create a uniformly sloping area to distribute runoff evenly across the length of the strip.

Vegetated roadway shoulders adjacent to a vegetated filter strip should be graded below the pavement surface at the edge of the roadway to allow for future sediment deposition and still maintain sheet flow into the filter strip. The length of vegetated roadway shoulders, measured in the direction of flow, is designed according to minimum roadway design standards but should not be included in calculating the vegetated filter strip length.

Where flows are concentrated, a level spreader should be used to distribute flows evenly across the width of the vegetated filter strip. One type of level spreader is an excavated trench running along the upgradient end of the vegetated filter strip that is filled with pea-gravel or other free-draining aggregate. For aggregate gradations with a maximum size equal to or greater than ½ inch, a non-woven filter fabric is needed along the sides and bottom of a trench to reduce the potential for migration of fines into the aggregate. The surface of the trench should be level.

The length of the vegetated filter strip depends on the slope of the filter strip. Charts and equations have been provided to determine the minimum length. The filter strip slope should be greater than 2 percent so that flow is conveyed away from the source but less than 17 percent (or 1V:6H) to keep velocities fairly low. Keep in mind that steeper slopes are less effective at treatment and will require a longer length to provide the necessary removals. Additionally, the charts and equations assume that surface soils are not compacted and have the ability to infiltrate stormwater.

Soils that have been compacted will need to be loosened to improve infiltration and increase water quality treatment effectiveness. The depth that should be disced or tilled is based on the estimated depth of compaction. Areas that have been disturbed as a result of construction traffic and staging should be disced or tilled along with any fill areas. Finished grades should be amended with topsoil.

Vegetated Filter Strip

DESIGN CRITERIA

Description	Value
Maximum Length of Contributing Pavement in the Direction of Flow	75 feet (Upgradient from the Vegetated Filter Strip) (Based on maximum flow rate of ** per foot of filter strip width)
Slope	2% - 17%
Minimum Length (direction of flow)	20 feet (see graph provided below)
Maximum Length (direction of flow)	100 feet (channelization concerns beyond this)
Minimum Width (perpendicular to flow)	Typically, the same as adjacent contributing basin area
Maximum Flow Depth	0.1 feet

DESIGN PROCEDURE

Step 1: Calculate the maximum discharge loading rate

Modify Manning's equation to calculate maximum discharge loading per foot of filter strip width (measured perpendicular to flow).

$$Q = 1.486/n * R^{2/3} * S^{1/2} * A$$

Q = Discharge rate (cfs)
A = flow area (ft^2) = wD (approximate for shallow flow)
D = flow depth (ft) (maximum allowable depth 0.1 feet)
n = Manning's coefficient (for sheet flow)

- n = 0.15 for Short Grass Prairie
- n = 0.24 for Dense Grass

R = hydraulic radius (ft) = A/WP
WP = Wetted Perimeter = w (approximate for shallow flow)
w = width (ft) (perpendicular to flow)
S = slope (ft/ft) (in the direction of flow)

Simplify the equation using the assumptions provided above to solve for maximum discharge loading rate:

$$q = \frac{Q}{w} = \frac{1.486 * D^{5/3} * S^{1/2}}{n}$$

q = maximum discharge loading rate per foot of filter strip (cfs/ft)

Step 2: Calculate Water Quality Volume Discharge Rate (Q_{WQ})

Calculate Q_{WQ} using the NRCS Curve Number (CN) Procedure as described in Chapter 3 of NDOT's Drainage *Design and Erosion Control Manual* **OR** limit the length of contributing pavement to 75 feet as provided in the design criteria table.

Step 3: Calculate the minimum width of vegetated filter strip and level spreader (if needed)

$$w_{min} = \frac{Q_{WQ}}{q}$$

w_{min} = minimum width of filter strip (ft)
(and level spreader if needed)

Step 4: Calculate the length of vegetated filter strip

The length of vegetated filter strip, measured in the direction of flow, is based on the following travel time equation. A graph is provided as well to help determine the minimum filter strip length.

$$L_f = \frac{T_t^{5/4} * P_{WQ}^{5/8} * S_{\%}^{1/2}}{3.34 * n}$$

L_f = minimum length of vegetated filter strip (ft)
T_t = travel time through filter strip (5 min)
P_{WQ} = rainfall depth (0.75 inch)
(P_{WQ} based on criteria for determination of Q_{WQ} as described in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*)
$S_{\%}$ = slope (percent)
n = Manning's coefficient

- n = 0.15 for Short Grass Prairie
- n = 0.24 for Dense Grass

For vegetated filter strips with multiple slopes, the designer should determine the travel time through the upgradient slope (T_{t1}) using the available length of the upgradient filter strip (L_{f1}).

$$T_{t1} = ((L_{f1} * 3.34 * n)/(P_{WQ}^{5/8} * S_{\%}^{1/2}))^{4/5}$$

T_{t1} = travel time through upgradient filter strip (min)
L_{f1} = length of upgradient filter strip (ft)

If the time is less than 5 minutes, then calculate the length of the downgradient slope (L_{f2}) using the remaining time (T_{t2}) in the equation above. Solve for the remaining time for water quality treatment:

$$T_{t2} = T_t – T_{t1}$$

T_{t2} = travel time through downgradient filter strip (min)

Vegetated Filter Strip

Solve for the minimum length of the downgradient filter using the equation provided above:

$$L_{f2} = \frac{T_t^{5/4} * P_{WQ}^{5/8} * S_{\%}^{1/2}}{3.34 * n}$$

L_{f2} = minimum length of down-gradient filter strip (ft)

Use the following graph, together with the proposed vegetated filter strip slope and Manning's coefficient, to determine the minimum filter strip length, L_f. For vegetated filter strips with multiple slopes, the graph may be used to calculate L_{f2}.

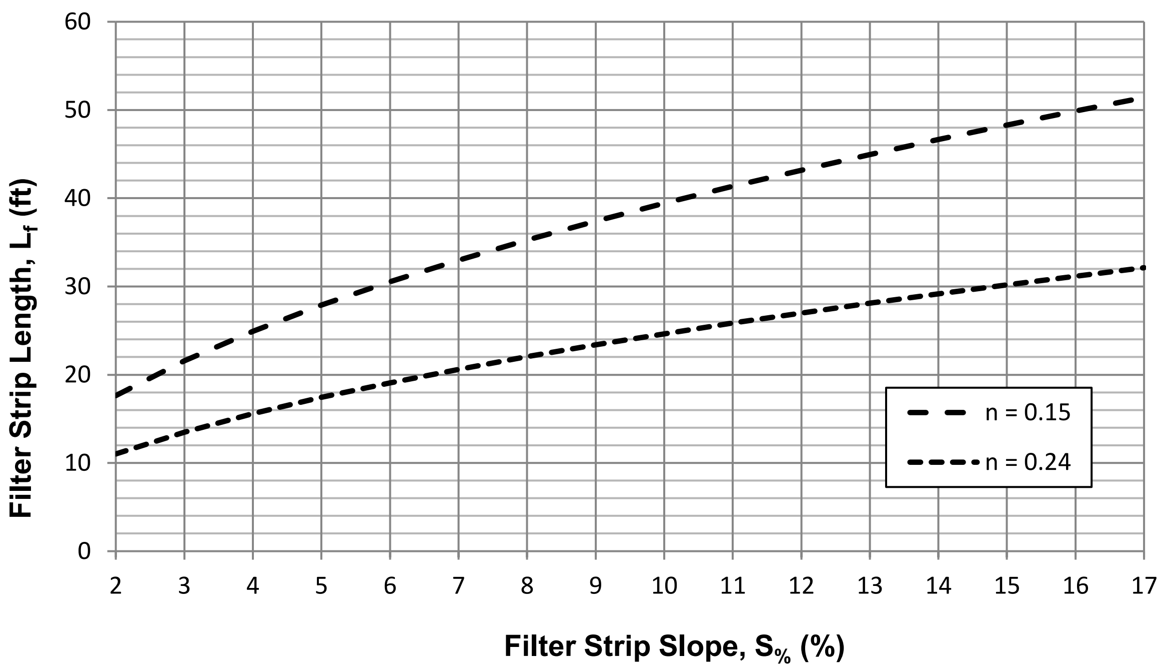
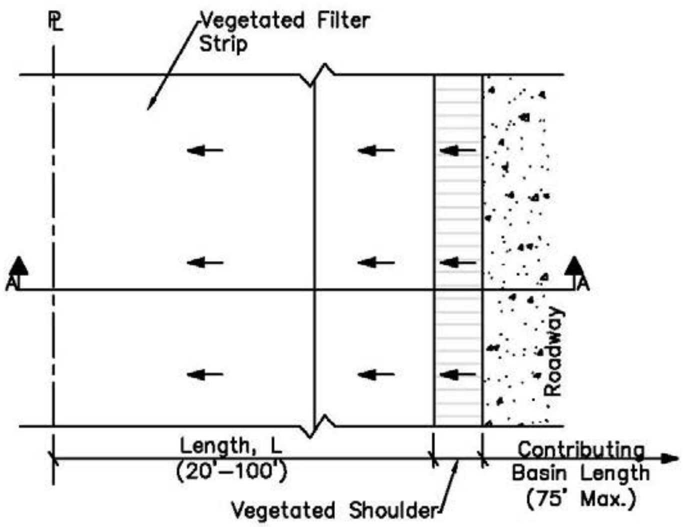


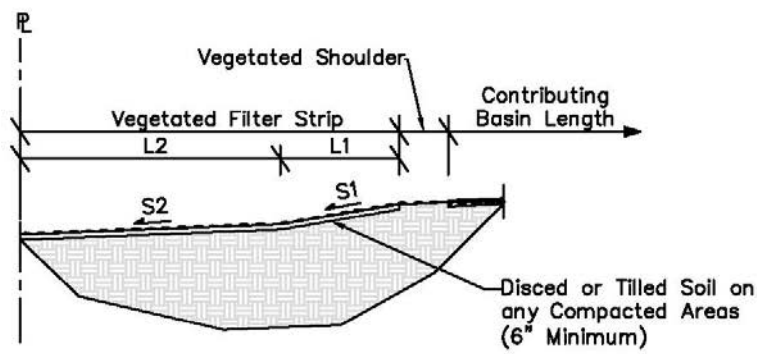
Figure 1. Vegetated Filter Strip Length as a Function of Slope and Manning's Coefficient (Using T_t = 5 minutes)

Vegetated Filter Strip

DESIGN EXAMPLE

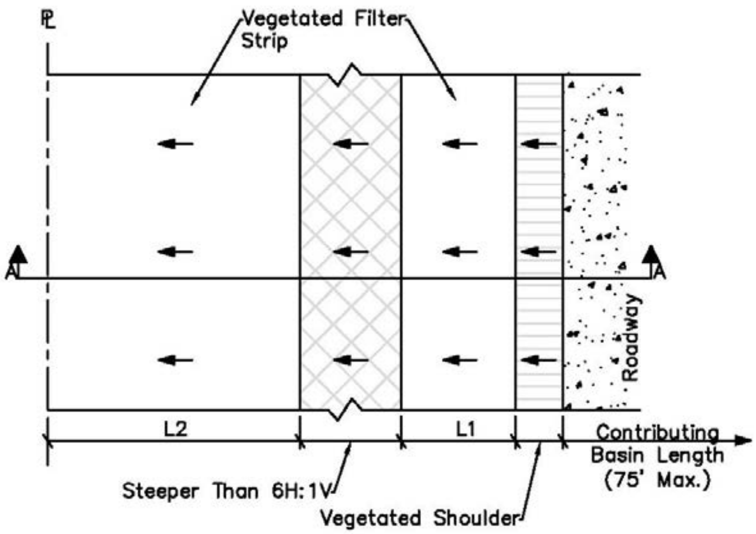


Vegetated Filter Strip
Plan View
No Scale

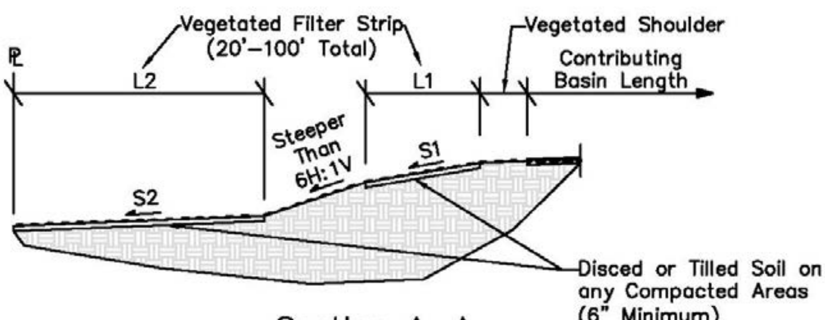


Section A–A
No Scale

Vegetated Filter Strip

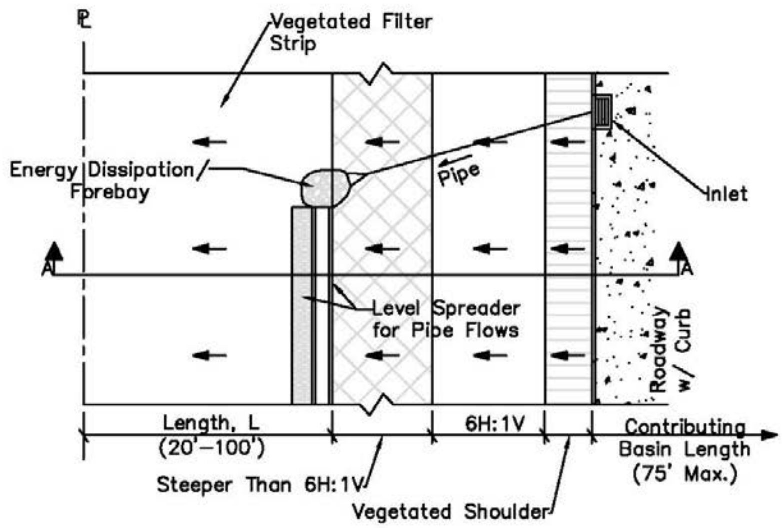


Vegetated Filter Strip
(6H:1V Into Steeper Section)
Plan View
No Scale

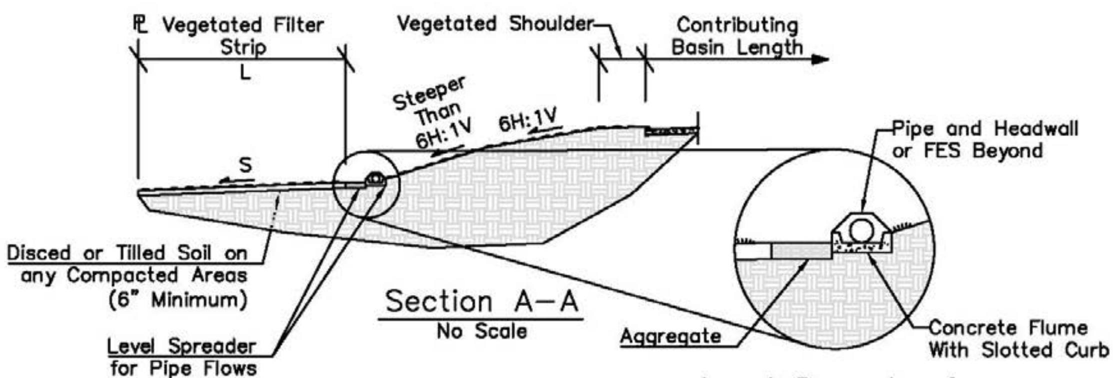


Section A–A
No Scale

Vegetated Filter Strip



Vegetated Filter Strip
(with Level Spreader for Pipe Flows)
Plan View
No Scale



Section A–A
No Scale

Level Spreader for Pipe Flows
No Scale

CONSTRUCTION CONSIDERATIONS

For vegetated filter strips, the following should be considered during design, as well as added into design plans and specifications as necessary.

- Avoid over compaction of soils in the filter strip during construction to preserve infiltration activities.
- Because the final grade of the filter strip is critical, inspect the filter area before placing seed or sod to ensure appropriate grading. Oftentimes, following soil amendment and placement of sod, the final grade is too high to accept sheet flow.
- Install filter strips at a time of year when there is a reasonable chance of successful establishment without irrigation. Rainfall in any given year may not be sufficient and temporary irrigation may be required.
- Implement erosion controls to protect seeds for at least 75 days after the first rainfall of the season.
- Maintain erosion and sediment control measures on upgradient disturbed areas to prevent excessive sediment loading to filter strip.
- If sod is used, place sod tiles so that there are no gaps in between tiles. Stagger the ends of the tiles to prevent formation of channels along the strip.
- Perform soil amending, fine grading, and seeding only after contributing drainage areas have been stabilized and any work crossing the filter has been completed.
- Include directions in the specifications for use of appropriate fertilizer and soil amendments.
- Delineate undisturbed natural areas of vegetation that have been identified on the plans with flagging before beginning construction activities.
- Ensure that other sediment control measures to be used in conjunction with the filter strip are in place and functioning properly.
- Minimize construction activities and traffic in the filter strip and immediate surrounding areas.
- Consider the timing of permanent seeding and whether or not temporary irrigation may be needed to help establish seeds.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for vegetated filter strips include providing litter control, keeping up the hydraulic and removal efficiency of the filter strip by repairing erosion and removing sediments or other obstructions, and maintaining a dense, healthy grass cover.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect filter strip to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect filter strip for erosion and damage by equipment or vehicles after every major rainfall event. Repair as needed. • Remove trash and debris accumulated in the filter strip.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect filter strip to ensure there is a dense, uniform stand of the intended vegetation. Reseed as needed. • Mow grass to control weeds. • Inspect filter strip for erosion and damage by equipment or vehicles. Repair as needed. • Inspect filter strip for sediment buildup, particularly along edge of pavement, and ponding or obstructions to ensure uniform sheet flow. Remove sediment once it has accumulated. • Inspect any level spreaders for uniformity, clogging, and sediment buildup. Replace first layer of aggregate if clogging appears to be only at the surface. • Remove trash and debris accumulated in the filter strip.
Annually (After NPDES Permit is closed)	• Inspect filter strip to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect filter strip for erosion and damage by equipment or vehicles. Repair as needed. • Inspect filter strip for sediment buildup, particularly along edge of pavement, and ponding or obstructions to ensure uniform sheet flow. Remove sediment once it has accumulated. • Inspect any level spreaders for uniformity, clogging, and sediment buildup. Replace first layer of aggregate if clogging appears only to be at the surface. • Remove trash and debris accumulated in the filter strip.

Vegetated Filter Strip

RESOURCES AND REFERENCES

Arizona Department of Transportation, *ADOT Post-Construction Best Management Practices Manual.* July 2009.

Atlanta Regional Commission. *Georgia Stormwater Management Manual – Volume 2: Technical Handbook*. August 2001.

California Department of Transportation. *BMP Retrofit Pilot Program – Final Report.* January 2004.

Iowa State University – Institute for Transportation. *Iowa Stormwater Management Manual.* October 2009.

Minnesota Pollution Control Agency. *Protecting Water Quality in Urban Areas - Best Management Practices for Dealing with Storm Water Runoff from Urban, Suburban and Developing Areas of Minnesota.* March 2000.

Nebraska Department of Transportation. *Drainage Design and Erosion Control Manual.* Current Edition.

Nebraska Department of Transportation. *Highway Mowing Guidelines.* Downloaded February 2012.

Nebraska Department of Transportation. *Plan for the Roadside Environment.* June 2008.

State of New Jersey. *New Jersey Stormwater Best Management Practice Manual.* February 2004.

StormwaterPA. *Pennsylvania Stormwater Best Management Practices Manual.* December 2006.

United States Environmental Protection Agency. *National Pollutant Discharge Elimination System (NPDES) Menu of BMPs.* Website Updated January 2008.

Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual Volume 3.* 2010.

Vegetated Filter Strip

Grass Swale

OVERVIEW

Source: Nebraska Department of Transportation (NDOT)

Definition

A grass swale is a densely vegetated drainage way with low pitched side slopes designed to convey runoff slowly. Grass swales are typically used in the following situations:

- Highways, interchanges, and facilities
- Small drainage areas
- Retrofit for a roadside ditch

Overview Table

Associated Costs	L	M	H
Design	X		
Construction	X		
Maintenance	X		
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients	X	X	
Heavy Metals		X	X
Hydrocarbons		X	X

Benefits

- Removes sediment and associated constituents through filtering.
- Reduces the length of storm sewer systems in upper portions of a watershed.
- Provides a less expensive and more attractive conveyance element.
- Can help reduce runoff volumes.

Limitations

- Grass swales require more area than traditional storm sewers.
- Erosion problems may occur if not designed and constructed properly.
- Performance is affected by length, depth, vegetation density and season of the year.
- Infiltration rates depend on soil types.

Grass Swale

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Grass swales are densely vegetated drainage ways and channels designed to convey runoff slowly. Grass swales have low longitudinal slopes and broad cross-sections that convey flow in a slow and shallow, facilitate sedimentation, infiltration, and filtration (straining).

Grass swales are primarily online stormwater treatment practices that convey runoff from larger storm events. They can receive runoff from point discharges, such as outfalls, as well as overland sheet flow from adjacent slopes and roadsides. They work well as pretreatment to and in conjunction with bioretention systems and other STFs.

STF COMPONENTS

Soils – The types of soils on site will partially determine how much water will be infiltrated. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics. Note any mixed and compacted soils for variability in soil characteristics.

Longitudinal Slope – Grass swales are designed to have enough of a slope in the direction of flow to keep water moving through the swale and avoid ponding but not too much slope that the water moves too quickly causing erosion and limiting infiltration.

Side Slope – Side slopes are kept shallow, when possible, to reduce erosion potential and increase infiltration area. Swales should have side slopes no steeper than 1V:3H.

Vegetation – The purpose of the vegetation is to effectively bind the soil, reduce velocities, and uptake water and some nutrients. A dense stand of vegetation that is drought tolerant, durable, and erosion resistant is desired. Salt tolerance, water, and nutrient needs should also be considered when selecting vegetation. The selection of seed, sod, and plantings will generally fall under the responsibility of the Roadside Development & Compliance Unit (RDCU) although there may be exceptions. Refer to NDOT's *Plan for the Roadside Environment*, June 2008, for region-specific information on vegetation.

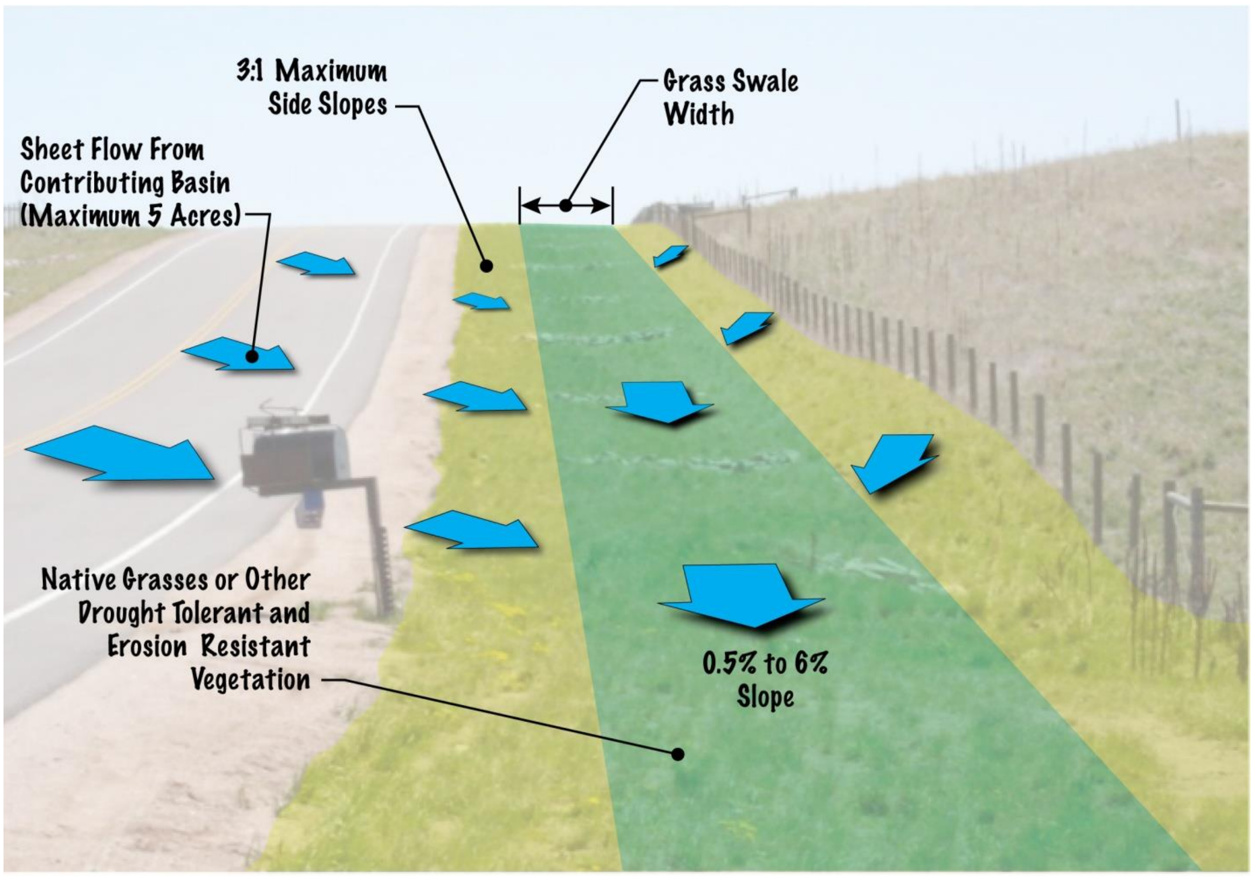
Length – The minimum length of the grass swale is measured in the direction of flow depends on vegetation height, flow depth, and flow velocities.

Width – The swale bottom should be wide enough to convey smaller flows uniformly but not so wide as to encourage braided channels. Width is measured perpendicular to flow.

Water Quality Volume Discharge Rate Depth, D_{WQ} – The depth of flow in the swale based on the Water Quality Volume Discharge Rate (Q_{WQ}) as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*.

Total Depth, D – The total depth of the channel which typically includes capacity to handle runoff from larger storm events as defined in Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual*.

Grass Swale



Grass Swale Example
Not to Scale

Grass Swale

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. How much area is available for the proposed grass swale adjacent to the project?

Q. Is groundwater present in the vicinity of the proposed grass swale?

Q. What types of soils are on the site and are they compacted?

Q. How does the proposed grass swale section interact with other design storm considerations?

Q. What is the slope of the proposed surface for the grass swale and can velocities be reduced?

Site characteristics are very important when designing grass swales and should be taken into consideration early in the design process.

Grass swales should be used only where the topography of the site allows appropriate slope and cross-sectional area for the swale.

Grass swales should be used only when the seasonally high groundwater table is at least 2 feet below the channel's flow line. Consider using stormwater wetlands for high groundwater conditions.

Soils that have been compacted should be loosened to improve infiltration and increase water quality treatment effectiveness. The depth that should be disced or tilled is based on the estimated depth of compaction and should include the sides, as well as the bottom of the swale. Areas that have been disturbed as a result of construction traffic and staging should be disced or tilled along with any fill areas. Finished grades should be amended with topsoil.

Because grass swales are typically online STFs, they need to be designed to treat the Water Quality Volume Discharge Rate (Q_{WQ}) and pass the design storm discharged referenced in Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual*. Grass swales should be used to treat small drainage areas of less than 5 acres.

When sizing the swale for the Water Quality Volume Discharge Rate, a maximum depth of flow of 4 inches is allowed, velocities should be less than or equal to 1 foot/second, and a travel time of 5 minutes or more is required. This allows sediment to drop out and improves infiltration and filtration. The designer will need to check velocities and permissible shear stress of the swale for erosive conditions from runoff during the design storm event. Energy dissipation should be provided at any outfalls discharging into the grass swale to reduce velocities. The use of any drops should provide energy dissipation as well.

Manning's coefficient "n" is an important parameter in sizing the swale and varies with the depth of flow and vegetative height and condition. For design purposes, assume a vegetative height of 6 inches. For flows less than or equal to 4 inches, as required for determining velocity for the Water Quality Volume Discharge Rate, use an "n" value of 0.15. For depths of flow greater than 4 inches, use an "n" value determined from Appendix B of NDOT's *Drainage Design and Erosion Control Manual*.

Grass Swale

DESIGN CRITERIA

Description	Value
Maximum Contributing Basin Area	5 acres
Longitudinal Slope	0.5% - 6%
Maximum Side Slopes of Swale	1:3
Bottom Width of Swale	2-10 feet
Design Storm	Water Quality Volume Discharge Rate, Q_{WQ}
Maximum Velocity for Q_{WQ}	1 foot/second
Maximum Depth for WQV Discharge Rate, D_{WQ}	4 inches
Minimum Travel Time for Q_{WQ}	5 minutes
Manning's Coefficient, n	0.15 (for flow depth ≤ 4 inches)

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume Discharge Rate (Q_{WQ})

Calculate the water quality volume discharge rate using the NRCS Curve Number (CN) Procedure as described in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* **OR** use Exhibit 3.5 - Water Quality Volumes and Peak Discharges for Selected Acreages for sites under 5 acres.

Step 2: Determine swale width using Q_{WQ}

Using Q_{WQ} from above, the slope (S) selected from (typically 1% to 4%), a Manning's "n" of 0.15, and a maximum water depth (D) of 4 inches (0.5 feet), calculate an appropriate width channel using the following modified form of Manning's equation:

$$Q_{WQ} = VA = 1.486/n * R^{2/3} * S^{1/2} * A$$

Q_{WQ} = WQV discharge rate (cfs)
V = velocity of flow (ft/s)
A = flow area (ft^2) = wD (approximate for shallow flow)
n = Manning's coefficient
R = hydraulic radius (ft) = A/WP
WP = Wetted Perimeter = w (approximate for shallow flow)
w = channel bottom width (ft)
D_{WQ} = flow depth for WQV discharge rate (ft)
(max. allowable depth 4 inches)
S = slope (ft/ft)

Simplify equation using the assumptions provided above; solve for bottom width:

$$w = \frac{Q_{WQ} * n}{1.486 * D_{WQ}^{5/3} * S^{1/2}} \quad \text{(check criteria for bottom width)}$$

Grass Swale

Step 3: Check velocity criteria

Using Q_{WQ} from above and cross-sectional area, solve for velocity:

$$V = \frac{Q_{WQ}}{w * D_{WQ}}$$

If velocity is too high for given criteria, consider reducing the slope, widening the channel, and, if close, calculating out wetted perimeter and hydraulic radius.

Step 4: Calculate minimum swale length

$$L_{min} = V * t_r * (60 \text{ s/min})$$

t_r = travel time (5 min)

Travel time is defined as the time it takes for runoff to travel from the upgradient end of the swale to the downgradient end of the swale.

Step 5: Check effective treatment of swale

Check that a minimum of 80 percent of the drainage area is captured upgradient from the upper limits of L_{min}.

$$\frac{A_{Lmin}}{A_{Total}} * 100 \geq 80\%$$

A_{Lmin} = Drainage area upgradient from the upper limits of the drainage way (assumed to be the treated area) (ac)
A_{Total} = Drainage area at the downstream end of the drainageway (ac)

If the percentage of the fully treated area is less than 80 percent, provide additional treatment for the remaining area (see diagram on next page).

Step 6: Check capacity of swale and velocity for larger storm events

Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check swale capacity and the velocities of larger storm events.

Grass Swale

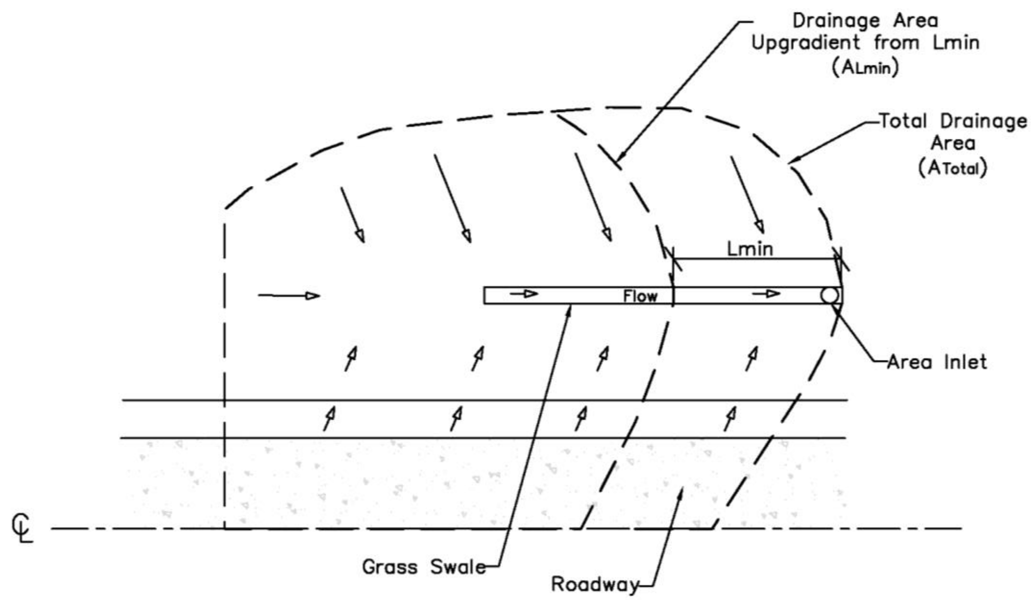
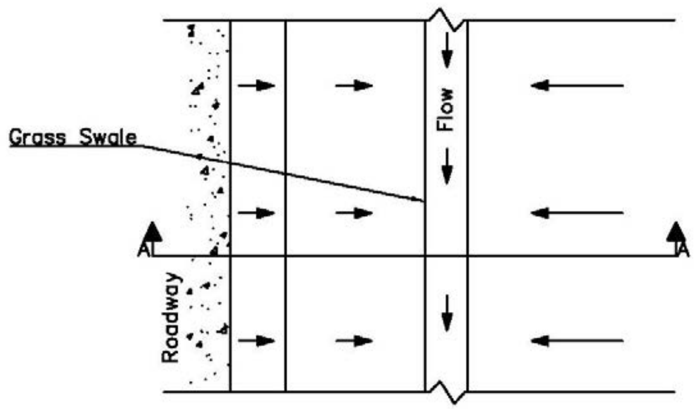
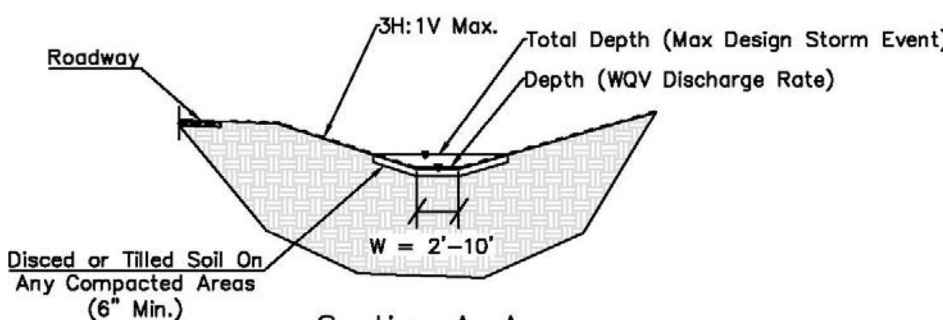


Figure 1. Drainage Area Diagram

DESIGN EXAMPLES

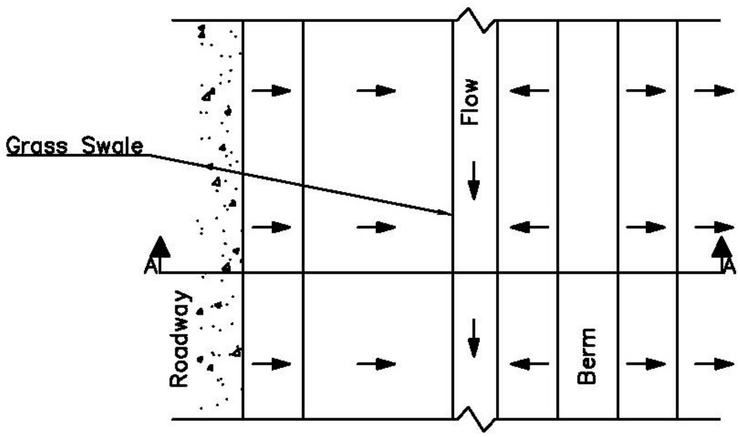


Grass Swale Along Backslope
Plan View
No Scale

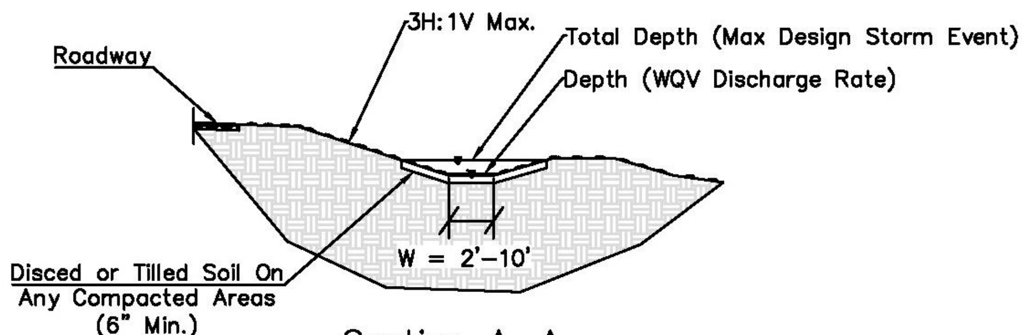


Section A–A
No Scale

Grass Swale



Grass Swale with Berm
Plan View
No Scale



Section A–A
No Scale

CONSTRUCTION CONSIDERATIONS

- Strip and stockpile topsoil before beginning earthwork and re-spread over finished areas, including the swale bottom and sides, to improve vegetative growth and infiltration.
- Perform fine grading, soil amendment, and seeding only after upgradient surfaces have been stabilized and utility work crossing the swale has been completed.
- Consider providing irrigation appropriate to the grass type.
- Weed the area during the establishment of vegetation by hand or by mowing. Mechanical weed control is preferred.
- Protect the swale from other construction activities.
- Include directions in the specifications for use of appropriate fertilizer and soil amendments based on soil properties and compared to the needs of the vegetation requirements.
- Install swales at the time of the year when there is a reasonable chance of successful vegetation establishment without irrigation; however, it is recognized that rainfall in a given year may not be sufficient and temporary irrigation may be used.
- If sod tiles must be used, place them so that there are no gaps between the tiles; stagger the ends of the tiles to prevent the formation of channels along the swale or strip.
- Where seeding is used, erosion controls will be necessary to protect seeds until vegetation is well established.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for vegetated swale systems include providing litter control, keeping up the hydraulic and removal efficiency of the swale by repairing erosion and removing sediments or other obstructions, and maintaining a dense, healthy grass cover.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect swale to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect swale for erosion and any damage by equipment or vehicles. Repair as needed. • Inspect swale for sediment buildup in the bottom of the swale. Remove sediment once it has accumulated. • Remove trash and debris accumulated in the swale.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect swale to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect grass swale for erosion and damage by equipment or vehicles. Repair as needed. • Inspect swale for sediment buildup in the bottom of the swale. Remove sediment once it has accumulated. • Remove excessive trash and debris accumulated in the swale.
Annually (After NPDES Permit is closed)	• Inspect swale to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect grass swale for erosion and damage by equipment or vehicles. Repair as needed. • Inspect swale for sediment buildup in the bottom of the swale. Remove sediment once it has accumulated. • Remove excessive trash and debris accumulated in the swale.

RESOURCES AND REFERENCES

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Infiltration Trench

OVERVIEW

Source: Nebraska Department of Transportation (NDOT)

Definition

Infiltration trenches temporarily store runoff in trenches filled with aggregate and infiltrate that runoff over a limited time period. Infiltration trenches are appropriate for smaller drainage areas. They are sited on relatively flat areas with porous soils.

Benefits

- Flexible system that can provide detention as well as water quality benefits in some cases.
- Provides for groundwater recharge.
- Can be configured in any shape to meet right-of-way restrictions.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction			X
Maintenance		X	
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients		X	X
Heavy Metals			X
Hydrocarbons		X	

Limitations

- Infiltration trenches are not suitable in areas of compacted fill or low permeability soils.
- Infiltration trenches are not suitable in areas with a high groundwater table or groundwater contamination issues.
- Minimum setbacks must be met.
- Pretreatment is encouraged to help reduce the potential for clogging.

Infiltration Trench

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Infiltration trenches are aggregate-filled trenches that capture and store surface runoff from the contributing watershed allowing it to infiltrate into subsurface soils. Trash and debris are screened at the surface, and infiltration occurring through underlying soils provides additional treatment. Infiltration trenches are designed to capture the Water Quality Volume (WQV) and infiltrate that volume within a 48-hour period (72 hours maximum). In areas with sediment laden stormwater, pretreatment is necessary to minimize the potential for clogging, extend the life of the system, and provide additional stormwater treatment.

Infiltration trenches can be designed offline or online. In an online system, storage for WQV is provided in the aggregate voids below a set intake structure elevation. In an offline system, infiltration trenches may be used with other STFs to provide peak flow control. Infiltration trenches can accept flow from an outfall as a point discharge or as sheet flow from adjacent runoff.

STF COMPONENTS

Pretreatment STF – A Pretreatment STF is one of any number of STFs (vegetated filter strips, grass swales, forebays, etc.) which provides a gross reduction in the amount of trash and sediment carried by stormwater before it enters the infiltration trench. Placement of a Pretreatment STF upgradient of the infiltration trench will reduce the likelihood of its clogging and failure. Many factors dictate the types of pretreatment STFs suitable for the site, including available space, an offline or online system, soil characteristics, site topography and cost. See the STF design guidelines of the various systems for additional information on how to design the pretreatment.

Aggregate-Filled Trench **–** An aggregate-filled trench is an excavated area filled with aggregate that holds stormwater and allows it to infiltrate into groundwater over a specified amount of time. The aggregate should be free-draining aggregate with a suitable void ratio to provide space for storage. For aggregate gradations with a maximum size equal to or greater than ½ inch, a non-woven filter fabric is needed along the sides of the trench and a free-draining sand-gravel layer or non-woven filter fabric on the bottom to reduce the potential for migration of fines into the free-draining aggregate. The bottom of the trench should be level.

Berm **–** A berm is an earthen ridge used to contain, block and/or divert stormwater flows. Berms are frequently used in stormwater management design to contain and/or direct water quality flows into STFs.

Soils **–** The soil(s) of concern for the Infiltration Trench STF are located one foot above the base of the trench to five feet below. The permeability of these subsurface soils will determine the infiltration rate of the STF and therefore the minimum size of the trench. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics for design.

Infiltration Trench

Length-Width Dimensions – The length and width dimensions define the area of the bottom of the infiltration trench, which depends on the volume of storage needed, soil properties and available space. Length is defined as the longer axis in an x-y plane. Width is defined as the shorter axis in an x-y plane.

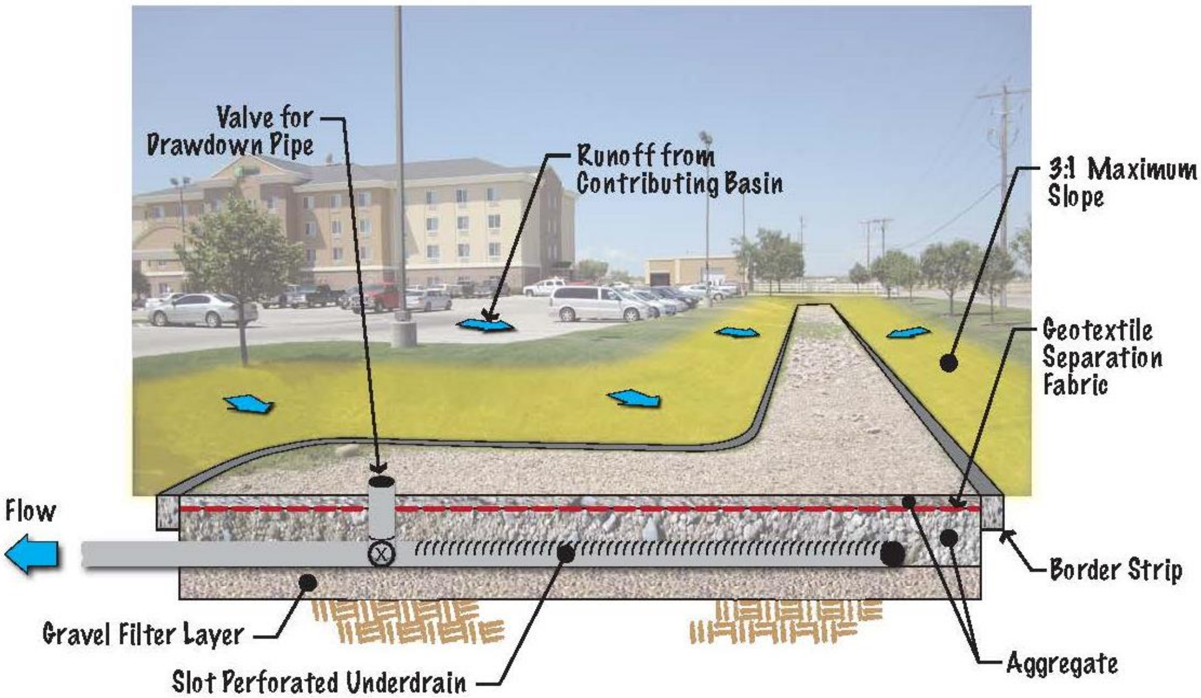
Depth – The depth of the infiltration trench generally depends on the volume of storage needed, soil infiltration rate, and available space. It can also be limited by shallow groundwater or bedrock. Depth is measured from the bottom of the excavation to the surface of backfilled material.

Observation Well – A 4" diameter observation well is recommended for every 5000 square feet of infiltration trench (L x W) so that the rate of infiltration can be observed as needed. The observation well is a slot-perforated drainpipe that extends from the bottom of the excavated trench to an elevation 6 inches above the surface of backfilled material. A fixed cap can be fitted to the bottom of the pipe, or the pipe can rest on a steel or plastic foot plate. The top of the pipe has a removable locking cap.

Drawdown Pipe – Where grades allow, the designer should include a drawdown pipe that would allow any standing water in the infiltration trench to be drawn down in the event of plugging or repairs. The drawdown pipe system includes a section of slot perforated drainpipe placed on the bottom of the trench that connects to an adjustable valve followed by a non-perforated outlet pipe that that discharges to a sewer pipe or is daylighted. A riser pipe is used to access and open or close the adjustable valve. Drawdown pipes should be 4-inch diameter or larger.

Overflow Spillway – An overflow spillway is a protected area of the online Treatment STF designed to convey peak discharges that exceed water quality events. Energy dissipation should be provided where velocities and turbulence are a concern. See Chapter 1 of NDOT's *Drainage Design and Erosion Control Manua*l for more information on energy dissipation.

Infiltration Trench



Infiltration Trench Example
Not to Scale

Infiltration Trench

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Does site topography allow for the placement of a level infiltration trench?

Q. What impact will infiltration have on adjacent pavement, buildings, water bodies, groundwater, etc.?

Q. What types of soils are on site and are they compacted?

Q. How does the proposed infiltration trench interact with other design storm considerations?

Q. What type of pretreatment is appropriate?

Site characteristics are very important when designing infiltration trenches and should be taken into consideration early in the design process.

Infiltration trenches are built with a level bottom and should be used only where the topography of the site allows for this. Check the available right-of-way when determining the footprint.

Infiltration trenches are susceptible to leaching pollutants into sensitive ground waters or saturating soils adjacent to infrastructure. The designer should reference design criteria for required setback distances.

The bottom of the infiltration trench is level and should be at least 4 feet above the seasonal high groundwater table, bedrock or other barrier layer.

A minimum infiltration rate of 0.50 inches/hour is required. In general, Hydrologic Soil Groups A and B meet the necessary infiltration rate; however, the soil should be tested for infiltration before finalizing the design to verify the assumed infiltration rate. Underlying soils with an infiltration rate greater than 12 inches/hour should not be used because of the higher potential for groundwater contamination.

Infiltration rates should be determined based on procedures outlined in the Nebraska Department of Environmental Quality Title 124 – Chapter 6 for soil percolation. Test holes should extend to a depth approximately 6 inches below the bottom of the proposed infiltration trench. Laboratory testing for permeability is also acceptable.

Where infiltration rates greater than 0.50 inches/hour cannot be achieved, an underdrain system may be used. The underdrain system should be designed for a 48-hour drawdown time and include a valve to help control discharge rates if needed. Cleanouts should also be provided.

Stabilize all basin outfalls that discharge into the infiltration trench to prevent scour and erosion. When used in an offline configuration, the Water Quality Volume is diverted to the infiltration trench. Stormwater flows greater than WQV should bypass the STF.

The following Design Criteria table provides pretreatment criteria that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

Infiltration Trench

DESIGN CRITERIA

Description	Value
Maximum Contributing Basin Area	5 acres
Width	3 – 20 feet
Depth	3 – 8 feet (Depth ≤ Width)
Minimum Infiltration Rate	0.5 inch/hour
Maximum Infiltration Rate	12 inches/hour
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Aggregate Porosity	0.30 (Geotechnical engineer will need to approve anything greater than 0.30)
Time for Infiltration	48 hours (72 hours maximum)
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 10% of WQV (minimum)
Setback Distances	Surface Water – 50 feet Private Drinking Water Wells – 100 feet Public Drinking Water Supply Wells (Non-Community System) – 100 feet Public Drinking Water Supply Wells (Community System) – 500 feet Water Lines (Pressure) – 25 feet Water Lines (Suction) – 100 feet Property Lines – 5 feet Foundations (NDOT)* – 20 feet (assumes no basement) Foundations (Neighbors)* – 30 feet (assumes no basement)

* Add 10 feet to setback distance when foundations are lower in elevation than water quality feature or adjacent to a full depth basement.

Infiltration Trench

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* **OR** use Exhibit 3.5 - Water Quality Volumes and Peak Discharges for Selected Acreages for sites under 5 acres.

Step 2: Size the infiltration trench volume, V_T

To determine the minimum volume of the infiltration trench, divide the WQV by the available pore space:

$$V_T = WQV/n$$

V_T = infiltration trench volume (ft^3)
n = aggregate porosity
(assume 0.30 or obtain from project geotechnical report)

Step 3: Calculate minimum bottom surface area

The minimum bottom surface area depends on the infiltration rate of the soil and drain time allowed.

$$A_{min} = \frac{WQV * (12 \text{ in/ft})}{I * t}$$

A_{min} = minimum bottom surface area (ft^2)
I = infiltration rate of underlying soil (in/hr)
(obtain from field or laboratory testing)
t = time to drain, hrs (design for 48 hours)

Step 4: Calculate trench depth

Calculate the trench depth by dividing the trench volume by the bottom surface area.

$$D = \frac{V_T}{A}$$

D = depth of trench (ft)

Infiltration Trench

Step 5: Check depth and revise area if needed

Check depth calculated against depth criteria provided in the Design Criteria table and revise area if needed.

- If $D \leq D_{min}$ use D_{min} and A_{min} for design ($A_{min} = A$)
- If $D > D_{min}$ select a depth between D and D_{max} and revise A_{min}

$$A = \frac{V_T}{D}$$

A = design area (ft^2)

Step 6: Determine trench area dimensions and check other Design Criteria

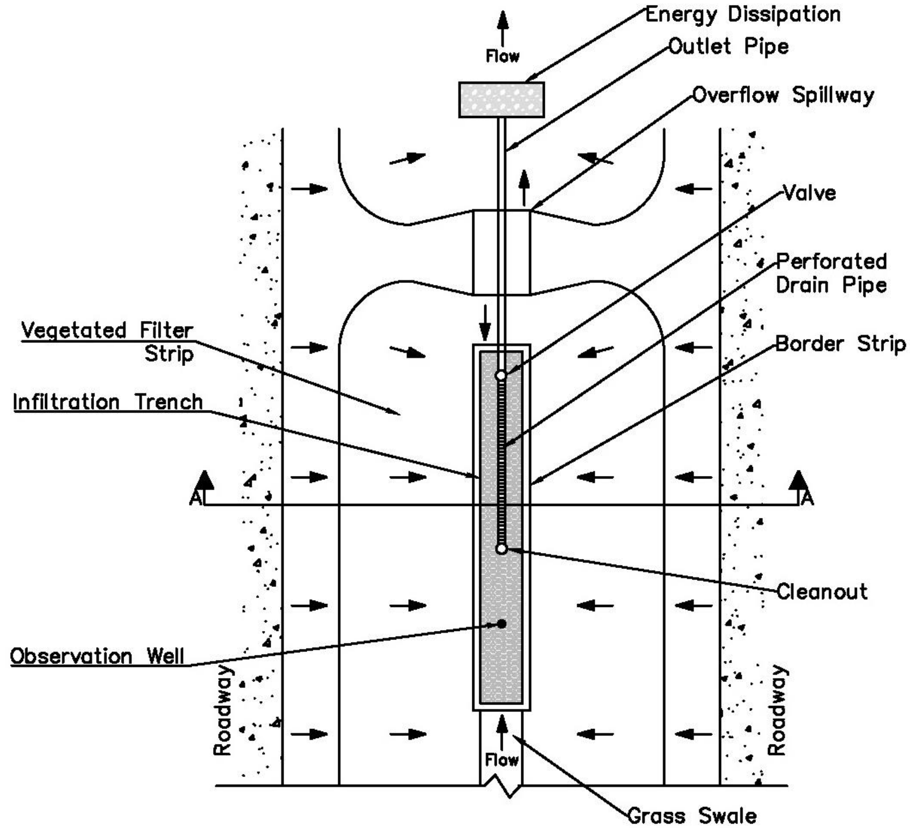
Designer needs to determine trench area dimensions using width criteria provided in the Design Criteria table and verify that $W \geq D$.

Step 7: Check diversion or storage and routing of larger storm events

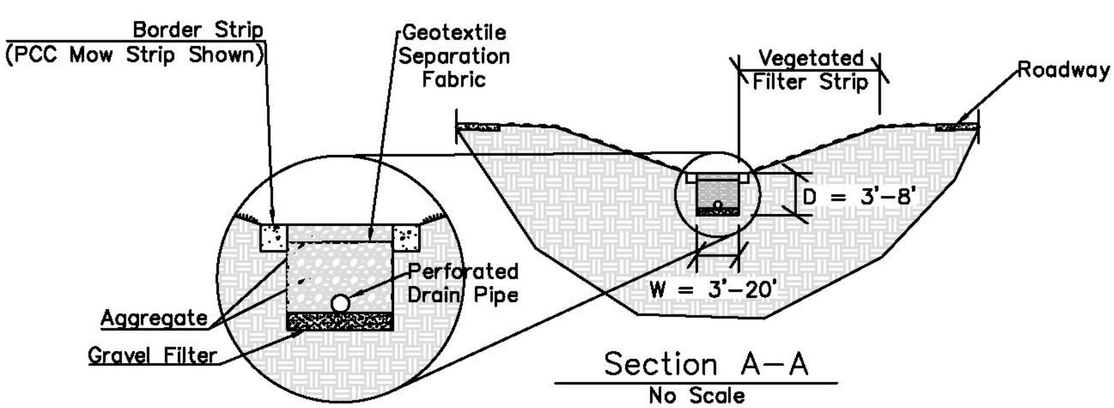
Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events. Design offline infiltration trenches to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online infiltration trenches for flow through and integrate any additional storage into the feature.

Infiltration Trench

DESIGN EXAMPLES



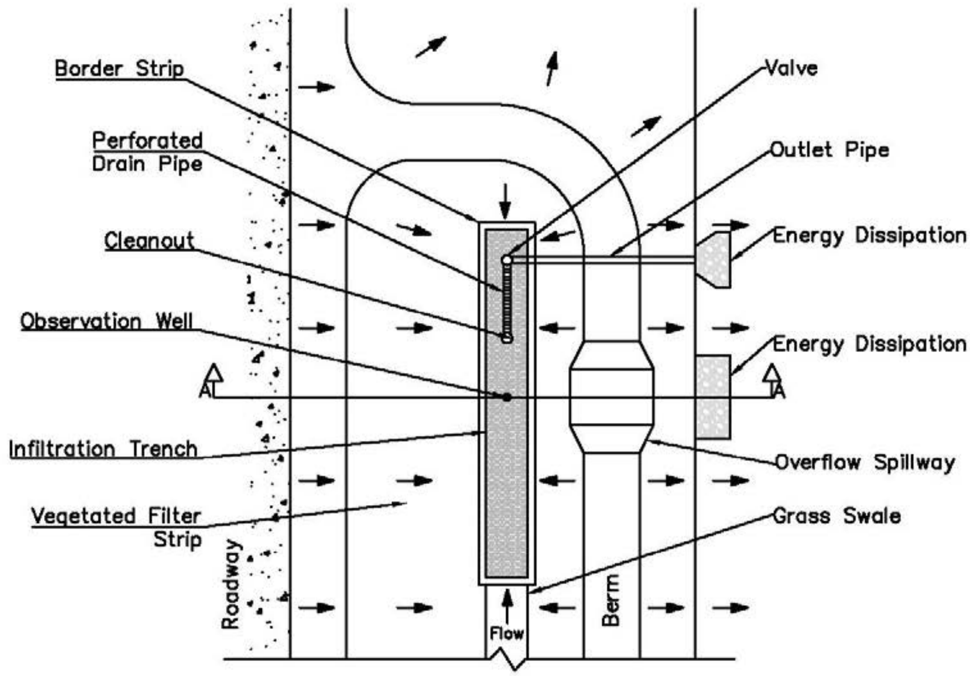
Infiltration Trench
(In Median)
Plan View



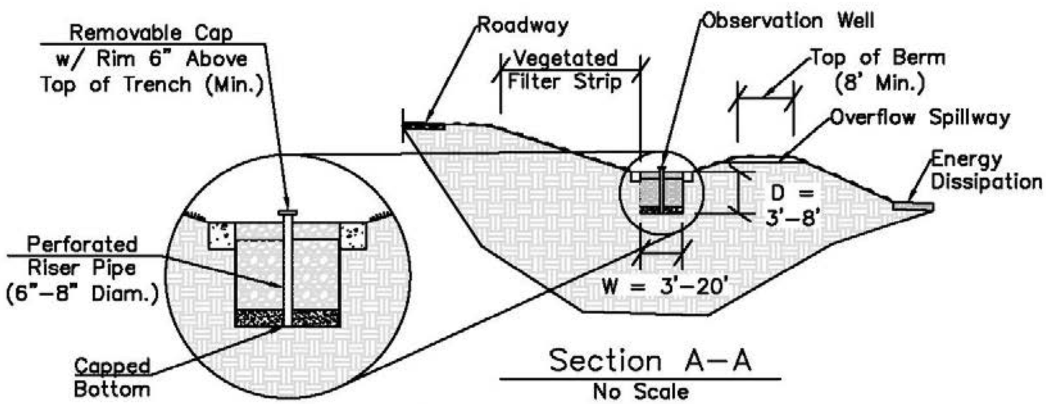
Section A–A
No Scale

Trench Section
No Scale

Infiltration Trench



Infiltration Trench
(Off Foreslope)
Plan View



Section A–A
No Scale

Observation Well
No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because infiltration trenches are prone to failure when inundated with a high sediment load.
- Stabilize upgradient contributing drainage area before putting infiltration trenches into operation.
- If it is not possible to stabilize upgradient before beginning construction and flow cannot be temporarily bypassed, provide erosion and sediment control protection for the infiltration trench.
- Because inlet protection is often not adequate during construction of an infiltration trench, the best protection is to bypass stormwater away from the facility until vegetation is established and all construction-related sediment has been controlled. Otherwise, the infiltration trench may be unusable immediately after implementation.
- Consider the space needed for pretreatment and any swale required for bypassed flow.
- If the infiltration area is being used as a sediment basin during construction, the bottom elevation of the sediment basin should be a minimum of 2 feet above the future infiltration bed invert elevation so that the trench can be excavated into native soils.
- Protect infiltration areas from construction or other traffic during the course of construction where practical. If this is not possible, take steps to reduce compaction of underlying soils.
- Excavate the bottom and sides of the trench in such a manner as to leave the soil in a natural, unsmeared, and uncompacted condition.
- Avoid using heavy equipment in the basin bottom during construction of the infiltration trench to maintain the infiltration rate.
- If infiltration areas do get compacted during construction, additional infiltration testing may be required.
- Consider trench stability and safety during construction. Refer to the Occupational Safety and Health Administration trench safety standards.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for infiltration trench systems include providing litter control, maintaining storage capacity within the aggregate, and maintaining infiltration rates of the aggregate and subsoils. Diversion structures, outlets, and forebays should also be inspected and maintained, along with any pretreatment STFs.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect infiltration trench for any surface ponding or indicators that water has ponded for an extended period of time. • Check observation wells 3 days (72 hours) after a major rainfall event to ensure proper drain time. • Inspect infiltration trench system for sediment buildup on the trench surface and any diversion structures, outlets, and forebays. Remove sediment as needed. • Inspect the trench and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect infiltration trench for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect infiltration trench for sediment buildup on the trench surface. Remove sediment as needed. • Inspect the trench and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Annually (After NPDES Permit is closed)	• Inspect infiltration trench for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect infiltration trench for sediment buildup on the trench surface. Remove sediment as needed. • Inspect the trench and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

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Infiltration Trench

Infiltration Basin

OVERVIEW

Source: Colorado Department of Transportation (CDOT)

Definition

Infiltration basins capture runoff and allow it to infiltrate through native soils over a limited time period. The bottom of the basin is flat, and pollutants are removed through sedimentation, filtration, and adsorption to underlying soils. The bottom of the basin is vegetated and delineated with a border strip.

Benefits

- Flexible system that can provide detention as well as water quality benefits
- Effective in removing most pollutants.
- Provides for groundwater recharge.
- Can be shaped to meet right-of-way restrictions.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction		X	X
Maintenance		X	
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients		X	X
Heavy Metals			X
Hydrocarbons			X

Limitations

- Infiltration basins are not suitable in areas of compacted fill or low permeability soils.
- Infiltration basins are not suitable in areas with a high groundwater table or groundwater contamination issues.
- Minimum setbacks must be met.
- Pretreatment is encouraged to help reduce the potential for clogging.
- Performance is reduced in cold weather.

Infiltration Basin

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Infiltration basins rely on infiltration through native soil to provide water quality treatment. They are designed to capture the Water Quality Volume (WQV) and infiltrate that volume in a 24-hour period (48 hours maximum). Infiltration basins can be designed offline or online and can be modified to include additional storage for peak flow reduction. They can accept flow from an outfall as a point discharge or as sheet flow from adjacent runoff. Pretreatment reduces the potential for clogging and extends the life of the system.

STF COMPONENTS

Pretreatment STF – A Pretreatment STF is one of any number of STFs (vegetated filter strips, grass swales, forebays, etc.) which provides a gross reduction in the amount of trash and sediment carried by stormwater before it enters the infiltration basin. Placement of a Pretreatment STF upgradient of the infiltration basin will reduce the likelihood of its clogging and failure. Many factors dictate the types of pretreatment STFs suitable for the site, including available space, an offline or online system, soil characteristics, site topography and cost. See the STF design guidelines of the various systems for additional information on how to design the pretreatment.

Soils **–** The types of soils on site will partially determine how much water will be infiltrated and whether this STF is a suitable option. Infiltration basins, as defined herein, rely on filtration through native soils. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics for design.

Berm **–** A berm is an earthen ridge used to contain, block and/or divert stormwater flows. Berms are frequently used in stormwater management design to contain and/or direct water quality flows into STFs.

Length-Width Dimensions – Length and width dimensions define the area of the bottom of the infiltration basin which depends on the soil properties and volume of runoff. The shape is dictated by the available space; however, a length to width ratio of 2:1 is generally desirable with the length measured from the primary inflow point to the location of the drawdown structure. Length is defined as the longer axis in an x-y plane. Width is defined as the shorter axis in an x-y plane.

Water Quality Volume Depth, D_{WQ} **–** The depth of storage in the infiltration basin based on the Water Quality Volume (WQV) as defined in Chapter 3 of the Nebraska Department of Transportation (NDOT) *Drainage Design and Erosion Control Manual*. It depends on the maximum soil infiltration rate and the volume of storage needed.

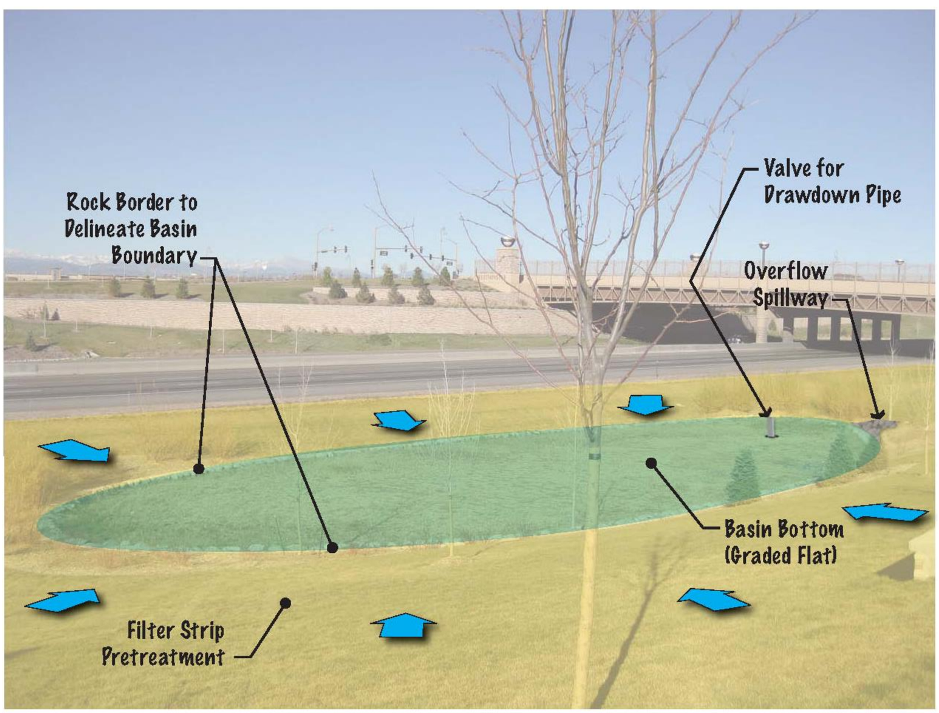
Total Depth **–** The depth of water stored in the infiltration basin. For a basin that is placed online, this depth includes the capacity to handle peak runoff for the design storm event and any additional storage for peak flow reduction.

Infiltration Basin

Drawdown Pipe – Where grades allow, the designer should include a drawdown pipe that would allow any standing water in the infiltration basin to be drawn down in the event the basin does not drain and as needed for repairs. The drawdown pipe would include a section of slot perforated drainpipe lying horizontally below the bottom of the basin that connects to an adjustable valve, any riser pipe needed to access the valve, and solid drainpipe that discharges to a sewer pipe or is daylighted. The valve will remain closed under normal operations. A cleanout should be provided on the upgradient end of the perforated pipe section. Drawdown pipes should be 4-inch diameter minimum. The length should be one half the length of the basin (10’ minimum).

Overflow Spillway – An overflow spillway is a protected area along a berm designed to convey overflow storm events instead of allowing overtopping of the berm. Energy dissipation should be provided where velocities and turbulence are a concern.

Border Strip – A border strip is a Portland cement mow strip, rock border, or other marker that delineates the basin from the surrounding landscape.



Infiltration Basin Example
Not to Scale

Infiltration Basin

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Does site topography allow for the placement of a level infiltration basin?

Q. What impact will infiltration have on adjacent pavement, buildings, water bodies, groundwater, etc.?

Q. What types of soils are on site and are they compacted?

Q. How does the proposed infiltration basin section interact with other design storms?

Q. What type of pretreatment is appropriate?

Infiltration basins often require a large flat area and should be used only where the topography of the site allows for this. Place infiltration basins outside lateral obstacle clearance zones since water is stored at depths up to 1 foot.

Infiltration basins are susceptible to leaching pollutants into sensitive waters or saturating soils adjacent to infrastructure. The designer should reference the design criteria for setback distances.

The bottom of the infiltration basin should be level and at least 4 feet above the seasonal high groundwater table, bedrock, or other barrier layer.

It is necessary for soils to have a minimum infiltration rate of 0.50 inches/hour for the basin to function. In general, Hydrologic Soil Groups A and B meet the necessary infiltration rate; however, the soil should be tested for infiltration before finalizing the design to verify the assumed infiltration rate. Soils with an infiltration rate greater than 12 inches/hour should not be used because of the higher potential for groundwater contamination.

Infiltration rates of native soil should be determined based on procedures outlined in the Nebraska Department of Environmental Quality's Title 124 – Chapter 6 for soil percolation. Test holes should extend to a depth approximately 6 inches below the bottom of the basin. Laboratory testing for permeability is also acceptable.

Infiltration basins may be modified to include additional storage for peak flow reduction as long as the Water Quality Volume (WQV) is provided below the lowest opening of any inlet structure.

Stabilize all basin outfalls that discharge into the infiltration basin to prevent scour and erosion. Stabilization at outfalls should also be designed to help distribute the flow uniformly across the basin.

The following Design Criteria table provides pretreatment criteria that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

Infiltration Basin

DESIGN CRITERIA

Description	Value
Maximum Contributing Basin Area	10 acres (online) 20 acres (offline)
Maximum WQV Depth, D_{WQ}	1 foot
Maximum Depth of Basin, D (Full Depth That Includes Detention Volume)	2 feet
Minimum Infiltration Rate	0.5 inch/hour
Maximum Infiltration Rate	12 inches/hour
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Time for Infiltration	24 hours (48 hours maximum)
Underdrain Length	10 feet minimum (50% of basin length typical)
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 10% of WQV (minimum)
Setback Distances	Surface Water – 50 feet Private Drinking Water Wells – 100 feet Public Drinking Water Supply Wells (Non-Community System) – 100 feet Public Drinking Water Supply Wells (Community System) – 500 feet Water Lines (Pressure) – 25 feet Water Lines (Suction) – 100 feet Property Lines – 5 feet Foundations (NDOT)* – 20 feet (assumes no basement) Foundations (Neighbors)* – 30 feet (assumes no basement)

* Add 10 feet to setback distance when foundations are lower in elevation than water quality feature or adjacent to a full depth basement.

Infiltration Basin

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate the WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* **OR** use Exhibit 3.5 - Water Quality Volumes and Peak Discharges for Selected Acreages for sites less than 5 acres if appropriate.

Step 2: Calculate bottom surface area

The minimum bottom surface area depends on the infiltration rate of the soil and drain time allowed.

$$A_{min} = \frac{WQV * (12 \text{ in/ft})}{I * t}$$

A_{min} = minimum bottom surface area, ft^2
I = infiltration rate of underlying soil (in/hr)
(obtain from field or laboratory testing)
t = time to drain, hrs (design for 24 hours)

Step 3: Calculate design area for a determined depth

For D = 1 foot, $A_{min} = A$

For D < 1 foot, use the following equation:

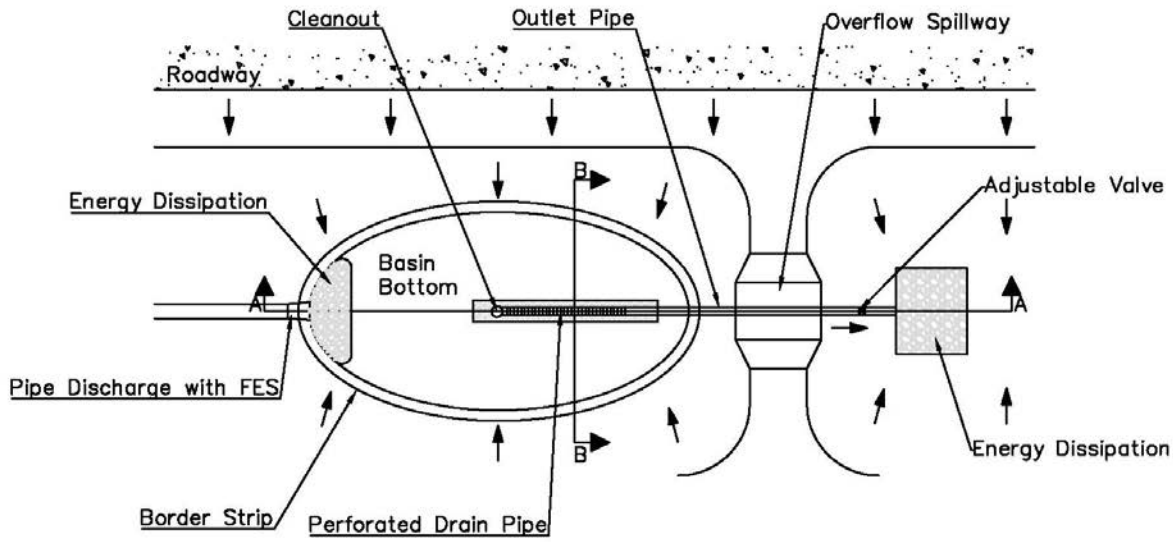
$$A = \frac{WQV}{D}$$

A = design area

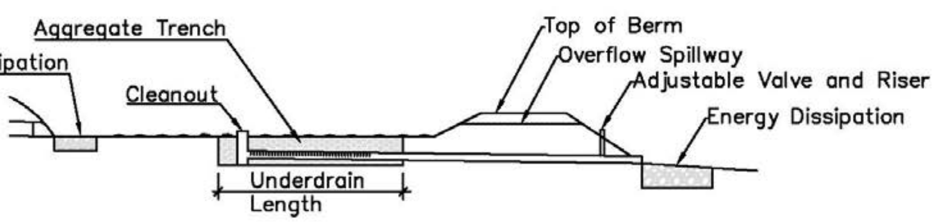
Step 4: Check diversion or storage and routing of larger storm events

Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events. Design offline infiltration basins to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online infiltration basins for flow through and integrate any additional storage into the feature. Additional storage to reduce peak runoff will be added above the WQV.

DESIGN EXAMPLES



Infiltration Basin
No Scale



Section A–A
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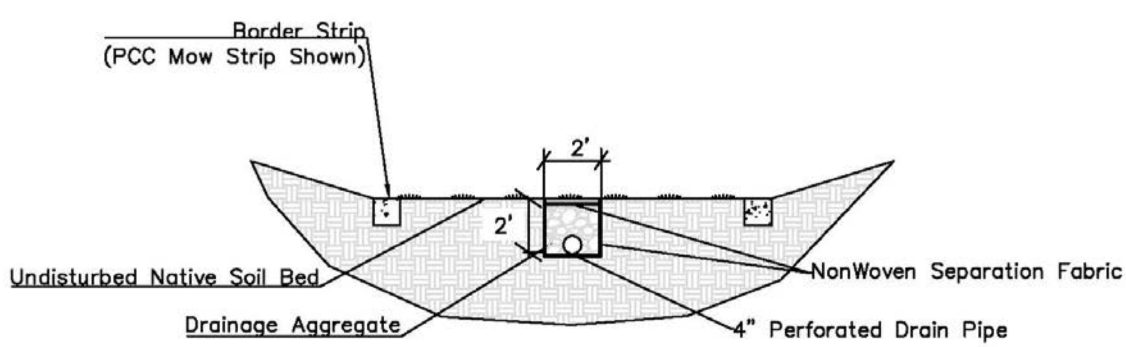


Section B–B
No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because infiltration basins are prone to failure when inundated with a high sediment load.
- Stabilize the upgradient contributing drainage area before putting infiltration basins into operation.
- Because inlet protection is often not adequate during construction of an infiltration basin, the best protection is to bypass stormwater away from the facility until vegetation is established and all construction-related sediment has been controlled. Otherwise, the infiltration basin may be unusable immediately after implementation.
- Consider the space needed for pretreatment and any swale required for bypassed flow.
- If upgradient stabilization is not possible before beginning construction and flow cannot be temporarily bypassed, provide erosion and sediment control protection for the infiltration basin.
- If the infiltration area is being used as a sediment basin during construction, the bottom elevation of the sediment basin should be a minimum of 2 feet above the future infiltration bed invert elevation so that the trench can be excavated in native soils.
- Protect infiltration areas from construction or other traffic during the course of construction. If this is not possible, take steps to reduce compaction of underlying soils.
- Excavate the bottom of the infiltration basin in such a manner as to leave the soil in a natural, unsmeared, and uncompacted condition.
- If infiltration areas do get compacted during construction, additional infiltration testing may be required.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for infiltration basin systems include providing litter control, monitoring sedimentation and erosion, and maintaining infiltration rates of the soil. Diversion structures, outlets, and forebays should also be inspected and maintained, along with any pretreatment STFs.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect infiltration basin for any surface ponding or indicators that water has ponded for an extended period of time. • Check infiltration basin 3 days (72 hours) after a major rainfall event to ensure proper drain time. • Inspect infiltration basin to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect infiltration basin for erosion and any damage by equipment or vehicles after every major rainfall events. Repair as needed. • Inspect infiltration basin system for sediment buildup on the bottom of the basin and at any diversion structures, outlets, and forebays. Remove sediment as needed. • Remove excessive trash and debris from the basin and any diversion structures, outlets, and forebays. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect infiltration basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect infiltration basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect infiltration basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect infiltration basin for sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect infiltration basin for any surface ponding or indicators that water has ponded for an extended period of time.

Infiltration Basin

Frequency	Inspection and Maintenance Activity
Annually (After NPDES Permit is closed)	• Inspect infiltration basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect infiltration basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect infiltration basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect infiltration basin for sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

Infiltration Basin

RESOURCES AND REFERENCES

Arizona Department of Transportation, *ADOT Post-Construction Best Management Practices Manual*. July 2009.

Atlanta Regional Commission. *Georgia Stormwater Management Manual – Volume 2: Technical Handbook*. August 2001.

California Department of Transportation. *BMP Retrofit Pilot Program – Final Report.* January 2004.

California Department of Transportation. *Storm Water Quality Handbook – Project Planning and Design Guide*. July 2010.

Iowa State University – Institute for Transportation. *Iowa Stormwater Management Manual.* October 2009.

The Low Impact Development Center, Inc.

Minnesota Pollution Control Agency. *Protecting Water Quality in Urban Areas - Best Management Practices for Dealing with Storm Water Runoff from Urban, Suburban and Developing Areas of Minnesota.* March 2000.

Nebraska Department of Environmental Quality. *Title 124 - Rules and Regulations for the Design, Operation and Maintenance of Onsite Wastewater Treatment Systems.* December 2007.

Nebraska Department of Transportation. *Drainage Design and Erosion Control Manual.* Current Edition.

State of New Jersey. *New Jersey Stormwater Best Management Practice Manual.* March 2003.

StormwaterPA. *Pennsylvania Stormwater Best Management Practices Manual.* December 2006.

United States Environmental Protection Agency. *National Pollutant Discharge Elimination System (NPDES) Menu of BMPs.* Last Updated January 2008.

United States Environmental Protection Agency. *Storm Water Technology Fact Sheet - Infiltration Trench.* September 1999.

Upper White River Watershed Alliance. *Green Infrastructure Fact Sheets.* Downloaded February 2012.

Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual Volume 3.* 2010.

Infiltration Basin

Bioretention

OVERVIEW

Source: Delaware Department of Transportation (DelDOT)

Definition

Bioretention systems temporarily store runoff in a shallow vegetated basin and infiltrate stormwater over a limited time period through underlying soils. An infiltration cell, consisting of a sand and compost layer with an underdrain, is part of this design. The infiltration cell helps provide water quality treatment in low permeability soils.

Benefits

- Suitable for low permeability soils when constructed with an infiltration cell and underdrain.
- Flexible system that can provide detention as well as water quality benefits.
- Can be shaped to meet right-of-way restrictions.
- Can beautify a site with landscaped feature.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction		X	
Maintenance		X	
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients		X	
Heavy Metals		X	X
Hydrocarbons		X	

Limitations

- Bioretention systems are not typically suitable in areas with a high groundwater table or groundwater contamination issues.
- Minimum setbacks must be met.
- Pretreatment is encouraged to help reduce the potential for clogging.
- Concentrated flows should include energy dissipation.

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Bioretention systems are a single basin or series of basins that capture and filter stormwater runoff through an infiltration cell and infiltrate it into surrounding soils. The infiltration cell consists of sand and compost filtration media and an underdrain. Surface soils within the basin are amended with compost. Bioretention systems are landscaped with non-invasive, preferably native, vegetation that is suitable for the dry and wet cycles of the basin. Shredded wood mulch is typically used for groundcover.

Bioretention basins are designed to capture the Water Quality Volume (WQV) and infiltrate that volume in a 24-hour period (48 hours maximum). Pretreatment reduces the potential for clogging and extends the life of the system.

Bioretention basins can be designed offline or online and can be modified to include storage to provide a reduction in peak runoff. Online bioretention basins should include an inlet structure to pass storm events that exceed the Water Quality event. Bioretention basins can accept flow from an outfall as a point discharge or as sheet flow from adjacent runoff; however, energy dissipation that spreads the flow out is needed for any point discharge.

STF COMPONENTS

Pretreatment STF – A Pretreatment STF is one of any number of STFs (vegetated filter strips, grass swales, forebays, etc.) which provides a gross reduction in the amount of trash and sediment carried by stormwater before it enters the bioretention basin. Placement of a Pretreatment STF upgradient of the bioretention basin will reduce the likelihood of its clogging and failure. Many factors dictate the types of pretreatment STFs suitable for the site, including available space, an offline or online system, soil characteristics, site topography and cost. See the STF design guidelines of the various systems for additional information on how to design the pretreatment.

Soils **–** The types of soils on site will partially determine how much water will be infiltrated in underlying soils. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics for design.

Infiltration Cell – The infiltration cell is an excavated section of the lower part of the bioretention basin which is backfilled with a filtration media. The filtration media usually includes a mix of fine sand and organic compost. An underdrain system is placed at the bottom of the infiltration cell. For aggregate gradations with a maximum size equal to or greater than ½ inch, a non-woven filter fabric is also needed along the sides of the trench and a free-draining sand-gravel layer or non-woven filter fabric on the bottom to reduce the potential for migration of fines into the free-draining aggregate. The bottom of the trench should be level.

Berm **–** A berm is an earthen ridge used to contain, block and/or divert stormwater flows. Berms are frequently used in stormwater management design to contain and/or direct water quality flows into STFs.

Bioretention

Length-Width Dimensions – Length and width dimensions define the area of the bottom of the bioretention basin which depends on the volume of runoff. The shape is dictated by the available space; however, a length to width ratio of 2:1 is generally desirable with the length measured from the primary inflow point to the location of the drawdown structure. Length is defined as the longer axis in an x-y plane. Width is defined as the shorter axis in an x-y plane.

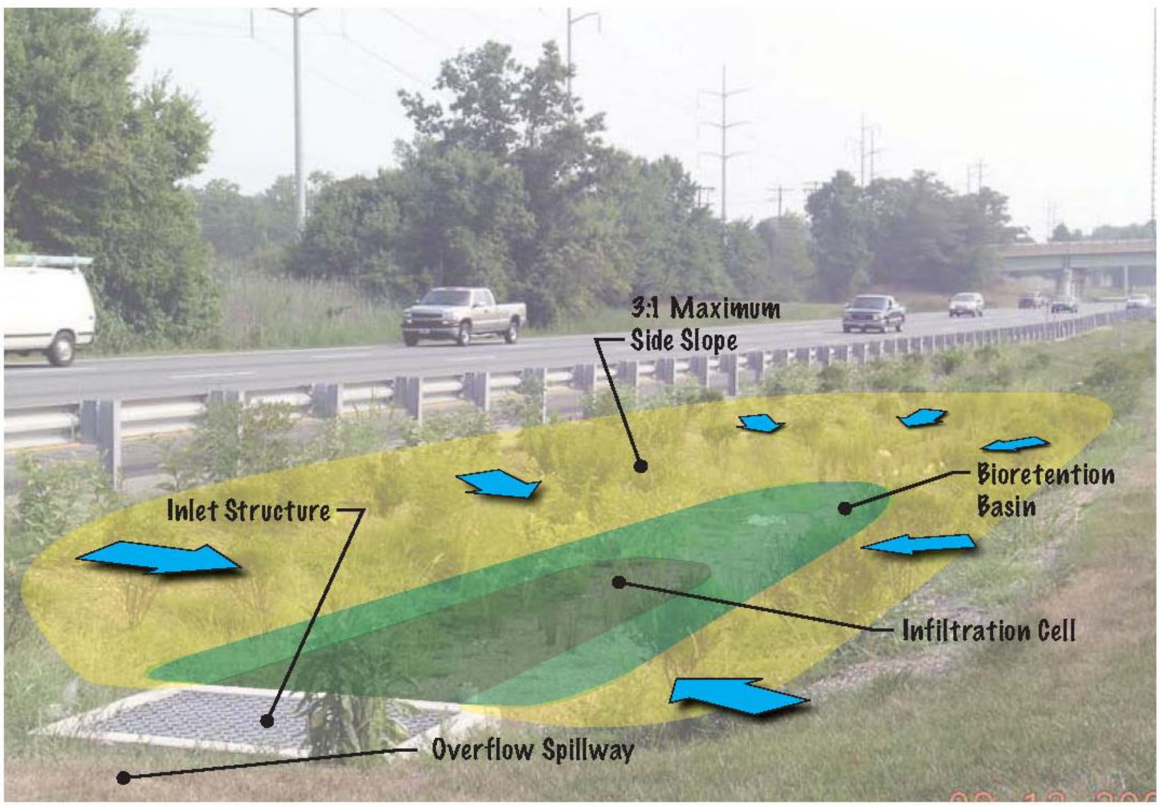
***Water Quality Volume Depth, D_{WQ}* –** The depth of water stored in the bioretention basin is based on the Water Quality Volume (WQV) as defined in Chapter 3 of the Nebraska Department of Transportation (NDOT) *Drainage Design and Erosion Control Manual*. It generally depends on soil properties and the volume of storage needed.

***Total Depth* –** The depth of water stored in the bioretention basin. For a basin that is placed online, this depth includes the capacity to handle peak runoff for the design storm event and any additional storage for peak flow reduction.

Underdrain – The designer should include an underdrain positioned at the bottom of the infiltration cell that would allow for water to drain out of the basin. The underdrain pipe would include a section of slot perforated drainpipe lying horizontally at the bottom of the chamber surrounded by a 6-inch layer of 1 ½-inch drainage aggregate. The slot perforated drainpipe should connect to an adjustable valve, any riser pipe needed to access the valve, and solid drainpipe that discharges to a sewer pipe or is daylighted. A cleanout should be provided on the upgradient end of the perforated pipe section. Underdrain pipes should be 4-inch diameter minimum.

Inlet Structure – An inlet structure is a feature within the STF which is designed to convey the peak runoff that exceeds the Water Quality event before negative impacts to the STF occurs. Flows up through the 100-year storm event may need to be considered. Energy dissipation should be provided where velocities and turbulence are a concern.

Bioretention



Bioretention Example
Not to Scale

Bioretention

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Does site topography allow for the placement of a level infiltration basin?

Q. What types of soils are on site and are they compacted?

Q. What impact will infiltration have on adjacent pavement, buildings, water bodies, groundwater, etc.?

Q. Is a reliable source of organic compost readily available?

Q. What type of pretreatment is appropriate?

Site characteristics are very important when designing bioretention systems and should be taken into consideration early in the design process.

Bioretention basins are built with a level bottom and should be used only where the topography of the site allows for this.

Multiple stepped basins may be needed on a sloped area to provide the necessary volume of treatment. Check site slopes and available right-of-way when determining the footprint.

The bottom of the bioretention basin should be level and at least 4 feet above the seasonal high groundwater table, bedrock or other barrier layer.

If the infiltration rate of underlying site soils is greater than 1.0 inch/hour, consider designing an infiltration basin instead of a bioretention basin.

Bioretention basins are susceptible to leaching pollutants into sensitive waters or saturating soils adjacent to infrastructure. The designer should reference design criteria for setback distances.

Organic compost used in the filtration media should be tested and approved for use in a planting bed. Check local availability of suitable organic compost.

Wood mulch, if used in the bioretention basin around plantings, should be triple shredded hardwood mulch that interlocks better and is less susceptible to floating. A suitable depth is usually 3 inches.

The upper 6 inches of soil outside the infiltration cell should be amended to improve soils for planting. NDOT staff will provide guidance on any amendments.

Stabilize all basin outfalls that discharge into the bioretention basin to prevent scour and erosion. Stabilization at outfalls should also be designed to help distribute the flow uniformly across the basin.

The following Design Criteria table provides pretreatment criteria that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

Bioretention

DESIGN CRITERIA

Description	Value
Maximum Contributing Basin Area	5 acres
Maximum WQV Depth, D_{WQ}	1 foot
Maximum Depth of Basin, D (Full Depth That Includes Detention Volume)	2 feet
Infiltration Cell Depth, D_F	2 feet (minimum) 3 feet (maximum) (Depth ≤ Width)
Infiltration Cell Coefficient of Permeability, k	0.25 feet/hour typical (Based on sand-compost mix design)
Time for Infiltration	24 hours (48 hours maximum)
Typical Underdrain Pipe Separation	10 feet
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 10% of WQV (minimum)
Setback Distances	Surface Water – 50 feet Private Drinking Water Wells – 100 feet Public Drinking Water Supply Wells (Non-Community System) – 100 feet Public Drinking Water Supply Wells (Community System) – 500 feet Water Lines (Pressure) – 25 feet Water Lines (Suction) – 100 feet Property Lines – 5 feet Foundations (NDOT)* – 20 feet (assumes no basement) Foundations (Neighbors)* – 30 feet (assumes no basement)

* Add 10 feet to setback distance when foundations are lower in elevation than water quality feature or adjacent to a full depth basement.

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate the WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* **OR** use Exhibit 3.5 – Water Quality Volumes and Peak Discharges for Selected Acreages for sites under 5 acres.

Step 2: Size the infiltration cell surface area, A_F

The minimum surface area of the infiltration cell is based on Darcy's law:

$$A_F = \frac{WQV*D_F}{k*t*(D_F+0.5D_{WQ})}$$

A_F = infiltration cell surface area (ft^2)
D_F = depth of infiltration cell (ft)
D_{WQ} = height of WQV above the filter bed (ft)
k = coefficient of permeability (ft/hr)
(assume 0.25 ft/hr for infiltration cell)
t = bioretention basin drain time, hrs (design for 24 hours)

Step 3: Determine the length and width of the infiltration cell using the calculated minimum surface area and check that the width is not less than the depth.

$$L = A_F/W_F$$

W_F = width of the infiltration cell
L = length of the infiltration cell

Verify that $W \geq D$

Step 4: Check Underdrain Spacing.

Verify that underdrains are spaced evenly 10 feet apart per Design Criteria

Step 5: Calculate the area of the basin.

$$A = \frac{WQV}{D_{WQ}}$$

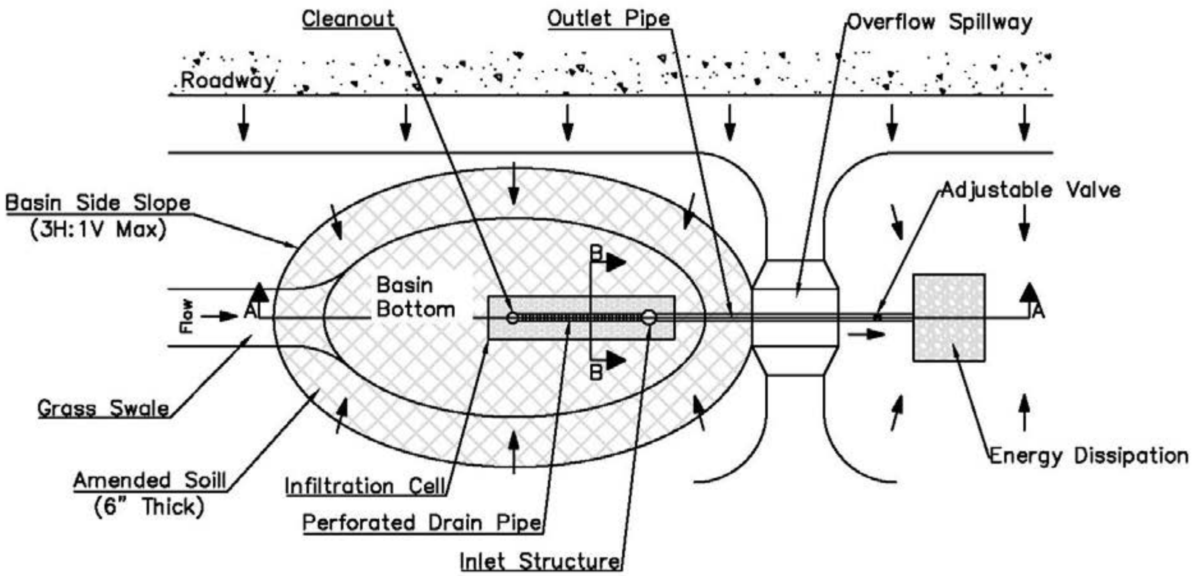
A = design area (ft^2)

Step 6: Check diversion or storage and routing of larger storm events

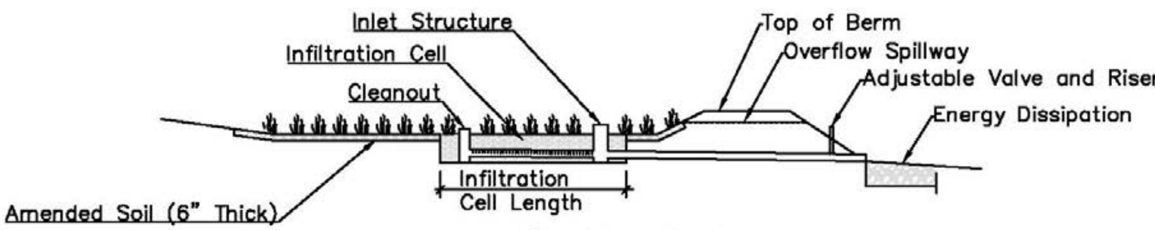
Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events. Design offline bioretention basins to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online bioretention basins for flow through and integrate any additional storage into the feature. Additional storage to reduce peak runoff will be added above the WQV.

Bioretention

DESIGN EXAMPLES



Bioretention Basin
No Scale



Section A–A
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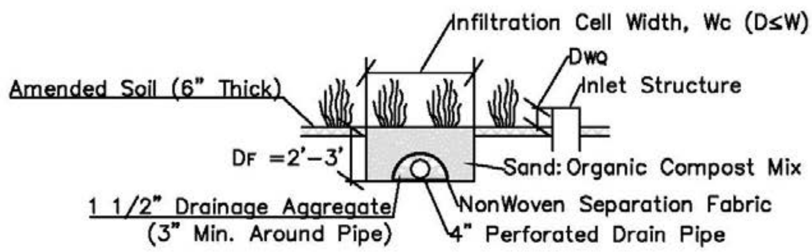

Section B–B
No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because bioretention basins are prone to failure when inundated with a high sediment load. Stabilize the upgradient contributing drainage area before putting bioretention basins into operation.
- Because inlet protection is often not adequate during construction of a bioretention basin, bypass stormwater away from the facility until vegetation is established and all construction-related sediment has been controlled for the best protection. Otherwise, the bioretention basin may be unusable immediately after implementation.
- Consider the space needed for pretreatment and any swale required for bypassed flow.
- If upgradient stabilization prior to construction is not possible and flow cannot be temporarily bypassed, provide erosion and sediment control protection for the infiltration basin.
- If the infiltration area is being used as a sediment basin during construction. the bottom elevation of the sediment basin should be a minimum of 2 feet above the future infiltration bed invert elevation so that the trench can be excavated in native soils.
- Protect infiltration areas from construction or other traffic during the course of construction. If this is not possible, take steps to reduce compaction of underlying soils.
- Excavate the bottom in such a manner as to leave the soil in a natural, unsmeared, and uncompacted condition.
- If infiltration areas do get compacted during construction, additional infiltration testing may be required.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for bioretention basins include providing litter control, monitoring sedimentation and erosion, and maintaining infiltration rates of the soil. Diversion structures, outlets, and forebays should also be inspected and maintained along with any pretreatment STFs.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect bioretention basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect bioretention basin 3 days (72 hours) after a major rainfall event to ensure proper drain time. • Inspect bioretention basin to ensure the intended vegetation is establishing well. Consider reseeding if needed. If a sand layer is used at the surface, remove any unwanted vegetation. • Inspect bioretention basin for erosion and any damage by equipment or vehicles after every major rainfall event. Repair as needed. • Inspect bioretention basin system for sediment buildup on the bottom of the basin and at any diversion structures, outlets, and forebays. Remove sediment as needed. • Remove trash and debris from the basin and any diversion structures, outlets, and forebays. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect bioretention basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect bioretention basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • If a sand layer is used at the surface remove any unwanted vegetation. • Mow grass to control weeds. • Inspect bioretention basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect bioretention basin for sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

Bioretention

Frequency	Inspection and Maintenance Activity
Annually (After NPDES Permit is closed)	• Inspect bioretention basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect bioretention basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. If a sand layer is used at the surface, remove any unwanted vegetation. • Mow grass to control weeds. • Inspect bioretention basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect bioretention basin for sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment build-up, and structural damage. Repair as needed. Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

RESOURCES AND REFERENCES

Arizona Department of Transportation, *ADOT Post-Construction Best Management Practices Manual*. July 2009.

Atlanta Regional Commission. *Georgia Stormwater Management Manual – Volume 2: Technical Handbook*. August 2001.

California Department of Transportation. *BMP Retrofit Pilot Program – Final Report.* January 2004.

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Upper White River Watershed Alliance. *Green Infrastructure Fact Sheets.* Downloaded February 2012.

Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual Volume 3.* 2010.

Median Filter

OVERVIEW

Source: Delaware Department of Transportation (DelDOT)

Definition

Media filters temporarily store runoff and filter it by gravity through a filter bed of sand or other filtration media. The filter bed is often confined within a concrete chamber and has underdrains that collect stormwater and discharge it away from the structure. A sediment chamber or sediment forebay provides pretreatment to the filter bed.

Benefits

- Suitable for highly urbanized areas and areas with direct runoff from impervious surface such as parking or drive lanes.
- Can fit in limited space and be located underground and under pavement.
- Suitable in areas where infiltration practices and groundwater contamination are a concern.

Overview Table

Associated Costs	L	M	H
Design			X
Construction			X
Maintenance			X
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients	X	X	
Heavy Metals		X	
Hydrocarbons		X	

Limitations

- Sand filters are appropriate for smaller drainage areas.
- Higher costs can be prohibitive.
- Pretreatment is encouraged to help reduce the potential for clogging.
- Media filters may create possible odor problems.
- Flotation of the structure should be considered in areas with a high groundwater table.

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Media filters rely on a filter bed made up of sand or other porous media to provide treatment. As described herein, the filter bed is confined within an underground concrete chamber, a "filtration chamber," and can be designed to accept flow from an outfall as a point discharge or as sheet flow from adjacent runoff. A surcharge zone above the filter bed is needed to allow head to develop and allow infiltration/filtration by gravity into an underdrain that discharges into a nearby channel, swale, or storm sewer.

Pretreatment of stormwater runoff is often provided by using a sedimentation chamber constructed alongside the filtration chamber underground. Furthermore, trash and debris are usually screened at the surface by a grate or other opening leading to the multi-chamber system.

Media filters are designed to capture the Water Quality Volume (WQV) and infiltrate that volume in a 24-hour period (48 hours maximum). Media filters can be designed offline or online but must allow for the bypass of runoff from larger storm events and in the event that the media becomes clogged.

STF COMPONENTS

Pretreatment – A sedimentation chamber constructed of reinforced Portland cement concrete is often used for pretreatment and is generally constructed as one unit with a filtration chamber. Pretreatment can also be achieved by using vegetated filter strips, grass swales, forebays, etc. Many factors dictate the types of pretreatment STFs suitable for your site, including available space, an offline or online system, soil characteristics, site topography, and cost. See design guidelines for additional information on the various STFs that provide pretreatment. A structural sedimentation chamber is shown in the Design Example.

Filtration Media – Filtration media is typically washed medium-grained concrete sand that meets the requirements of ASTM C-33. Other variations include sand mixed with a compost mixture or other media that enhances adsorption and/or absorption. Filtration media should be separated from adjacent soils by containment within a concrete chamber, an impermeable membrane, or a non-woven filter fabric. The aggregate should allow for permeability at rates that are not too slow or too fast.

Length-Width Dimensions – The area of the bottom of the media filter depends on the volume of storage needed, infiltration media properties, and available space. A filter area that is too small may clog prematurely. A larger filter area will decrease the frequency of maintenance. Length is defined as the longer axis in an x-y plane. Width is defined as the shorter axis in an x-y plane.

Filter Bed Depth, D_f – The depth of the filter bed generally depends on the volume of storage needed, infiltration media properties, and available space. It can also be limited by shallow groundwater or bedrock. Depth is measured from the bottom of a concrete chamber or excavation to the surface of the filter bed (the thickness of filtration media).

Median Filter

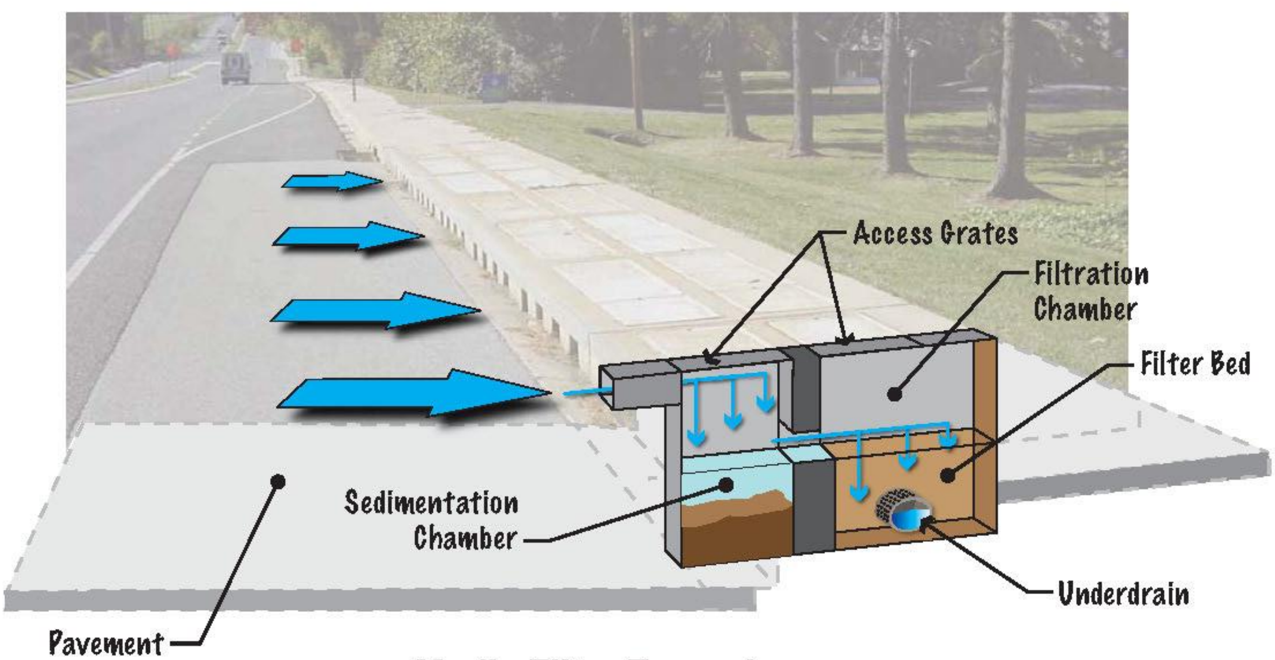
Sedimentation Chamber Volume – The sedimentation chamber volume is the volume of storage within the sedimentation chamber using a depth that is equivalent to the filter bed depth.

Surcharge Volume – The surcharge volume is the volume of storage above the filter bed in both the filtration chamber and sedimentation chamber.

Underdrain – The designer should include an underdrain positioned at the bottom of the filter bed that would allow water to drain out of the chamber or excavated area. The underdrain pipe would include a section of slot perforated PVC pipe lying horizontally on a thin layer of sand at the bottom of the chamber or excavation that connects to an adjustable valve and any riser pipe needed to access the valve. It should also include a section of solid PVC pipe that penetrates the concrete chamber wall and discharges to a sewer pipe or is daylighted. Underdrain pipes should be 6-inch diameter minimum.

Overflow Weir – An overflow weir is a weir section within an interior wall of a chamber designed to divert overflow storm events instead of allowing excessive surcharge within a structure. Energy dissipation should be provided where velocities and turbulence are a concern.

Outlet Chamber (for structural media filter) – An outlet chamber is a separate chamber within a media filter structure to collect the underdrain flow and overflow before discharging it by pipe.



Media Filter Example

Not to Scale

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. What are the site constraints?

Q. What impact will infiltration have on adjacent pavement, buildings, water bodies, groundwater, etc.?

Q. What type of pretreatment is appropriate?

Q. How does the proposed media filter interact with other design storms?

Q. Is the media filter designed for ease of maintenance?

Media filters are typically used in constrained areas. Check the available right-of-way and consider the various types of media filters. They are generally constructed below grade and can be constructed as an enclosed structure.

The designer will need to consider structural design elements not provided herein for any concrete structures and design so that the structure does not float if high groundwater levels are expected.

If the media filter is not contained in a concrete chamber, consideration should be given to the possibility of leaching pollutants into sensitive waters or saturating soils adjacent to infrastructure.

The following table provides pretreatment criteria that should be followed to the extent practical. An example of a sedimentation chamber is provided in the design examples. For other types of pretreatment, the designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

Check hydraulic grade lines to make sure there is enough head to allow for gravity filtration and not cause backup.

Minimum wall thickness for a concrete chamber is 6 inches, and minimum thickness of a PVC geomembrane liner is 30-mil.

The design should minimize re-suspension of any sediment in the sedimentation chamber and scour in the filtration media. Energy dissipation is needed for any pipes discharging into the structure.

Design the underdrain system for a 12-hour drawdown time. An orifice plate or a pipe is typically used to control drawdown time. A valve may be used to help control discharge rates if needed. Slot perforations sizes in the underdrain should be checked for potential loss of material or clogging. Cleanouts should also be provided.

An outlet chamber should be used to collect discharge from the underdrains and any overflow within a structure. Additional flow restrictions may be incorporated to provide additional detention within or downstream from the structure.

Median Filter

DESIGN CRITERIA

Description	Value
Maximum Contributing Basin Area	2 acres (online – up to 10-year storm event) 5 acres (offline – WQV only)
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Depth of Filter Bed, D_f	1.5 feet (2 feet maximum) (Note: Underdrain should be positioned below this depth)
Minimum Total Storage Volume	0.75 * WQV (Exclude any storage in filter bed)
Surcharge Volume	0.5 * WQV
Sedimentation Volume	0.25 * WQV
Sedimentation Chamber Area, A_s	Same as Filter Bed Surface Area
Coefficient of Permeability for Filtration Media	0.15 feet/hour (sand)
Filter Bed Drain Time	24 hours (48 hours maximum)
Maximum Underdrain Pipe Separation	10 feet
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 25% of WQV (minimum)

Median Filter

DESIGN PROCEDURE

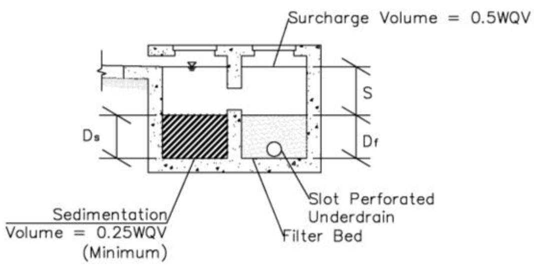
Step 1: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* **OR** use Exhibit 3.5 - Water Quality Volumes and Peak Discharges for Selected Acreages for sites under 5 acres.

Step 2: Calculate the surcharge depth and size the filter bed surface area, A_f

Given that the surcharge volume above the filter bed is approximately one half of the total surcharge volume (1/2 of 0.5*WQV) calculate the surcharge depth.

$$V_{sf} = 0.25*WQV = A_f*S$$



V_{sf} = surcharge volume above the filter bed (ft^3)
S = surcharge depth (ft)
A_f = filter bed surface area (ft^2)

Area of the filter bed is found using Darcy's equation.

$$A_f = \frac{WQV*D_f}{k*t*(D_f+0.5*S)}$$

A_f = filter bed area (ft^2)
D_f = depth of filter bed (ft) (1.5' typical – 2' maximum)
k = coefficient of permeability (ft/hr)
(assume 0.15 ft/hr for sand filtration media)
t = drain time (hours) (design for 24 hours)

Use Darcy's equation for filter bed surface area and solve for surcharge depth, S.

$$0.25*WQV = \frac{WQV*D_f*S}{k*t*(D_f+0.5*S)}$$

$$\frac{0.25*k*t}{D_f} = \frac{S}{(D_f+0.5*S)}$$

$$S = 0.25*k*t + \frac{0.125*k*t*S}{D_f}$$

$$S*\left(\frac{D_f - 0.125*k*t}{D_f}\right) = 0.25*k*t$$

$$S = \frac{0.25*k*t*D_f}{(D_f - 0.125*k*t)}$$

Plug the calculated surcharge depth back into Darcy's equation to size the filter bed surface area, A_f.

Step 3: Size the sedimentation chamber

The sedimentation chamber area will be equal to the area of the filter bed.

$$A_s = A_f$$

A_s = sedimentation chamber area (ft^2)

The depth used to calculate sedimentation volume, equals the depth of the filter bed.

$$D_s = D_f$$

D_s = depth for sedimentation volume (ft) (measured from the floor of the chamber to the top of the filter bed)

The sedimentation chamber typically has a 2:1 length to width ratio (minimum). The filter bed surface area dimensions typically match those of the sedimentation chamber.

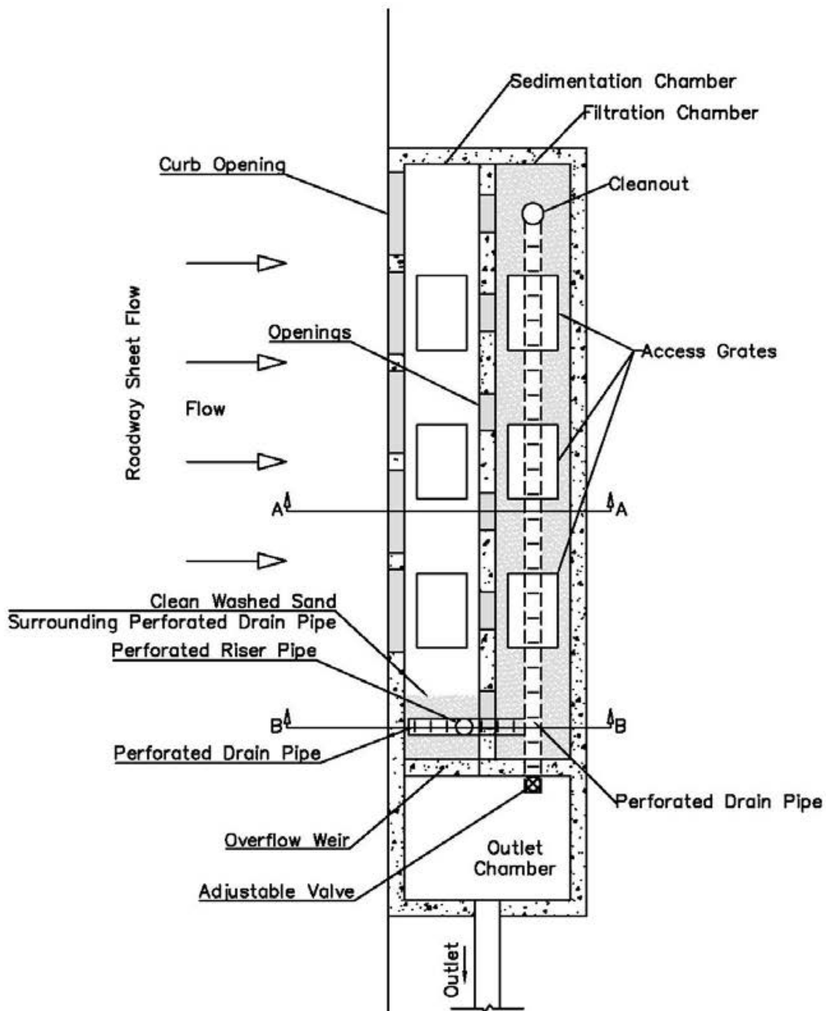
Step 4: Check diversion or storage and routing of larger storm events

Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events. Media filters are generally not designed to handle more than a 10-year storm event. They are also not typically designed with additional storage to reduce peak runoff.

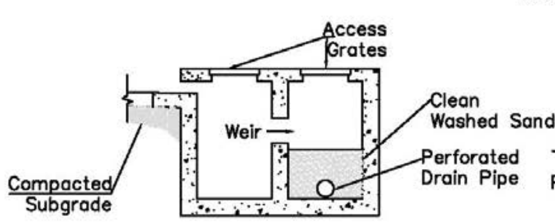
Media filters need to be designed to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Size the overflow weir at the end of the sedimentation chamber to handle excess inflow, set at the WQV elevation.

Median Filter

DESIGN EXAMPLES



Media Filter
with Curb Openings
(Upper Inlet)
No Scale



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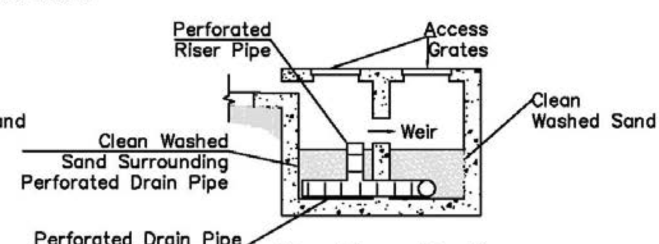
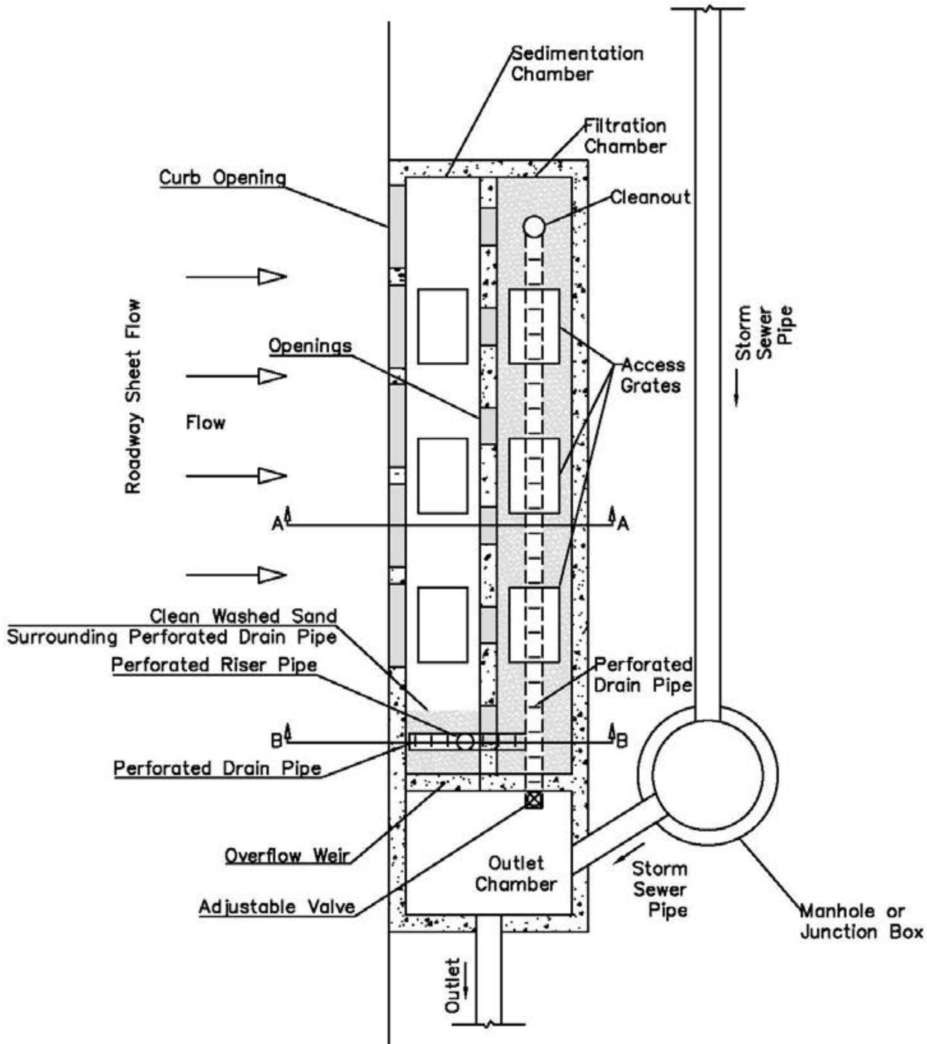
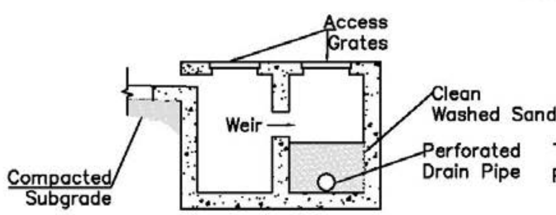


Section B–B
No Scale

Median Filter



Media Filter
with Curb Openings
(Intermediate Inlet)
No Scale



Section A–A
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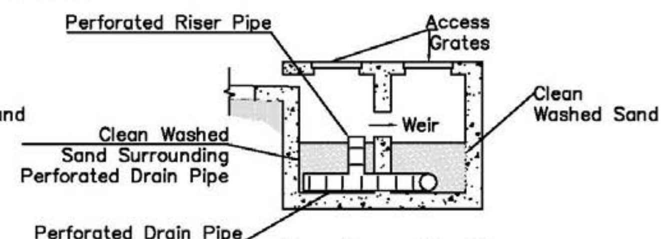
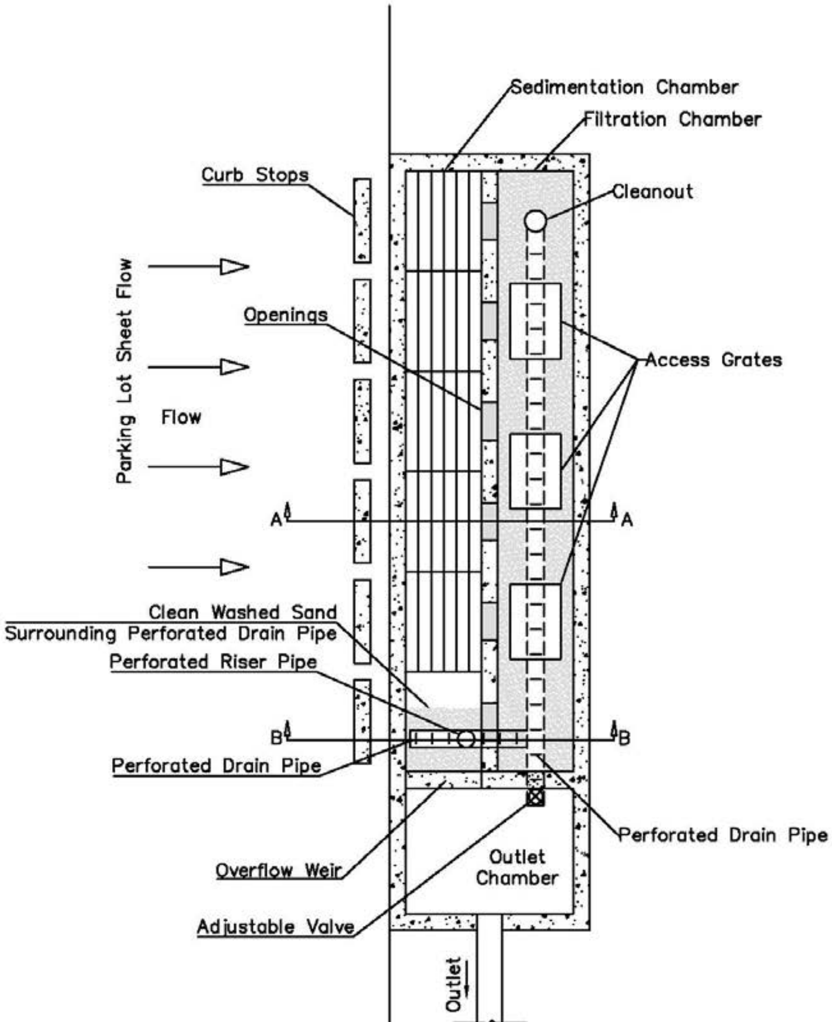
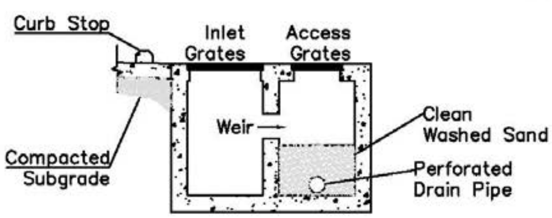


Section B–B
No Scale

Median Filter



Media Filter
with Grate Openings
(Upper Inlet)
No Scale



Section A–A
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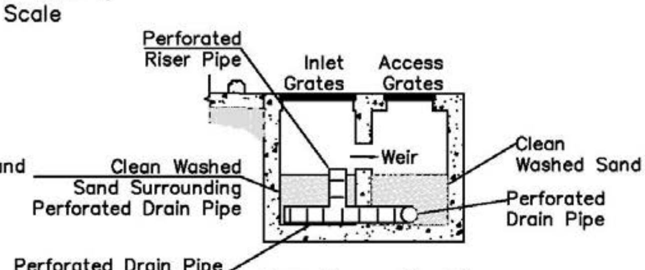

Section B–B
No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because media filters are prone to failure when inundated with a high sediment load.
- Stabilize the upgradient contributing draining area before putting the media filters into operation.
- If it is not possible to stabilize the upgradient before beginning construction and flow cannot be temporarily bypassed, provide erosion and sediment control protection for the media filter and consider adding the filtration media after the site has been stabilized.
- Because inlet protection is often not adequate during construction of a media filter, the best protection may be to bypass stormwater away from the facility until vegetation is established and all construction-related sediment has been controlled. Otherwise, the media filter may be unusable immediately after implementation.
- Consider the space needed for any measures required for bypassed flow.
- Check that the structure is watertight before adding the sand bedding by filling the structure with water and checking the water volume after 24 hours. The water volume loss should not exceed 5 percent.
- Make sure that the filtration media is hydraulically compacted by filling the filtration media to the crossover weir height, filling the sedimentation chamber and sand bed full of water, and allowing the filtration media to consolidate as the water drains down. After 24 hours, add filtration media backup to the crossover weir height.
- Consider trench stability and safety during construction. Refer to the Occupational Safety and Health Administration trench safety standards.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for media filter systems generally include removing litter and sediment and maintaining filtration rates in the filter bed. Inlets, diversion structures, underdrains, and outlets should also be inspected and maintained.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect filter bed for any surface ponding or backups. • Check observation wells 2 days (48 hours) after a major rainfall event to ensure proper drain time. • Inspect media filter system for sediment buildup in the sedimentation chamber or forebay and on the filter bed surface. Remove sediment when depth in sediment chamber or forebay exceeds 6 inches. Remove sediment on the filter bed surface together with the upper 2 to 3 inches of filtration media at same time as sediment chamber. Replace filtration media that is lost in the process. • Inspect the media filter for trash and debris at the inlets, outlet, and any diversion structure. Remove accumulated trash and debris as needed. • Inspect the contributing area for stabilization and erosion. Repair and reseed as needed.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect filter bed for any surface ponding or backups. • Inspect media filter system for sediment buildup in the sedimentation chamber or forebay and on the filter bed surface. Remove sediment when depth in sediment chamber or forebay exceeds 6 inches. Remove sediment on the filter bed surface together with the upper 2 to 3 inches of filtration media at same time as sediment chamber. Replace filtration media that is lost in the process. • Inspect the media filter for trash and debris at the inlets, outlet, and any diversion structure. Remove accumulated trash and debris as needed. • Inspect the contributing area for stabilization and erosion. Repair and reseed as needed. • Inspect sedimentation chamber permanent pool for any leaks. Remove sediment and repair as needed. • Inspect for any damage, cracking, or deterioration of concrete. Repair as needed. • Inspect for noticeable odors.

Median Filter

Frequency	Inspection and Maintenance Activity
Annually (After NPDES Permit is closed)	• Inspect filter bed for any surface ponding or backups. • Inspect media filter system for sediment buildup in the sedimentation chamber or forebay and on the filter bed surface. Remove sediment when depth in sediment chamber or forebay exceeds 6 inches. Remove sediment on the filter bed surface together with the upper 2 to 3 inches of filtration media at same time as sediment chamber. Replace filtration media that is lost in the process. • Inspect the media filter for trash and debris at the inlets, outlet, and any diversion structure. Remove accumulated trash and debris as needed. • Inspect the contributing area for stabilization and erosion. Repair and reseed as needed. • Inspect sedimentation chamber permanent pool for any leaks. Remove sediment and repair as needed. • Inspect for any damage, cracking, or deterioration of concrete. Repair as needed. • Inspect for noticeable odors.

RESOURCES AND REFERENCES

Arizona Department of Transportation, *ADOT Post-Construction Best Management Practices Manual*. July 2009.

Atlanta Regional Commission. *Georgia Stormwater Management Manual – Volume 2: Technical Handbook*. August 2001.

California Department of Transportation. *BMP Retrofit Pilot Program – Final Report.* January 2004.

California Department of Transportation. *Storm Water Quality Handbook – Project Planning and Design Guide*. July 2010.

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The Low Impact Development Center, Inc.

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Nebraska Department of Environmental Quality. *Title 124 - Rules and Regulations for the Design, Operation and Maintenance of Onsite Wastewater Treatment Systems.* December 2007.

Nebraska Department of Transportation. *Drainage Design and Erosion Control Manual.* Current Edition.

State of New Jersey. *New Jersey Stormwater Best Management Practice Manual.* March 2003.

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United States Environmental Protection Agency. *Storm Water Technology Fact Sheet - Infiltration Trench.* September 1999.

Upper White River Watershed Alliance. *Green Infrastructure Fact Sheets.* Downloaded February 2012.

Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual Volume 3.* 2010.

Extended Dry Detention

OVERVIEW

Source: Colorado Department of Transportation (CDOT)

Definition

Extended dry detention basins provide temporary storage of stormwater runoff that is released over a specified time. They are typically designed to attenuate the water quality volume and larger storm events. An outlet structure controls the rate of flow out of the basin. Storage can be above or below ground.

Benefits

- Suitable for large drainage areas (typically greater than 10 acres).
- Suitable for low permeability soils.
- Flexible system that provides water quality benefits, as well as detention.
- May be suitable as a retrofit to standard detention basins.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction	X		
Maintenance	X		
Pollutant Removal	**L**	**M**	**H**
Suspended Solids		X	
Nutrients	X	X	
Heavy Metals		X	
Hydrocarbons	X		

Limitations

- Site topography may dictate size and shape.
- Pollutant removal efficiency is lower than most infiltrative measures.
- Design standards and requirements may increase with larger basins and/or higher embankments (see Dam Safety regulations).
- Extended dry detention is not typically suitable in areas with a high groundwater table or groundwater contamination issues.

Extended Dry Detention

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Extended dry detention basins provide temporary storage of runoff and are frequently constructed with a multi-stage outlet structure to draw the Water Quality Volume (WQV) down over a minimum of 24 hours (72 hours maximum). The extended drawdown time is provided to improve pollutant removal efficiencies and is the primary difference between a standard detention basin and an extended detention basin. Basins are designed to fully drain between storm events.

Extended dry detention basins are typically landscaped with non-invasive, preferably native, vegetation that is suitable for the dry and wet cycles of the basin. They can be vegetated with turf grass, as well, if needed.

The removal efficiencies of extended dry detention basins, while greater than standard dry detention, are not as great as infiltration type STFs. Pretreatment is desirable to help control sediment from being deposited in the basin. Permanent micro-pools may be added to the basin to provide additional treatment.

The designer is referred to wet detention or stormwater wetland systems if a constant flow source is evident or the seasonal high groundwater table rises above the bottom of the proposed basin.

Extended dry detention basins are typically designed online though offline systems may yield higher removal efficiencies. Extended detention basins can accept sheet flow from adjacent ground or flow from an outfall as a point discharge. Energy dissipation may be required for any point discharge to avoid erosion and spread the flow out.

STF COMPONENTS

Pretreatment STF – Pretreatment can be achieved by using vegetated filter strips, grass swales, forebays, etc. Forebays are recommended for end-of-pipe treatment. Many factors dictate the types of pretreatment STFs suitable for the site, including available space, an offline or online system, soil characteristics, site topography, and cost. See design guidelines for additional information about the various STFs that provide pretreatment.

Berm (Embankment) – A berm is a compacted earthen ridge designed to capture and detain stormwater flows in the extended dry detention basin.

Extended Detention Depth **–** The depth of water stored in the extended dry detention basin based on the Water Quality Volume (WQV) as defined in Chapter 3 of the Nebraska Department of Transportation (NDOT) *Drainage Design and Erosion Control Manual*. It generally depends on the available area and the volume of storage needed.

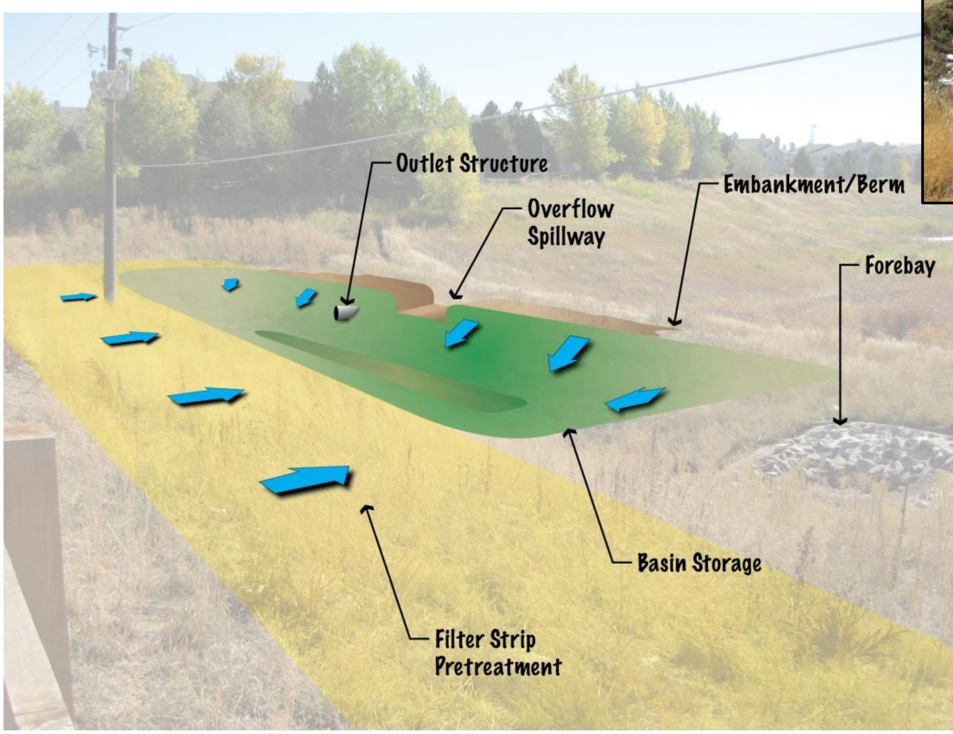
Total Depth **–** The total depth of the extended dry detention basin from the lowest elevation of the bottom to the top of the berm or embankment. It includes a minimum of 1 foot of freeboard from the design water surface elevation in the emergency spillway to the top of the berm or embankment.

Extended Dry Detention

Outlet Structure (Principal Spillway) – An outlet structure is a standpipe or other structure designed to draw down stormwater that is stored in the basin. In an extended dry detention basin, a multi-stage structure is typically needed. A pipe or an orifice opening is typically provided at the bottom of the basin to draw down the WQV. Other controls (orifice or weir) are provided to draw down larger storm events at predetermined maximum rates.

Overflow Spillway (Auxiliary or Emergency Spillway) – An overflow spillway is a protected area along a berm or an excavated channel designed to convey overflow storm events instead of allowing overtopping of the berm. Consideration should be given to the velocity of flow over the spillway, at any intermediate grade changes and at the toe. Protection or energy dissipation should be provided where velocities and turbulence are a concern.

Outlet Structure Example



Extended Dry Detention Example
Not to Scale

Extended Dry Detention

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Is the site suitable for extended dry detention?

Q. What can be done if the bottom slope is fairly steep?

Q. What type of pretreatment is appropriate?

Q. Are there other considerations for large basins?

The size and depth of the extended dry detention basin largely depends on the natural topography of the site. The designer should try to minimize excavation but will need enough soil to construct an embankment.

The bottom of the basin should be gently sloped to reduce the potential for erosion from incoming stormwater runoff. A low flow channel, preferably natural, may be needed to route trickle flow. A low flow channel that is serpentine in form may help decrease the channel slope and reduce velocities. Short circuiting between a point discharge source and the outlet structure should be avoided. An earthen berm or other type of baffle may be needed to prevent this.

If the infiltration rate of underlying site soils is greater than 1.0 inch/hour, consider designing an infiltrative STF instead of an extended dry detention. If the bottom of the extended dry detention basin is at or below the seasonal high groundwater table, consider designing a wet detention basin or stormwater wetland.

For soils that are compacted due to the placement of fill material or construction equipment, the upper 6 inches of soil on the bottom and sides of the basin may need to be amended to improve soils for planting. NDOT staff will provide guidance on any amendments.

The following table identifies pretreatment criteria that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

The Nebraska Department of Natural Resources (NDNR) has jurisdiction over “dams” as defined in Chapter 46, Article 16: Safety of Dams and Reservoirs and Title 458, Nebraska Administrative Code, Chapters 1-13; NDNR Rules for the Safety of Dams and Reservoirs. Verify whether or not your project falls within NDNR jurisdiction and may need to meet NDNR dam design standards; particularly for embankments 6 feet high or greater and those with an impounding capacity at maximum storage elevation greater than 15 acre-feet

Required elements include trash racks, anti-seepage collars, and energy dissipation.

The design should consider the volume of sedimentation that will occur. The designer should assume that a 10 percent reduction in volume will occur before maintenance. The forebay and micro-pool should be used to account for sediment volume. Designers should include design dimension and depth of sediment markers in the detention basin design.

Extended Dry Detention

DESIGN CRITERIA

Description	Value
Minimum Contributing Basin Area	5 acres
Maximum Extended Detention Depth	3 feet
Maximum Berm Height	5 feet
Minimum Basin Bottom Slope	0.5% - 1.0%
Maximum Basin Bottom Slope	2% (typical cross slope 2%)
Extended Detention Volume (V_{EX})	1.1 * WQV (accounts for sedimentation)
Minimum WQV Drawdown Time	24 hours (72 hours maximum)
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 10% of WQV (minimum)

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate the WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*.

Step 2: Calculate extended detention volume

Multiply WQV by 1.1 to account for storage lost to sediment dissipation.

$$V_{EX} = WQV*1.1$$

V_{EX} = extended detention volume (cf)

Step 3: Design extended detention pond

Pond design is an iterative process. Estimate the minimum area needed using the extended detention volume and the maximum extended detention basin depth of three (3) feet.

$$A_{MIN} = \frac{V_{EX}}{3}$$

A_{MIN} = minimum area for extended detention (sf)

Approximate the location of a berm or control point for overflow and calculate the actual volume taking into consideration the maximum depth allowed, existing grades, and any excavation that may be needed. If the volume is insufficient or maximum depths are exceeded, re-evaluate the location of the berm and/or excavation and calculate the volume again. Repeat this process until all applicable design criteria, including minimum bottom slope, are met.

Extended Dry Detention

Step 4: Design low flow channel

Design the low flow channel meet minimum slope using a meandering flow path if needed to meet that criterion. Typical low flow channel dimensions are shown as part of Design Examples.

Step 5: Size the orifice for WQV drawdown

Calculate the orifice size for the Minimum WQV Drawdown Time provided in the Design Criteria table above using the average discharge rate and average hydraulic head.

Find the average discharge rate:

$$Q = WQV/t/3600$$

Q = average orifice discharge rate (cfs)
t = WQV drawdown time (hours)

Find the orifice area:

$$A = Q/[C*(2*g*h)^{0.5}]$$

A = orifice area (ft^2)
C = orifice discharge coefficient, dimensionless (0.60 typ)
g = acceleration of gravity (ft/s^2)
h = average hydraulic head (ft)
(height measured from orifice invert to midpoint of extended detention depth – assumes orifice is small relative to total height)

Find the orifice diameter:

$$d = (4*A/3.14)^{0.5}*12$$

d = orifice diameter (in)

Step 6: Check diversion or storage and routing of larger storm events

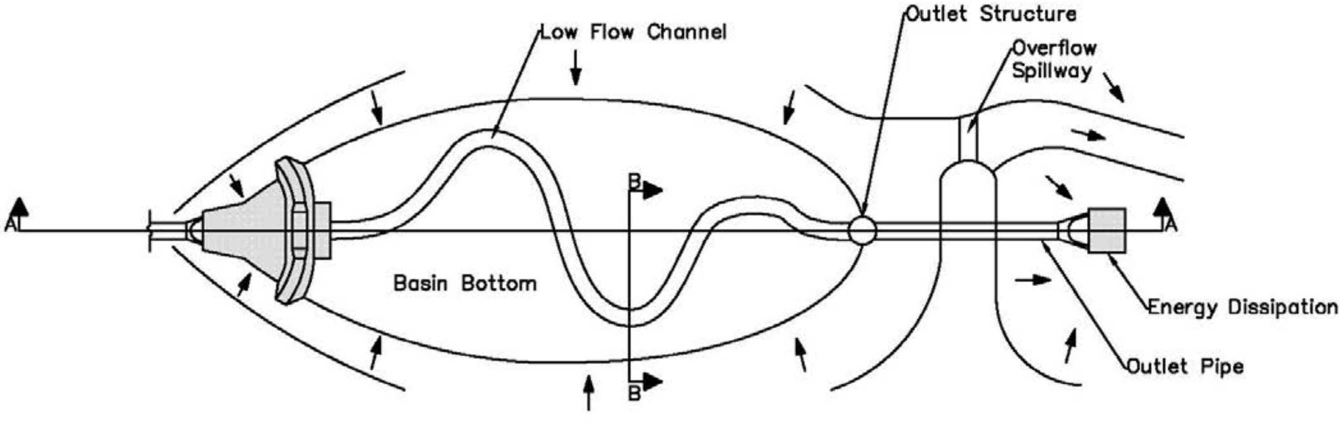
Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events.

Design offline extended dry detention basins to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online extended dry detention basins for the WQV and integrate any additional storage into the feature. Additional storage to reduce peak runoff will be added above the WQV.

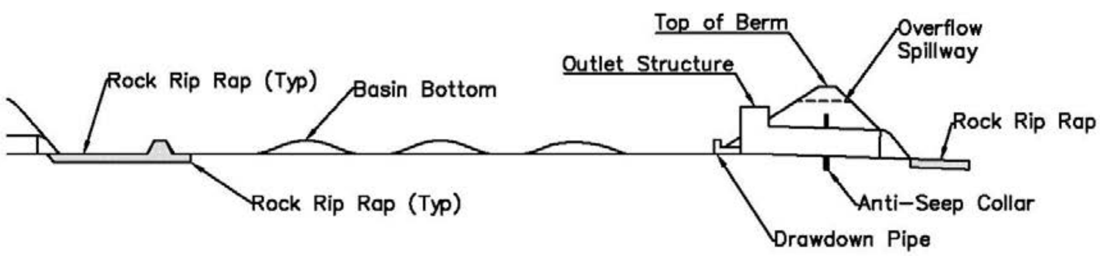
NDOT recommends the use of the NRCS TR-55 procedure (See Chapter 1) to determine the runoff volume and discharge rate used to design a detention system for a project site. The Rational Method and Modified Rational Method will not be acceptable.

Extended Dry Detention

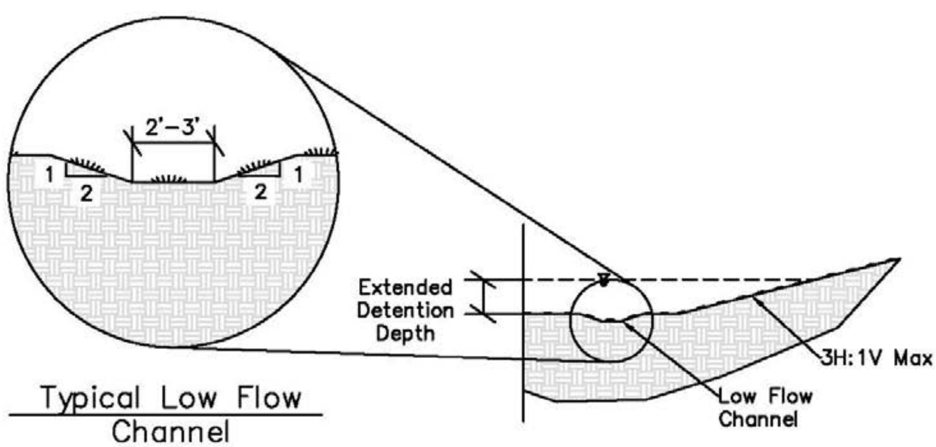
DESIGN EXAMPLES



Extended Dry Detention
No Scale



Section A–A
No Scale



Typical Low Flow Channel
No Scale

Section B–B
No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because, unless accounted for in the design, inundation with a high sediment load will decrease the intended storage capacity and increase maintenance costs.
- Depending on the size of the basin, inlet protection alone may not be adequate during construction. Intermediate erosion control upstream from the inlet will help reduce sedimentation in the basin.
- If the extended detention basin area is being used as a sediment basin during construction, particularly as wet sediment basin, construct the bottom elevation of the sediment basin higher in elevation than the future basin bottom elevation so that saturated soils may be removed, along with accumulated sediment, and so that seeding and/or planting can take place.
- Take care to limit compaction of the bottom of the basin area from construction or other traffic during the course of construction. If this is not possible, take steps to reduce compaction of underlying soils.
- Temporary irrigation of the extended dry detention basin may be needed to help with the establishment of vegetation.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for an extended dry detention basin include providing litter control, monitoring erosion and sedimentation, and maintaining outlet control structures. Diversion structures, discharge points, and forebays should also be inspected and maintained, along with any other pretreatment STFs.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect extended dry detention basin for any unintended surface ponding or indicators that water has ponded for an extended period of time. • Check extended dry detention basin 3 days (72 hours) after a major rainfall event to ensure drainage of the basin. • Inspect extended dry detention basin to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect extended dry detention basin for erosion and any damage by equipment or vehicles. Repair as needed. • Inspect extended dry detention basin for sediment buildup on the bottom of the basin and at any diversion structures, outlets, and forebays. Remove sediment as needed. • Remove trash and debris bi-weekly from the basin and any diversion structures, outlets, and forebays. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect extended dry detention basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect extended dry detention basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect extended dry detention basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect extended dry detention basin for sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

Extended Dry Detention

Frequency	Inspection and Maintenance Activity
Annually (After NPDES Permit is closed)	• Inspect extended dry detention basin for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect extended dry detention basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect extended dry detention basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect extended dry detention basin for sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

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Extended Dry Detention

Wet Detention

OVERVIEW

Source: City of Omaha

Definition

Wet detention basins retain a permanent pool of water and provide temporary storage for stormwater runoff above the permanent pool that is released over a specified time. They are typically designed to attenuate larger storm events as well. An outlet structure controls the rate of flow for the various conditions.

Benefits

- Suitable for large drainage areas.
- Effective in removing pollutants, including dissolved solids.
- Offers a flexible system that can provide water quality benefits as well as detention in some cases.
- Suitable for sites with shallow groundwater.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction	X		
Maintenance	X		
Pollutant Removal	**L**	**M**	**H**
Suspended Solids		X	X
Nutrients		X	
Heavy Metals		X	
Hydrocarbons		X	

Limitations

- Site topography may dictate size and shape.
- A minimum drainage area is needed to maintain a permanent pool (or a groundwater source is needed).
- Design requirements may increase with larger basins and/or higher embankments.
- Mosquito and algae problems may develop.
- Wet detention requires moderate to high permeability soils (unless stable groundwater sources are present)

Wet Detention

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Wet detention basins are generally formed when constructing an embankment to capture runoff or excavating to detain runoff and provide storage for a permanent pool of water. The minimum design volume of water in the permanent pool is based on a fraction of the Water Quality Volume (WQV). Temporary storage is provided above the permanent pool elevation to capture the remaining WQV and to release that volume over a minimum of 24 hours (72 hours maximum).

Wet detention basins are often constructed with a multi-stage outlet structure to attenuate peak flow from major storm events in addition to the WQV. The extended drawdown time improves pollutant removal efficiencies and reduces the size of the typical wet detention basin.

Wet detention basins are typically landscaped around the perimeter of the pool with non-invasive, preferably native, vegetation that is suitable for the wet and dry cycles associated with extended detention. The shallow area along the shoreline is typically suitable for wetland plantings and can improve water quality, habitat, and aesthetics.

Pretreatment is desirable to help improve removal efficiencies and contain some of the sediment for easier removal at a later time.

STF COMPONENTS

Pretreatment STF – Pretreatment can be achieved by using vegetated filter strips, grass swales, forebays, etc. Forebays are recommended for end-of-pipe treatment. Many factors dictate the types of pretreatment STFs suitable for the site, including available space, an offline or online system, soil characteristics, site topography, and cost. See design guidelines for additional information on the various STFs that provide pretreatment.

Soils – The types of soils on site will partially determine whether the site is suitable to maintain a permanent pool. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics for design.

Berm (Embankment) – A berm is a compacted earthen ridge designed to capture and detain stormwater flows in the wet detention basin.

Permanent Pool Depth – The permanent pool depth represents the depth of storage in the wet detention basin. It generally depends on the available area and the volume of storage needed.

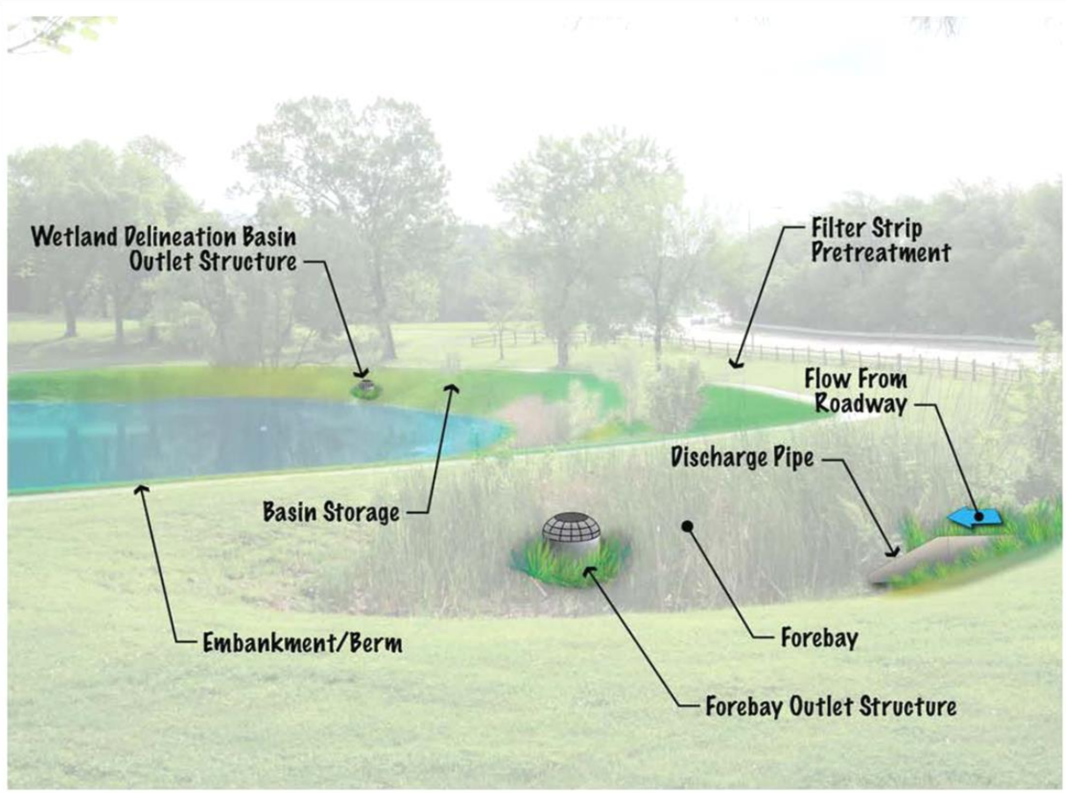
Extended Detention Depth – The extended detention depth represents the depth of water above the permanent pool elevation that is temporarily stored and released over a specific period of time.

Wet Detention

Total Depth – The total depth of the wet detention basin from the lowest elevation of the bottom to the top of the berm or embankment. For a basin that is online, this depth includes the capacity required to handle runoff from the design storm frequency defined in Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* and any additional storage for peak flow reduction. It includes a minimum of 1 foot of freeboard from the design water surface elevation in the emergency spillway to the top of the berm or embankment.

Outlet Structure (Principal Spillway) – An outlet structure is a standpipe or structure designed to draw down stormwater that is stored in the basin. In a wet detention basin, a multi-stage structure is typically needed. A pipe or an orifice opening is typically provided at the bottom of the basin to draw down the WQV. Other controls (orifice or weir) are provided to draw down larger storm events at predetermined maximum rates.

Overflow Spillway (Auxiliary or Emergency Spillway) – An overflow spillway is a protected area along a berm or an excavated channel designed to convey overflow storm events instead of allowing overtopping of the berm. Consideration should be given to the velocity of flow over the spillway, at any intermediate grade changes, and at the toe. Protection or energy dissipation should be provided where velocities and turbulence are a concern.



Wet Detention Example
Not to Scale

Wet Detention

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Is there a source of water to maintain a permanent pool?

Q. What about impacts on streams and dam safety requirements?

Q. How does the proposed wet detention basin interact with other design storms?

Q. What are some other safety features to consider?

Site characteristics are very important when designing a wet detention basin and should be taken into consideration early in the design process.

Check available right-of-way when determining the footprint. Consider the ramifications of standing water adjacent to the roadway and any safety considerations, such as locating the stormwater wetland outside clear recovery zones and whether fencing is needed.

Site topography dictates whether an embankment can be constructed to create the storage need or whether excavation is necessary.

Wet detention basins need a drainage area of sufficient size to maintain a permanent pool. The minimum suggested ratio of drainage area to pond volume (acres to acre-feet) is 15:1; however, the ability to maintain the pool varies from site to site and may be closer to 60:1 in western Nebraska. Calculations should be done to check the water balance. Groundwater is a source that should be considered in these calculations.

The Nebraska Department of Natural Resources (NDNR) has jurisdiction over “dams” as defined in Chapter 46, Article 16: Safety of Dams and Reservoirs and Title 458, Nebraska Administrative Code, Chapters 1-13; NDNR Rules for the Safety of Dams and Reservoirs. Verify whether or not your project falls within NDNR jurisdiction and may need to meet NDNR dam design standards; particularly for embankments 6 feet high or greater and those with an impounding capacity at maximum storage elevation greater than 15 acre-feet.

Wet detention basins are susceptible to leaching pollutants into sensitive waters or saturating soils adjacent to infrastructure. The designer should reference design criteria for setback distances.

A 25-foot vegetative buffer (minimum) is required to help provide additional water quality benefits, shoreline protection, and habitat. A 10-foot to 15-foot-wide zone of shallow water (0 to 18 inches in depth) along the shoreline is required for similar reasons.

A 10-foot to 15-foot-wide safety bench above the shoreline is required if sideslopes are steeper than 1V:4H. Any mowing should stop on the safety bench.

The following table provides pretreatment criteria that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

Wet Detention

DESIGN CRITERIA

Description	Value
Minimum Contributing Basin Area	10 acres (verify water budget to ensure the design elevation for the permanent pool is maintained)
Watershed Area to Permanent Pool Volume Ratio	See Figure 1 for preliminary design (verify water budget to ensure the design elevation for the permanent pool is maintained)
Typical Pool Length to Width Ratio	2:1 or greater
Permanent Pool Depth	3 feet to 7 feet (Minimum 50% of area ≥ 5 feet)
Maximum Permanent Pool Depth	10 feet
Maximum Extended Detention Depth	3 feet (above permanent pool)
Minimum Permanent Pool Volume (V_{PP})	0.6 * WQV
Extended Detention Volume (V_{EX})	0.5 * WQV
Minimum V_{EX} Drawdown Time	24 hours (72 hours maximum)
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 10% of WQV (minimum)
Setback Distances	Surface Water – 50 feet Private Drinking Water Wells – 100 feet Public Drinking Water Supply Wells (Non-Community System) – 100 feet Public Drinking Water Supply Wells (Community System) – 500 feet Water Lines (Pressure) – 25 feet Water Lines (Suction) – 100 feet Property Lines – 5 feet Foundations (NDOT)* – 20 feet (assumes no basement) Foundations (Neighbors)* – 30 feet (assumes no basement)

* Add 10 feet to setback distance when foundations are lower in elevation than water quality feature or adjacent to a full depth basement.

Wet Detention

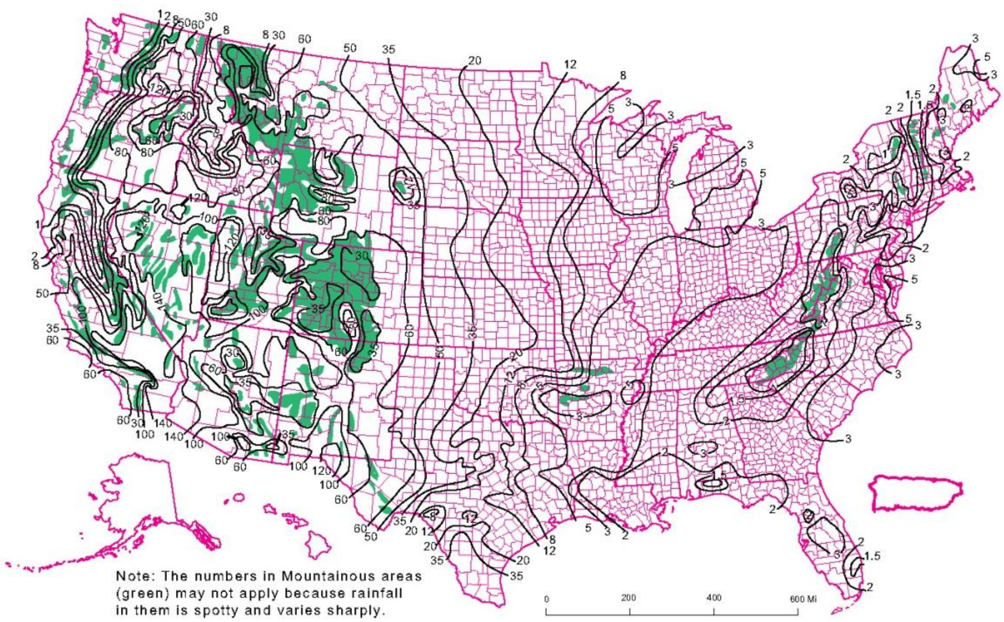


Figure 1. Guide for Estimating Required Drainage Area (Acres) for Each Acre-Foot of Storage in Basin

Wet Detention

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*.

Step 2: Check to see whether the drainage can support the permanent pool in a stormwater wetland.

Using Figure 1, determine the approximate permanent pool volume sustained for a given drainage area for the project area.

$$V_{PMax} = \frac{A_{Total}}{A_{AF}} * 43{,}560$$

V_{PMax} = maximum permanent pool volume (ft^3)
A_{Total} = actual drainage area (ac)
A_{AF} = minimum drainage area per ac-ft of storage (ft^{-1}) (From Figure 1)

Check to see whether estimated WQV is less than V_{PMax}

If WQV ≤ V_{PMax}, it's likely that the drainage area can sustain the minimum permanent pool. Verify this by calculating the water budget.

If WQV > V_{PMax}, it's less likely that the drainage area can sustain the minimum permanent pool. Verify this by calculating the water budget or selecting another type of STF.

Step 3: Allocate WQV to determine volumes and areas of the permanent pool and extended detention

Use the ratios provided in the design criteria table above to determine the volumes of the permanent pool and extended detention.

$$V_{PP} = 0.6*WQV$$

V_{PP} = permanent pool volume (cf)

$$V_{EX} = 0.5*WQV$$

V_{EX} = extended detention volume (cf)

Once the volumes have been determined, use the typical depths associated with each zone to lay out the stormwater wetland. Take into consideration the typical permanent pool length to width ratio (2:1) and provide a 10'-15' wide shallow water zone (< 18" depth) around the perimeter of the permanent pool as illustrated in Design Examples, Section B-B.

Provide a 25' wide vegetative buffer along the edge of the permanent pool. If slopes adjacent to the extended detention pond volume are steeper than 1V:4H, also provide a 10' to 15' wide safety bench (≤ 4%).

Wet Detention

Step 4: Size the orifice for WQV drawdown

Use the average discharge rate and the average hydraulic head to calculate the orifice size for the Minimum WQV Drawdown Time provided in the design criteria table above.

Find the average discharge rate:

$$Q = V_{EX}/t/3600$$

Q = average orifice discharge rate (cfs)
t = V_{EX} drawdown time (hours)

Find the orifice area:

$$A = Q/[C*(2*g*h)^{0.5}]$$

A = orifice area (ft^2)
C = orifice discharge coefficient, dimensionless (0.60 typical)
g = acceleration of gravity (ft/s^2)
h = average hydraulic head (ft)
(height measured from orifice invert to midpoint of extended detention depth – assumes orifice is small relative to total height)

Find the orifice diameter:

$$d = (4*A/3.14)^{0.5}*12$$

d = orifice diameter (in)

Step 5: Check diversion or storage and routing of larger storm events

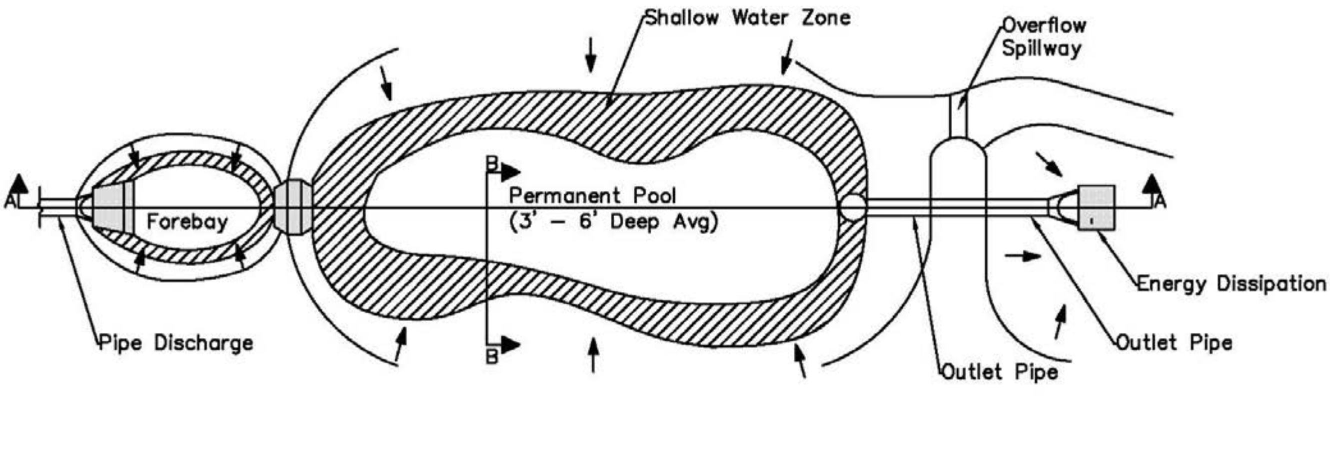
Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events.

Design offline extended detention basins to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online extended detention basins for the WQV and integrate any additional storage into the feature. Additional storage to reduce peak runoff will be added above the WQV.

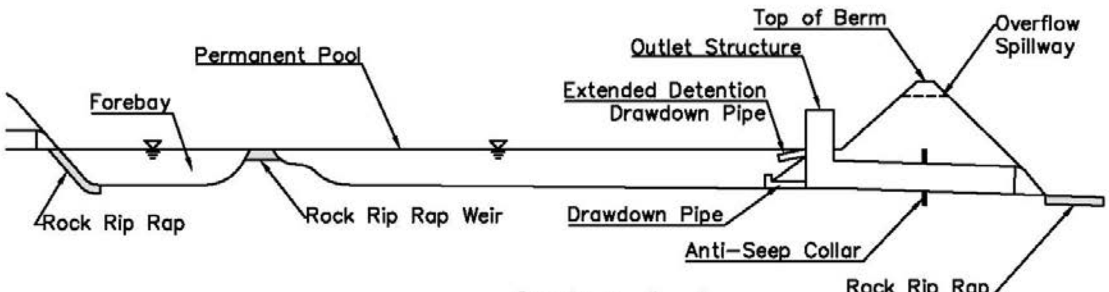
NDOT recommends the use of the NRCS TR-55 procedure (See Chapter 1) to determine the runoff volume and discharge rate used to design a detention system for a project site. The Rational Method and Modified Rational Method will not be acceptable.

Wet Detention

DESIGN EXAMPLES



Wet Detention Pond
No Scale



Section A–A
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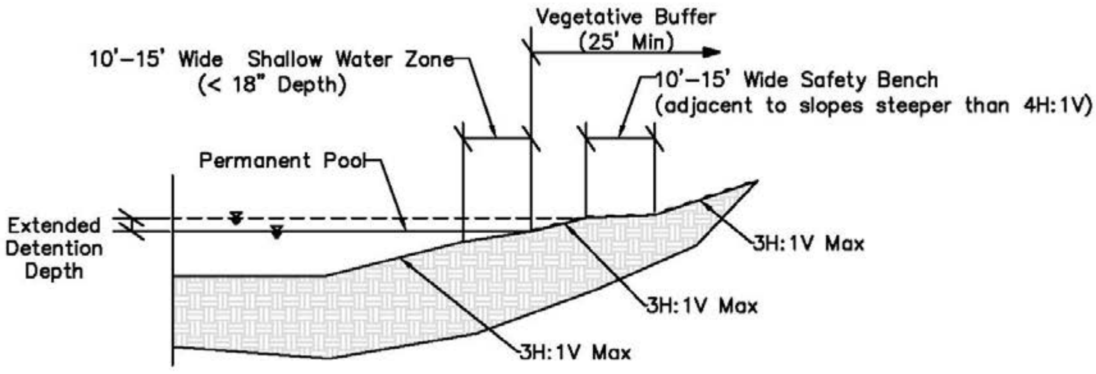

Section B–B
No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because, unless accounted for in the design, inundation with a high sediment load will decrease the intended storage capacity and increase maintenance costs.
- If possible, use the basin to control sediment during construction. However, the sediment will have to be removed before completing the project unless additional sedimentation volume has been incorporated into the design and that design amount has not been exceeded.
- Provide stabilization above the permanent pool elevation once sediment has been removed and any drawdown valves are closed to establish the permanent pool.
- If groundwater is a water source for the wet detention basin, and excavation below the groundwater table is necessary, excavate the bottom in such a manner as to leave the soil in a natural, unsmeared, and uncompacted condition.
- Consider ordering any shoreline wetland plant stock early on to minimize potential delays and time the arrival of that stock to ensure water supply and optimum survival.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for a wet detention basin include providing litter control, monitoring erosion and sedimentation, and maintaining outlet control structures. Diversion structures, discharge points, and forebays should also be inspected and maintained, along with any other pretreatment STFs.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Check wet detention basin 3 days (72 hours) after a major rainfall event to ensure drainage of the basin to permanent pool elevation. • Inspect wet detention basin for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect wet detention basin to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect wet detention basin for erosion and any damage by equipment or vehicles after every major rainfall event. Repair as needed. • Inspect wet detention basin for excessive sediment buildup on the bottom of the basin and at any diversion structures, outlets, and forebays. Remove sediment as needed. • Remove trash and debris from the basin and any diversion structures, outlets, and forebays. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect wet detention basin for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect wet detention basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds around the perimeter of the pool. • Inspect wet detention basin for erosion and damage by equipment or vehicles. Repair as needed.

Wet Detention

Frequency	Inspection and Maintenance Activity
	• Inspect wet detention basin for excessive sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Annually (After NPDES Permit is closed)	• Inspect wet detention basin for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect wet detention basin to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds around the perimeter of the pool. • Inspect wet detention basin for erosion and damage by equipment or vehicles. Repair as needed. • Inspect wet detention basin for excessive sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the basin and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

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Wet Detention

Stormwater Wetland

OVERVIEW

Source: Nebraska Department of Transportation (NDOT)

Definition

Stormwater wetlands are shallow, heavily vegetated basins with varying topography below a permanent pool elevation that creates low and high marshes and pools. They are generally a flow-through type system with temporary storage and can be online or offline.

Benefits

- Good pollutant removal rates.
- Flexible system that can provide water quality benefits, as well as detention in some cases.
- Suitable for shallow groundwater conditions.
- Often provides good habitat and aesthetic value.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction		X	
Maintenance		X	
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients		X	
Heavy Metals		X	X
Hydrocarbons		X	

Limitations

- Larger drainage areas that provide continuous baseflow or a groundwater source are needed.
- Considerable space is needed for this type STF.
- Stormwater wetlands may not be suitable for sites with high pollutant loadings if groundwater is present.
- Minimum setbacks must be met if groundwater is present.

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Stormwater wetlands are typically basins that capture stormwater runoff and pass it through a pool of water of varying depths that support wetland and aquatic vegetation. Stormwater wetlands require continuous baseflow or groundwater to maintain a permanent pool of water and vegetation.

The minimum design volume of water in the permanent pool is based on a fraction of the Water Quality Volume (WQV). Temporary storage for extended detention is provided above the permanent pool elevation. The stored volume is released over a minimum of 24 hours (72 hours maximum). The extended drawdown time is provided to improve pollutant removal efficiencies.

Stormwater wetlands are often constructed online with a multi-stage outlet structure to attenuate peak flow from major storm events in addition to passing the WQV. They can also be designed as an offline system. Pretreatment is desirable to help control sediment from being deposited in the stormwater wetland.

STF COMPONENTS

Pretreatment STF – Pretreatment can be achieved by using vegetated filter strips, grass swales, forebays, etc. Forebays are recommended for end-of-pipe treatment. Many factors dictate the types of pretreatment STFs suitable for the site, including available space, an offline or online system, soil characteristics, site topography, and cost. See design guidelines for additional information on the various STFs that provide pretreatment.

Soils **–** The types of soils on site will partially determine whether the site is suitable to maintain a permanent pool. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics for design.

Berm (Embankment) – A berm is a compacted earthen ridge designed to capture and detain stormwater flows in the stormwater wetland.

Permanent Pool **–** The permanent pool is the body of water stored during “normal” conditions when the wetland water surface is at the lowest control elevation on an outlet structure (excluding any drawdown pipe intended for maintenance).

Low Marsh Zone **–** A low marsh zone is a zone in the permanent pool that supports vegetation in depths of water ranging from 6 to 18 inches as measured from the permanent pool elevation.

High Marsh Zone – A high marsh zone is a zone in the permanent pool that supports vegetation in depths of water less than 6 inches as measured from the permanent pool elevation.

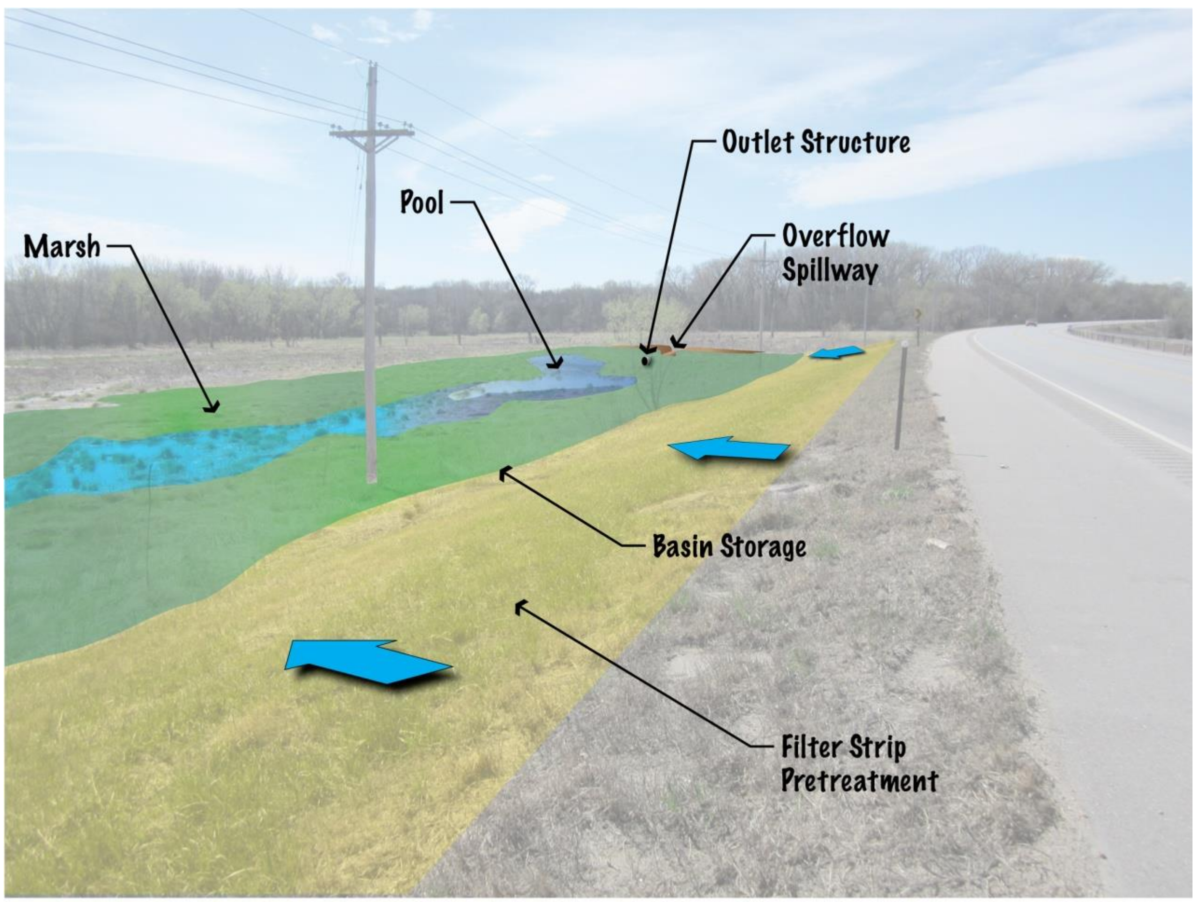
Micropool **–** A micropool is a deeper zone in the permanent pool with depths of water ranging from 1.5 feet to 6 feet. Micropools are needed at the outlet structure to provide additional storage for re-suspended sediment, help prevent the outlet from clogging, and help mitigate thermal effects. They can also be placed in other areas to improve aquatic function.

Stormwater Wetland

Extended Detention Depth – The extended detention depth is the depth of water above the permanent pool elevation that is temporarily stored and released over a specific period of time.

Outlet Structure (Principal Spillway) – An outlet structure is a standpipe or structure designed to draw down stormwater that is stored in the basin. In a stormwater wetland, a multi-stage structure is typically needed. A pipe or an orifice opening is typically provided at the bottom of the basin to draw down the WQV. Other controls (orifice or weir) are provided to draw down larger storm events at predetermined maximum rates.

Overflow Spillway (Auxiliary or Emergency Spillway) – An overflow spillway is a protected area along a berm or an excavated channel designed to convey overflow storm events instead of allowing overtopping of the berm. Consideration should be given to the velocity of flow over the spillway, at any intermediate grade changes, and at the toe. Protection or energy dissipation should be provided where velocities and turbulence are a concern.



Stormwater Wetland Example
Not to Scale

Stormwater Wetland

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Is there a source of water to maintain a permanent pool?

Q. What about impacts on streams and dam safety requirements?

Q. What impact will infiltration have on adjacent pavement, buildings, water bodies, groundwater, etc.?

Q. What are some other safety features to consider?

Site characteristics are very important when designing stormwater wetlands and should be taken into consideration early in the design process.

Check available right-of-way when determining the footprint. Consider the ramifications of standing water adjacent to the roadway and any safety considerations such as locating the stormwater wetland outside clear recovery zones and whether fencing is needed.

Site topography dictates whether an embankment can be constructed to create the storage need or whether excavation is necessary.

Stormwater wetlands need a drainage area of sufficient size to maintain a permanent pool. The minimum suggested ratio of drainage area to pond volume (acres to acre-feet) is 15:1. However, the ability to maintain the pool is going to vary from site to site and may be closer to 60:1 in western Nebraska (see design criteria). Calculations should be done to check the water balance. Groundwater is a source that should be considered in these calculations.

The Nebraska Department of Natural Resources (NDNR) has jurisdiction over “dams” defined in Chapter 46, Article 16: Safety of Dams and Reservoirs and Title 458, Nebraska Administrative Code, Chapters 1-13; NDNR Rules for the Safety of Dams and Reservoirs. Verify whether or not your project falls within NDNR jurisdiction and may need to meet NDNR dam design standards, particularly for embankments 6 feet high or greater and those with an impounding capacity at maximum storage elevation greater than 15 acre-feet.

Stormwater wetlands are susceptible to leaching pollutants into sensitive waters or saturating soils adjacent to infrastructure. The designer should reference design criteria for setback distances.

A 25-foot wide vegetative buffer (minimum) is required to help provide additional water quality benefits, shoreline protection, and habitat. A 10-foot to 15-foot-wide zone of shallow water (0 to 18 inches in depth – low and high marsh zones) along the shoreline is required in areas where the permanent pool depth is 3 feet or greater.

A 10-foot to 15-foot-wide safety bench above the shoreline is required if sideslopes are steeper than 1V:4H. Any mowing should stop on the safety bench.

The following table provides pretreatment criteria that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected pretreatment STF for additional information on function and design considerations.

Stormwater Wetland

DESIGN CRITERIA

Description	Value
Minimum Contributing Basin Area	10 acres (verify water budget to ensure the design elevation for the permanent pool is maintained)
Watershed Area to Permanent Pool Volume Ratio	See Figure 1 for preliminary design (verify water budget to ensure the design elevation for the permanent pool is maintained)
Typical Permanent Pool Length to Width Ratio	2:1 or greater
Maximum Permanent Pool Depth	6 feet
Maximum Extended Detention Depth	3 feet (above permanent pool)
Volume of Forebay (V_{FB}) and Micropool (V_{MP})*	0.2 to 0.4 * WQV (where V_{FB} = 0.1 * WQV) (accounts for volume lost to sedimentation)
Marsh Volume (V_M)*	0.6 to 0.8 * WQV (50% high marsh & 50% low marsh)
Extended Detention Volume (V_{EX})	0.5 * WQV
Minimum V_{EX} Drawdown Time	24 hours (72 hours maximum)
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Pretreatment Criteria	Grass Swale Length – 10 feet (minimum) Vegetated Filter Strip Length – 10 feet (minimum) Forebays – 10% of WQV (minimum)
Setback Distances	Surface Water – 50 feet Private Drinking Water Wells – 100 feet Public Drinking Water Supply Wells (Non-Community System) – 100 feet Public Drinking Water Supply Wells (Community System) – 500 feet Water Lines (Pressure) – 25 feet Water Lines (Suction) – 100 feet Property Lines – 5 feet Foundations (NDOT)** – 20 feet (assumes no basement) Foundations (Neighbors)** – 30 feet (assumes no basement)

* The permanent pool volume equals the forebay, micropool, and marsh volume combined and should be equal to or greater than the WQV.

** Add 10 feet to setback distance when foundations are lower in elevation than water quality feature or adjacent to a full depth basement.

Stormwater Wetland

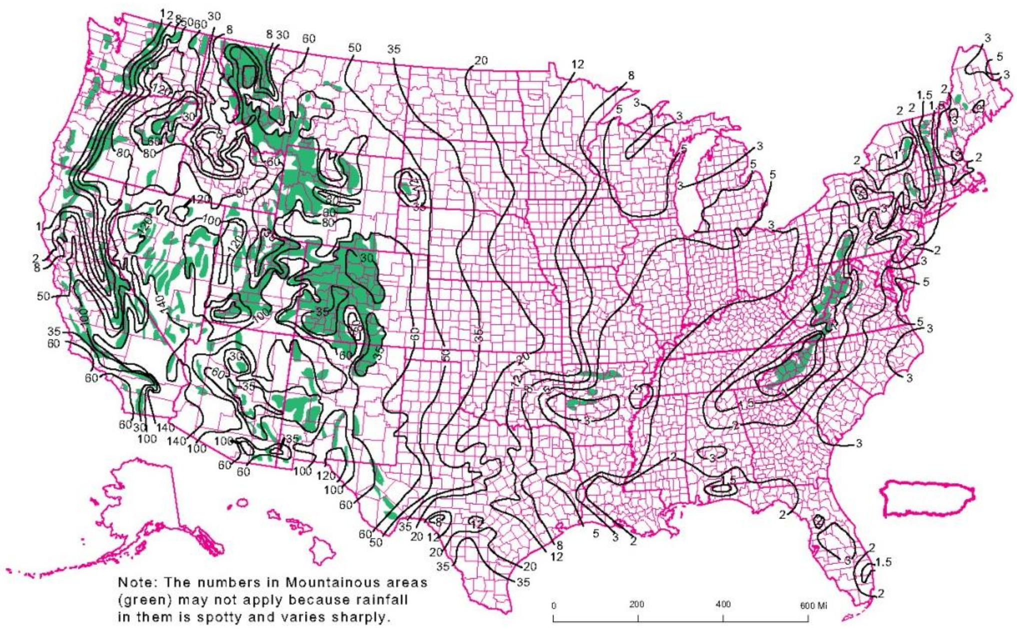


Figure 1 - Guide for Estimating Required Drainage Area (Acres) for Each Acre-Foot of Storage in Basin

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*.

Step 2: Check to see whether the drainage can support the permanent pool in a stormwater wetland.

Using Figure 1, determine the approximate permanent pool volume sustained for a given drainage area for the project area.

$$V_{PMax} = \frac{A_{Total}}{A_{AF}}$$

V_{PMax} = maximum permanent pool volume (ft^3)
A_{Total} = actual drainage area (ac)
A_{AF} = minimum drainage area per ac-ft of storage (ft^{-1}) (From Figure 1)

Check to see whether estimated WQV is less than V_{PMax}

If WQV ≤ V_{PMax}, it's likely that the drainage area can sustain the minimum permanent pool. Verify this by calculating the water budget.

If WQV > V_{PMax}, it's less likely that the drainage area can sustain the minimum permanent pool. Verify this by calculating the water budget or select another type of STF.

Step 3: Allocate WQV to determine volumes and areas of the forebay and micropools, marsh pools, and extended detention

Use the ratios provided in the design criteria table above to determine the volumes of the forebay and micropools, marsh pools (both high and low), and extended detention.

$$V_{FB} = 0.1*WQV$$

$$V_{MP} = (0.3-0.1)*WQV$$

$$V_{MLow} = \tfrac{1}{2}*(1.0-0.3)*WQV$$

V_{MLow} = low marsh volume (ft^3)

$$V_{MHigh} = \tfrac{1}{2}*(1.0-0.3)*WQV$$

V_{MHigh} = high marsh volume (ft^3)

$$V_{EX} = 0.5*WQV$$

Once the volumes have been determined, use the typical depths associated with each zone to lay out the stormwater wetland. Take into consideration the typical permanent pool length to width ratio (2:1) and provide a 10'-15' wide shallow water zone (< 18" depth) around the perimeter of the permanent pool as shown in Design Examples, Section B-B.

Provide a 25' wide vegetative buffer along the edge of the permanent pool. If slopes adjacent to the extended detention pond volume are steeper than 1V:4H, also provide a 10' to 15' wide safety bench (≤ 4%).

Step 4: Size the Orifice for Extended Detention Volume Drawdown

Use the average discharge rate and average hydraulic head to calculate the orifice size for the Minimum V_{EX} Drawdown Time provided in the design criteria table above.

Find the average discharge rate:

$$Q = V_{EX}/t/3600$$

Q = average orifice discharge rate (cfs)
t = V_{EX} drawdown time (hours)

Find the orifice area:

$$A = Q/[C*(2*g*h)^{0.5}]$$

A = orifice area (ft^2)
C = orifice discharge coefficient, dimensionless (0.60 typ)
g = acceleration of gravity (ft/s^2)
h = average hydraulic head (ft)
(height measured from orifice invert to midpoint of extended detention depth – assumes orifice is small relative to total height)

Find the orifice diameter:

$$d = (4*A/3.14)^{0.5}*12$$

d = orifice diameter (in)

Step 5: Check diversion or storage and routing of larger storm events

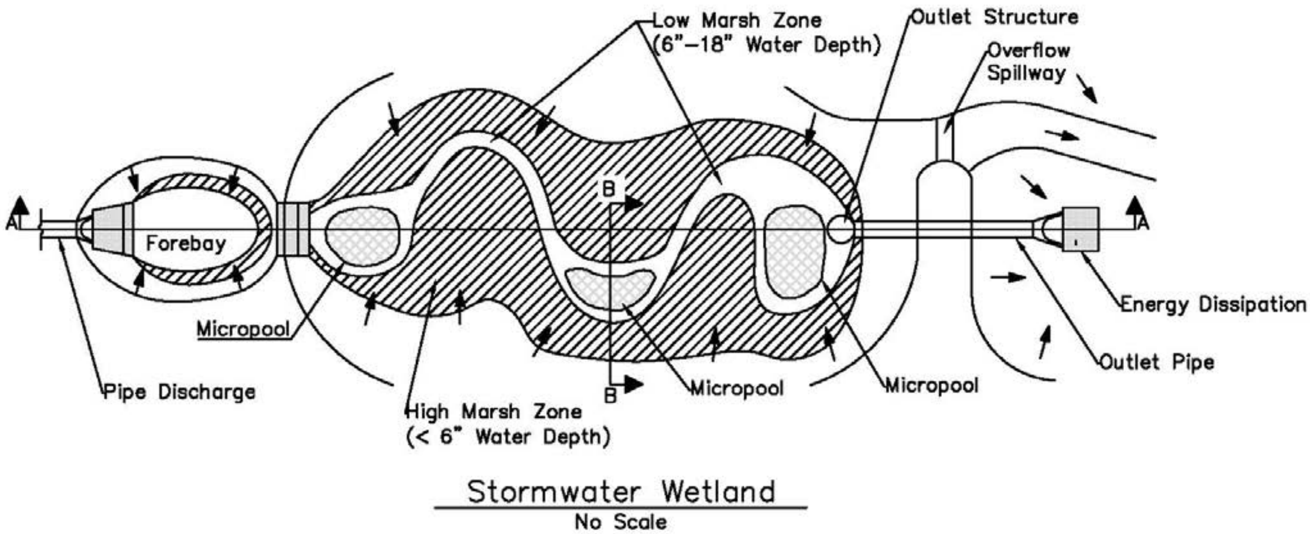
Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events.

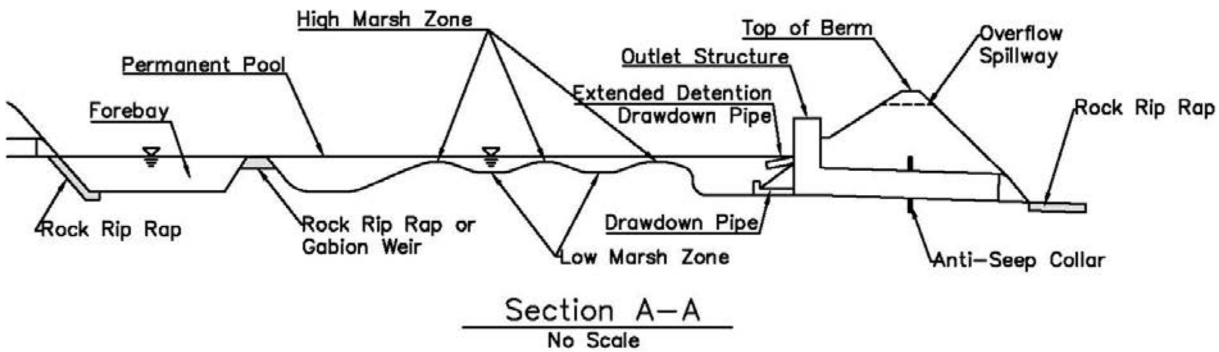
Design offline extended detention basins to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online extended detention basins for the WQV and integrate any additional storage into the feature. Additional storage to reduce peak runoff, if needed, is added above the WQV.

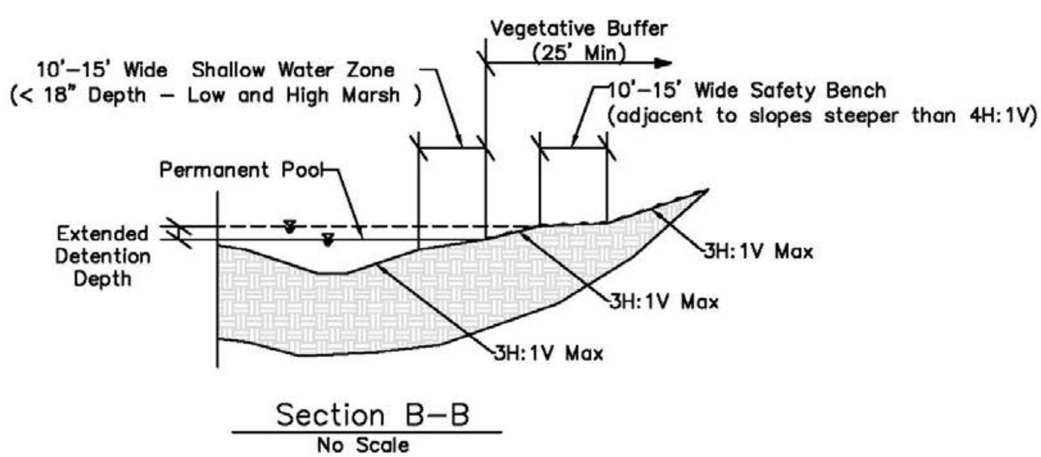
NDOT recommends the use of the NRCS TR-55 procedure (See Chapter 1) to determine the runoff volume and discharge rate used to design a detention system for a project site. The Rational Method and Modified Rational Method will not be acceptable.

Stormwater Wetland

DESIGN EXAMPLES







CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because stormwater wetlands are prone to failure when inundated with a high sediment load.
- Provide inlet protection while the stormwater wetland is being constructed. The best protection may be to bypass stormwater away from the facility until the stormwater wetland is fine graded and areas above the permanent pool elevation have been stabilized.
- Consider the space needed for pretreatment and any swale required for bypassed flow.
- If it is not possible to stabilize upgradient before beginning construction and flow cannot be temporarily bypassed, provide erosion and sediment control protection for the stormwater wetland.
- If the stormwater wetland relies on groundwater for a water source, excavate the bottom in such a manner as to leave the soil in a natural, unsmeared, and uncompacted condition.
- Protect completed stormwater wetland areas from construction or other traffic during the course of construction.
- Consider ordering any wetland plant stock early to minimize potential delays. Time the arrival of that stock to ensure water supply and optimum survival.
- If possible, delay the installation of any plantings until after you've had a chance to close the drawdown pipe, fill the wetland to permanent pool elevation, and observe actual depths of water. Adjust planting types and placement accordingly for optimum survival.
- De-water the stormwater wetland at least 3 days before planting, assuming the water source is surface water fed, to try to dry the site.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for a stormwater wetland include providing litter control, monitoring erosion and sedimentation, and maintaining outlet control structures. Diversion structures, discharge points, and forebays should also be inspected and maintained, along with any other pretreatment STFs.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Check stormwater wetland 3 days (72 hours) after a major rainfall event to ensure drainage of the basin to permanent pool elevation. • Inspect stormwater wetland for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect stormwater wetland to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect stormwater wetland for erosion and any damage by equipment or vehicles after every major rainfall event. Repair as needed. • Inspect stormwater wetland for excessive sediment buildup on the bottom of the basin and at any diversion structures, outlets, and forebays. Remove sediment as needed. • Remove trash and debris biweekly from the basin and any diversion structures, outlets, and forebays. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect stormwater wetland for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate the cause (i.e. inflow, drought, or excessive seepage). • Inspect stormwater wetland to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Inspect stormwater wetland for erosion and damage by equipment or vehicles. Repair as needed. • Inspect stormwater wetland for excessive sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the wetland and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed.

Stormwater Wetland

Frequency	Inspection and Maintenance Activity
Annually (After NPDES Permit is closed)	• Inspect and maintain pretreatment STFs in accordance with their respective design guidelines. • Inspect stormwater wetland for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect stormwater wetland to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Inspect stormwater wetland for erosion and damage by equipment or vehicles. Repair as needed. • Inspect stormwater wetland for excessive sediment buildup on the bottom of the basin. Remove sediment as needed. • Inspect the wetland and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

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Stormwater Wetland

Pervious Pavement

OVERVIEW

Source: City of Omaha

Definition

Pervious pavement systems allow the infiltration of stormwater runoff through a pavement surface into an aggregate base. The aggregate base provides temporary storage of captured rainfall where it then infiltrates into underlying soils or is collected by an underdrain. Pervious surfaces typically include concrete, asphalt, or pavers.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction			X
Maintenance			X
Pollutant Removal	**L**	**M**	**H**
Suspended Solids			X
Nutrients		X	
Heavy Metals		X	X
Hydrocarbons		X	X

Benefits

- Suitable for use on highly urbanized sites and other locations where space is limited.
- Flexible system that can provide water quality benefits, as well as detention.
- Suitable for use in areas of compacted fill or low permeability soils.
- Less likely to form ice than conventional pavement and reduces hydroplaning.

Limitations

- Pervious pavement is not typically suited to traffic with heavy loads and/or high volume.
- Higher costs are associated with this feature.
- Pervious pavement is generally not appropriate where high sediment loads are a concern due to clogging and associated maintenance.
- Pervious pavement is not suitable in areas with a high groundwater table.
- Special consideration should be given to expansive or collapsible soils.

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Pervious pavement systems (also known as permeable or porous pavement systems) allow stormwater runoff, typically sheet flow, to infiltrate and pass-through voids in the surface layer material and into base and subbase aggregates where it is temporarily stored and released slowly. Stormwater then infiltrates into the soil or is collected by an underdrain and discharged to a sewer pipe or daylighted. Surface materials covered herein include pervious concrete, pervious asphalt, and pervious pavers.

Pervious pavement systems are designed to capture the Water Quality Volume (WQV) and infiltrate that volume over a minimum of 24 hours (48 hours maximum). However, they can be modified to provide both water quality control and peak flow control with the appropriate overflow measures in place.

STF COMPONENTS

Pavement Surface – The pavement surface is a pervious wearing course with durability and strength to withstand the appropriate traffic loadings. The various pavement surfaces included in these design guidelines include pervious concrete, pervious asphalt, and pervious pavers.

Pervious concrete has fewer fines than traditional Portland cement concrete and relies on the cementious paste to bond the larger aggregate. Voids left by the absence of fines allow water to pass through.

Pervious asphalt has fewer fines than traditional asphaltic concrete mixes and relies on the asphalt binder to bond the larger aggregate. Voids left by the absence of fines allow water to pass through.

Pervious pavers, also known as Permeable Interlocking Concrete Pavement (PICP), are traditional pavers formed and laid with gaps that are filled with an open-graded aggregate that provides void structure that allows water to pass through the layer. Paver thickness may vary between manufacturers but is typically 3 1/8 inches thick for vehicular traffic use. Pervious pavers are set on a leveling course or bedding of material similar to that used for filling the gap between pavers. The material is often AASHTO No. 8, 89, or 9 stone.

Aggregate Base Course – The base course layer of aggregate provides structural support and a suitable surface for laying pervious concrete and pervious asphalt on. It should be a clean washed, uniformly graded, free-draining aggregate and can be used for the full thickness of the reservoir. The base course should also be of a gradation that will not fill voids in the underlying subbase course if used. The recommended base course is typically an AASHTO No. 57 stone.

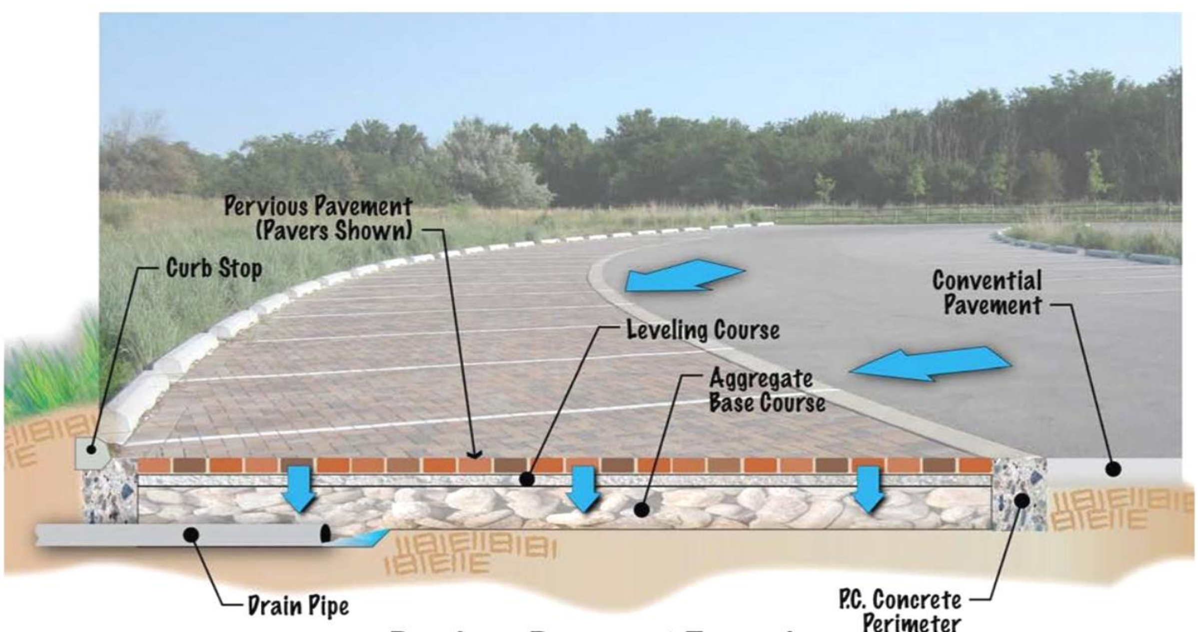
Separation Fabric, Filter Course, and Geogrid – Non-woven geotextile fabric or an appropriately sized filter stone course should be used to keep fines from entering the pervious pavement system. They also help keep the base layer aggregate from being pushed into the subgrade when the soil becomes saturated and the pavement is loaded. Geogrid is also recommended on low California Bearing Ratio (CBR) soils, such as saturated clays and silt, to help distribute loads.

Pervious Pavement

Soils **–** The types of soils on site will partially determine how much water will be infiltrated. Additionally, the wetting and saturation of expansive or collapsible soils will require special considerations. Refer to the project geotechnical report and the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey for the project area to help determine project soil characteristics for design.

Depth – The depths of the individual surface and aggregate layers vary depending on the selected material for the surface course and loads. However, the minimum total depth is a function of frost depth. It also depends on the volume of storage needed, soil properties, and available space. Shallow groundwater or bedrock can also limit it.

Underdrain – An underdrain positioned at the bottom of the base course allows for water to drain out of the basin. The underdrain pipe is typically a section of slot perforated PVC pipe lying horizontally at the bottom of the chamber surrounded by a 6-inch layer of 1 ½-inch drainage aggregate. The slot perforated PVC pipe should connect to an adjustable valve, any riser pipe needed to access the valve, and solid PVC pipe that discharges to a sewer pipe or is daylighted. A cleanout should be provided on the upgradient end of the perforated pipe section. Underdrain pipes should be 6-inch diameter minimum.



Pervious Pavement Example
Not to Scale

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Are site and traffic conditions suitable for pervious pavement?

Q. What types of soils are on site and are they compacted?

Q. What impact will infiltration have on adjacent pavement, buildings, water bodies, groundwater, etc.?

Q. How does the proposed pervious pavement system interact with other design storm considerations?

Because of the higher cost of the pervious pavement system, placement is generally limited to sites with physical constraints where space is an issue and/or where the cost of land is an issue.

Pervious pavement systems are better suited to sites with lower traffic volumes and speeds. Loads may also be a factor, and systems may need to be modified from the recommended minimum design criteria to account for higher loadings and traffic volumes. Pervious pavement systems are generally limited to parking stalls, drives, trails, and roadway shoulders. Pavers should be laid in a herringbone pattern to help distribute the loads more effectively.

A perimeter barrier (typically made of Portland cement concrete – 6 inches wide minimum with a depth that extends to the bottom of the base aggregate course) is required for pervious pavers and recommended for pervious asphalt. When pervious pavement abuts traditional pavement either a concrete perimeter barrier or an impermeable geomembrane liner shall be used to reduce the risk of saturating adjacent subgrade soils.

Site topography should be considered in design and is best suited where the topography of the site allows for placement on the contour or flat area. Pervious pavement systems require a level bottom when infiltration is suitable. On slopes greater than 1 percent, transverse barriers should be used to provide the necessary storage in aggregate reservoirs in stepped sections. Transverse barriers may be constructed using concrete or a 30 mil PVC geomembrane. The top of the transverse barrier should be positioned several inches below the pervious pavement section.

The underdrain system should be designed for a 24-hour drawdown time and include a valve to help control discharge rates. Cleanouts should also be provided.

When expansive soils are encountered, an impermeable liner will likely be necessary to reduce risks to the pavement surface course. When collapsible soils are encountered, an impermeable liner is an option, as well as the excavation and re-compaction of upper subgrade soils. Consult the geotechnical engineer for further recommendations and design guidance if these materials are encountered.

Pervious pavement systems designed for infiltration are susceptible to leaching pollutants into sensitive waters or saturating soils adjacent to infrastructure. The designer should reference design criteria for setback distances.

Pervious Pavement

The bottom of the pervious pavement system should be level when infiltration is desired and at least 4 feet above the seasonal high groundwater table. If a water table is not present, then the bottom of the aggregate course should be at least 4 feet above bedrock or other barrier layer.

Pervious pavement systems may be modified to include additional storage for peak flow reduction with an appropriate outlet structure that allows the release of the Water Quality Volume (WQV) at prescribed rates and overflow for larger storm events.

Pervious pavement systems should be designed to safely convey larger storm events whether the system is designed offline or online or becomes plugged. The collection and/or conveyance of larger storm events will need to be incorporated into the overall design.

Pervious Pavement

DESIGN CRITERIA

Description	Value
Maximum Contributing Basin Area	5 acres
Contributing Basin Area to Pervious Pavement Surface Area Ratio	3:1 to 5:1 (typical)
Surface Depth (typical for conditions provided in text – use engineering judgment)	Pervious Concrete – 6 inches Pervious Asphalt – 5 inches Pervious Pavers – 3 1/8 inches (+ 1 inch bedding minimum for pavers)
Base Course Depth	12 inches minimum (even with filter course)
Filter Course Depth (Optional)	6 inches typical
Minimum Total Depth	24 inches (2/3 minimum frost depth typical)
Minimum Infiltration Rate	0.5 inch/hour
Maximum Infiltration Rate	12 inches/hour
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Aggregate Porosity	0.30 Base Course Aggregate 0.25 Filter Course Aggregate (Geotechnical engineer will need to approve anything greater)
Time for Infiltration	24 hours (48 hours maximum)
Setback Distances	Surface Water – 50 feet Private Drinking Water Wells – 100 feet Public Drinking Water Supply Wells (Non-Community System) – 100 feet Public Drinking Water Supply Wells (Community System) – 500 feet Water Lines (Pressure) – 25 feet Water Lines (Suction) – 100 feet Property Lines – 5 feet Foundations (NDOT)* – 20 feet (assumes no basement) Foundations (Neighbors)* – 30 feet (assumes no basement)

* Add 10 feet to setback distance when foundations are lower in elevation than water quality feature or adjacent to a full depth basement.

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* **OR** use Exhibit 3.5 - Water Quality Volumes and Peak Discharges for Selected Acreages for sites less than 5 acres.

Step 2: Determine the minimum depth of the pervious pavement system

To determine the minimum total depth, add the minimum reservoir depth (base aggregate and, if used, the filter course) to the pervious pavement thickness (and bedding if pavers are used) and check the total against minimum design criteria based on frost depth. Make sure all minimum criteria are met.

To determine the minimum volume of the reservoir, divide the WQV by the available pore space:

$$V_R = WQV/n$$

V_R = pervious pavement reservoir volume (ft^3)
n = aggregate porosity
(assume 0.30 initially or obtain from project geotechnical report – actual depth should be based on composite "n")

The minimum bottom surface area depends on the infiltration rate of the soil and drain time allowed.

$$A = \frac{WQV * (12\ in/ft)}{I * t}$$

A = bottom surface area (ft^2)
I = infiltration rate of underlying soil (in/hr)
(obtain from field or laboratory testing)
t = time to drain (24 hours)

The minimum reservoir depth is determined by dividing the reservoir volume by the bottom surface area.

$$D_R = V_R/A$$

D_R = minimum depth of reservoir
(calculated depth or frost depth, whichever is greater)

Add the type of pervious pavement surface and any bedding layer to the minimum reservoir depth to get the total minimum depth. Check minimum total depth calculated against minimum depth criteria based on frost depth provided in the design criteria table (use the greater depth).

Step 3: For underdrain systems, size the valve for WQV drawdown

Use the average discharge rate and average hydraulic head to calculate the orifice size for the Minimum WQV Drawdown Time provided in the design criteria table. Round the calculated orifice size up to the nearest standard valve size and include it in the design.

Find the average discharge rate:

$$Q = WQV/t/3600$$

Q = average orifice discharge rate (cfs)
t = WQV drawdown time (hours)

Find the orifice area:

$$A = Q/[C*(2*g*h)^{0.5}]$$

A = orifice area (ft^2)
C = orifice discharge coefficient, dimensionless (0.60 typ)
g = acceleration of gravity (ft/s^2)
h = average hydraulic head (ft)
(height measured from orifice invert to midpoint of extended detention depth – assumes orifice is small relative to total height)

Find the orifice diameter:

$$d = (4*A/3.14)^{0.5}*12$$

d = orifice diameter (in)

Round the calculated orifice size up to the nearest standard valve size.

Step 4: Check diversion or storage and routing of larger storm events

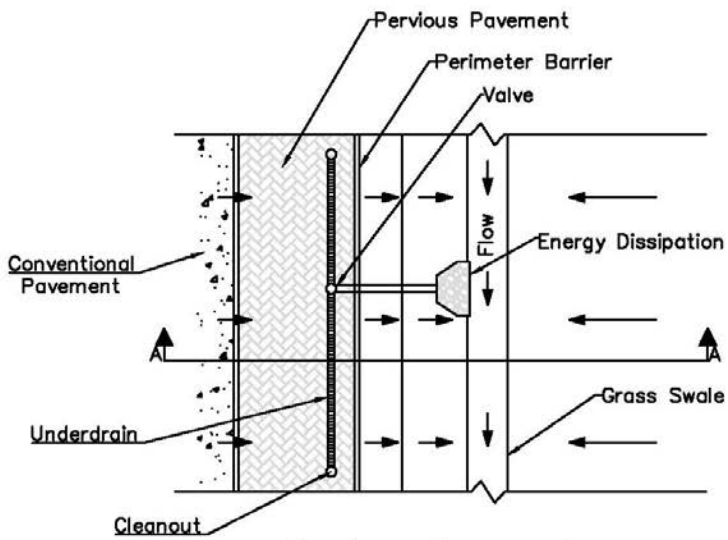
Reference Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual* to check the routing of larger design storm events.

Design offline pervious pavement systems to divert runoff greater than the Water Quality Volume Discharge Rate (Q_{WQ}) away from the system. Design online pervious pavements systems for the WQV and integrate any additional storage into the feature. Additional storage to reduce peak runoff will be added above the WQV and controlled through a multistage outlet structure.

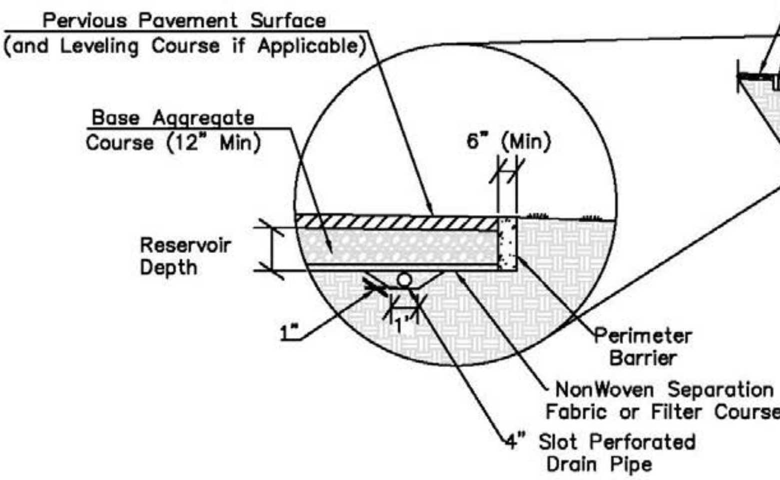
NDOT recommends the use of the NRCS TR-55 procedure (See Chapter 1) to determine the runoff volume and discharge rate typically used to design a detention system for a project site. The (Modified) Rational Method will not be acceptable.

Pervious Pavement

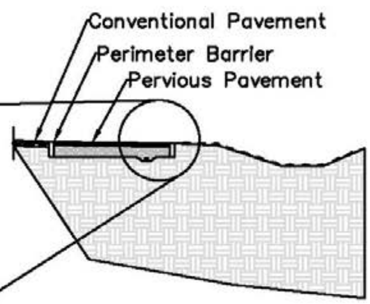
DESIGN EXAMPLES



Pervious Pavement
Plan View
No Scale



Drain Pipe and
Perimeter Barrier
No Scale



Section A–A
No Scale

CONSTRUCTION AND CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction because pervious pavement systems are prone to failure when inundated with a high sediment load.
- Stabilize the upgradient contributing drainage area before putting the pervious pavement systems into operation.
- If it is not possible to stabilize upgradient before construction and flow cannot be temporarily bypassed, provide erosion and sediment control protection for the pervious pavement system or cover with a tarp or other impermeable cover.
- As with conventional pavement systems, uniformity of subgrade support is important for pervious pavement systems. Proof-roll or check the pavement subgrade in some other manner for uniformity. If subgrade soils are not suitable for infiltration, then compact and rework subgrade soils if necessary.
- If subgrade soils are suitable for infiltration, then protect subgrade soils from compaction. Avoid using heavy equipment in the basin bottom during excavation of the pervious pavement system to maintain the infiltration rate.
- Excavate the bottom and sides in such a manner as to leave the soil in a natural, unsmeared, and uncompacted condition.
- If infiltration areas do get compacted during construction, additional infiltration testing may be required.
- Compact aggregate courses and, ideally flush them with water, to seat the aggregates before placing the pavement surface.
- Protect pervious pavement system from construction or other traffic during the course of construction and after construction where practical. If this is not possible, take steps to reduce loads, compaction, tracking of mud and debris, and deposition of sediments.
- Follow industry standards for the appropriate mixing, transport, placement and consolidation, jointing, finishing, and curing of the various pavement surfaces. Follow weather restrictions for the construction of the pervious pavement system.
- Appropriately train an adequate number of construction team members for specified pavement surface construction. Typically, this number would be at least one out of four crew members.
- Conduct a pre-paving conference with the construction team, design team, and NDOT representatives before beginning construction.
- Test panels or sections are recommended 30 days before construction for the various pervious pavement systems.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for pervious pavement systems include maintaining infiltration rates and storage capacity of the surface pavement, underlying aggregates, and subsoils. Outlet structures should also be inspected and maintained, along with vegetation in the contributing basin.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit	• Inspect pervious pavement system for any surface ponding or indicators that water has ponded for an extended period of time. • Check observation wells 3 days (72 hours) after a major rainfall event to ensure proper drain time. • Inspect pervious pavement system for sediment buildup on the pavement surface and any outlets. Remove sediment as needed. • Remove trash and debris from the trench and any diversion structures, outlets, and forebays. • Inspect and maintain vegetated areas.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Inspect pervious pavement system for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect pervious pavement system for sediment buildup on the trench surface. Remove sediment as needed. • Inspect the trench and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Annually (After NPDES Permit is closed)	• Inspect pervious pavement system for any surface ponding or indicators that water has ponded for an extended period of time. • Inspect pervious pavement system for sediment buildup on the trench surface. Remove sediment as needed. • Inspect the pervious pavement system and any diversion structures, outlets, and forebays for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

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Proprietary Structural Treatment Controls

OVERVIEW

Source: CDS® Hydrodynamic Separator from Contech Website (Not an Endorsement)

Definition

Proprietary Structural Treatment Controls are structural stormwater treatment systems manufactured by private companies. They are often prefabricated units designed and sized based on criteria determined by the manufacturer for a given treatment volume or flow rate. The types of pollutants and removal efficiencies vary with each system.

Benefits

- Suitable for highly urbanized areas and areas with direct runoff from impervious surface such as parking lots or roadways.
- Can fit in limited space and be located underground and under pavement.
- Often suitable in areas where infiltration practices and groundwater contamination are a concern.

Overview Table

Associated Costs	**L**	**M**	**H**
Design		X	
Construction			X
Maintenance		X	X
Pollutant Removal	**L**	**M**	**H**
Suspended Solids	X	X	X
Nutrients	X	X	X
Heavy Metals		X	
Hydrocarbons		X	

Limitations

- Typically designed for treatment of smaller drainage areas.
- Can be cost prohibitive.
- Requires reliance on manufacturer's claims for treatment efficiency if not third-party verified and field tested.
- May present possible odor problems.
- Consider floatation of the structure in areas with high groundwater.
- May have special maintenance needs.

Proprietary Structural Treatment Controls

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Proprietary structural treatment controls (PSTC) are commercially available stormwater treatment systems. They come in many sizes to treat a range of stormwater flow rates or volumes provided by the designer. These systems treat stormwater in various ways with mechanisms ranging from gravity and vortex separation of solids to filtration with catch basin inserts. The structures may be fitted online or offline.

An overview of PSTCs is provided below. Products shown following the descriptions below are provided as examples. They are not an endorsement of the product, nor are they necessarily on the Nebraska Department of Transportation (NDOT) Approved Products List. The designer should verify whether a product is on the NDOT Approved Products List before specifying its use.

STF CATEGORIES

Hydrodynamic Separators (Figure 1) – Hydrodynamic separators are flow-through structures that rely on vortex flow to assist in solids separation and gravity for deposition into a sedimentation chamber. Solids removed are often courser material due to limited retention times and indirect filtering methods. Designs often include the ability to separate oil and grease, floatables, and other debris with baffles and screens. Structures are placed underground.

Wet Vaults (Figure 2) – Wet vaults are underground structures with multiple chambers that allow sedimentation to occur by gravity. The chambers are designed to hold a “permanent” pool of water but may also be designed to retain the water quality storm for an extended period to allow for sedimentation. The structures are often fitted with weirs and baffles to reduce energy and sediment re-suspension and to trap oil, grease, and floatable debris.

Catch Basin Inserts (Figure 3) – Catch basin inserts are filters designed to fit inside a storm drain inlet. Catch basin inserts often take the form of a basket, a system of trays, or filter socks. Baskets may simply be used to collect gross solids or can be lined with an absorbent or a filter, Tray type systems are inserts with multiple layers of various absorbent or filtration media to remove the desired pollutant, and socks are generally a fabric filter in the form of a sock that is inserted in an inlet to trap pollutants. Some inserts allow for overflow bypass while others may not. Some inserts may be vacuumed out or removed and cleaned out while others may need complete replacement of the insert or treatment media.

Media Filtration Systems (Figure 4) – Media filtration systems are manufactured units that often contain a pretreatment settling chamber and a chamber with filtration or absorbent media, typically in the form of a cartridge. They can be designed with storage to retain the water quality storm for an extended period to assist with sedimentation during pretreatment. The filtration or absorbent media cartridge is replaced as needed. Overflow bypass is typically provided.

Landscape Filtration Systems (see Figure 5) – Landscape filtration systems are manufactured units that allow stormwater to filter through landscape vegetation or turf and infiltrate through a growing media, typically a sand and compost mix. Pretreatment may be built into the system to reduce gross solids and coarse sediment. Overflow bypass is typically provided in these systems.

Proprietary Structural Treatment Controls

Figure 1. Hydrodynamic Separator
Source: Vortechs® System

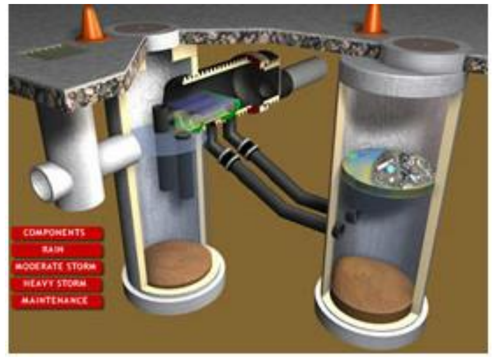

Figure 2. Wet Vaults
Source: Bay Separator® System

Figure 3. Catch Basin Inserts
Source: Triton® Catch Basin Inserts

Figure 4. Media Filtration Systems
Source: StormFilter® System

Figure 5. Landscape Filtration Systems
Source: Filterra® System

Proprietary Structural Treatment Controls

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. What are the site constraints?

Q. What types of pollutants are you trying to remove?

Q. How effective is the structure at removing pollutants?

Q. How does the proposed treatment control interact with larger storm events?

Q. Is the proprietary structural treatment control designed for ease of maintenance?

PSTCs typically require minimal space and are well suited for dense urban environments. Each manufactured system is different. The designer should verify space requirements for the system, including any additional space needed for pretreatment or bypass.

The designer should consider the types of pollutants PSTCs will remove and match the type of structure with the intended purpose as much as possible.

There may be limitations to the level of treatment because of the removal mechanisms involved, the product's size, limitations to the time for removal or sedimentation, etc. The designer should be aware of those limitations.

The treatment system must have demonstrated capability of meeting stormwater goals for its intended use. The manufacturer needs to provide independent third-party scientific verification of the treatment system's performance. The manufacturer should also demonstrate effectiveness of the treatment system with testing in the field, specifically testing the structure in climates similar to Nebraska's and for uses similar to the ones you expect on your project.

Hydraulic grade lines should be checked to make sure there is enough head to allow for the treatment system to function properly and not cause backup. Consideration should be given to how the treatment system functions during a larger storm event. The treatment system should allow for bypass of larger storm events with minimal resuspension or remobilization of pollutants.

The designer needs to review the maintenance of structural treatment controls, and maintenance personnel need to be aware of what may be involved. Maintenance may include vacuuming out a sedimentation reservoir or filter basket, or replacing a filter media or filter media cartridge. Manufacturer's recommendations on maintenance should be followed. More frequent inspections occur the first year of operation (perhaps quarterly) to gauge the amount of pollutants trapped. Inspection frequency and maintenance could be adjusted after the first year to more suitably meet site conditions.

Proprietary Structural Treatment Controls

MAINTENANCE AND INSPECTION REQUIREMENTS

Please refer to the manufacturer's specifications for inspection and maintenance requirements.

RESOURCES AND REFERENCES

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California Department of Transportation. *Storm Water Quality Handbook – Project Planning and Design Guide*. July 2010.

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Proprietary Structural Treatment Controls

Forebays

OVERVIEW

Source: Lincoln, NE 56th and Pine Lake

Definition

A sediment forebay is a settling basin that provides pretreatment. It is designed to allow sediment to drop out of stormwater runoff before entering another basin: extended dry detention basins, wet detention basin, and stormwater wetlands.

Benefits

- Helps dissipate energy and reduce velocity from outlet discharge.
- Reduces accumulation of sediment in detention basins and improves water quality.
- Limits the area of heavy sedimentation so it can be maintained more effectively.

Overview Table

Associated Costs	L	M	H
Design	X		
Construction	X		
Maintenance		X	
Pollutant Removal	**L**	**M**	**H**
Suspended Solids		X	
Nutrients	X		
Heavy Metals		X	
Hydrocarbons	X		

Limitations

- Site topography may dictate size and shape.
- Design requirements may increase with larger basins and/or higher embankments.
- Mosquito and algae problems may develop in wet forebays.
- Typically removes coarser sediment particles.

Forebays

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

Forebays are traditionally used to help dissipate energy and allow sedimentation to occur before stormwater enters the retention basin. Forebays accept runoff from pipes, swales, or sheet flow. They can take any shape but are typically designed to fit the topography and should have a minimum length to width ratio of 2:1. Forebays may be lined with turf reinforcement mats (TRMs), transition mats, riprap, concrete, or a combination thereof to help spread flows, dissipate energy and reduce the potential for end-of-pipe erosion. They should be designed online to decrease sedimentation in the larger downstream basin.

The volume of the forebay will vary based on the Water Quality Volume (WQV) and anticipated depth of the forebay. The forebay will need to have the sediment removed when 50% of the capacity is lost. The forebay should be examined annually (through the use of a vertical sediment depth marker) after the spring melt to see if sediment removal is needed and will vary depending on the composition of the contributing drainage area.

STF COMPONENTS

Stone Weeper Weir **-** A forebay outlet structure comprised of a stone weir that allows water to pass over it during large storm events and seep through it to drain the basin down after the storm has passed and during small storm events. Stone weeper weirs are utilized to maintain a dry forebay with the extended dry detention basin design.

> ***Effective Weir Length (L')*** - The length of the weir (ft), perpendicular to flow, that water can pass over.
>
> ***Riprap Length (L)*** - The total length of riprap (ft), measured perpendicular to the direction of flow, that includes the weir length and extends past it to tie into the earthen embankment.
>
> ***Weir Height (H)*** - The height of the weir (ft) will be designed to pass 10-20% of WQV (depending on forebay depth selected).
>
> ***Weir Breadth (W)*** - The width of the weir (ft), measured parallel to flow, that water passes over.

Riprap Seepage Cover and Embankment - Forebays for wet detention and stormwater wetlands should have an earthen embankment with 2 ft of rock riprap cover constructed just below the permanent pool elevation to form the forebay back side. The riprap seepage cover helps prevent erosion and allows flow through when the water surface elevation falls below the permanent pool elevation.

Energy Dissipation - Energy dissipation is required if the discharge velocity exceeds 15 fps, and should be evaluated on a case-by-case basis for any discharge greater than 8 fps. Riprap aprons are intended to reduce velocities and dissipate energy at pipe outlets. Minimum forebay dimensions are based on minimum requirements for sizing riprap aprons whether they are needed or not.

Total Depth **-** The total depth of the forebay is measured from the top of the embankment to the bottom of the forebay basin.

Forebays

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. How much area is available and will 10%, 15%, or 20% of WQV be treated?

Q. Does the discharge velocity exceed 8 fps?

Q. How deep is the forebay, and will it need an aquatic bench or an embankment higher than 6 ft?

Site characteristics are important when designing a forebay and should be taken into consideration early in the design process.

Check available right-of-way when determining the footprint. Consider the ramifications of standing water adjacent to the roadway and any safety considerations, such as locating the forebay outside clear recovery zones and whether fencing is needed.

The volume of the forebay varies based on the WQV. The forebay should be sized to contain a minimum of 10% of the WQV with a standard minimum depth of 2 ft. However, if a larger area is available for the forebay and a smaller minimum depth is selected (1.5 or 1 ft) then 15% or 20% of the respective WQV will be the design volume for the forebay. This forebay storage volume is part of the total WQV and can be subtracted from the WQV for permanent pool sizing.

Regardless of whether energy dissipation is needed or not, the minimum length and width of the forebay will be sized using equations for riprap apron sizing. While the minimum depth will vary based on volume of water delivered and the WQV percentage treated, forebays should not exceed 4 feet in depth or will require an aquatic bench if maximum depth is exceeded. Forebays should be constructed with side slopes no steeper than 1V:3H.

The Nebraska Department of Natural Resources (NDNR) has jurisdiction over “dams” as defined in Chapter 46, Article 16: Safety of Dams and Reservoirs and Title 458, Nebraska Administrative Code, Chapters 1-13; NDNR Rules for the Safety of Dams and Reservoirs. Verify whether or not your project falls within NDNR jurisdiction and may need to meet NDNR dam design standards; particularly for embankments 6 feet high or greater and those with an impounding capacity at maximum storage elevation greater than 15 acre-feet.

Site topography dictates whether an embankment can be constructed to meet the storage need or whether excavation is necessary. Stone Weeper weirs should be utilized for an outlet for extended dry detention forebays. The slopes of stone weeper weirs should not be steeper than 1V:3H and should be underlain with geotextile fabric. Additionally, the stone weeper should have a 10 ft minimum breadth of weir (4’ top of berm) and be constructed with a 2 ft layer of 1” washed stone over a layer of rock riprap (Type A typical). The weir section should be able to withstand shear stresses from 10-year return period design storms. Riprap should be provided on the downstream side of the stone weeper weir to provide erosion control as well and should be constructed as a channel down the backslope.

Wet forebays will have a riprap seepage cover (min 2 ft deep) on an earthen embankment. The minimum top of embankment width is 4 ft.

The design criteria table provides standards that should be followed to the extent practical. The designer should refer to the design guideline specific to the selected STF for additional information on function and design considerations.

Forebays

DESIGN CRITERIA

Description	Value
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*
Typical Forebay Length: Width Ratio	2:1 or as dictated by riprap apron area and topography
Minimum Forebay Length	Based on riprap apron length
Minimum Forebay Width	Based on riprap apron length
Maximum Forebay Depth	4 ft
Minimum Forebay Depth	10% WQV treated= 2 ft 15% WQV treated= 1.5 ft 20% WQV treated= 1 ft
Forebay Volume	10%-20% of WQV
Effective Weir Length (L')	Varies based on Contributing Drainage Area (see Table 2)
Riprap Length (L)	Varies based on L' (see Table 2)
Setback Distances	Same design considerations as STF implemented

Forebays

DESIGN PROCEDURE

Step 1: Calculate flow to STF

Calculate the flow to the forebay for a 10-yr return period storm using the Rational Method (NDOT Drainage and *Erosion Control Manual* - Chapter 1: Section 6).

Step 2: Determine riprap size (FHWA HEC 14)

Even though a riprap apron is not required for velocities less than 8 fps, the size of the theoretical apron will still dictate the minimum size of the forebay.

$$D_{50} = 0.2D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{\frac{4}{3}}\left(\frac{D}{TW}\right)$$

D_{50}= riprap size (ft)
Q= design discharge (ft^3/s)
D= circular culvert diameter (ft)
TW= tailwater depth (ft) = 0.4D
g= acceleration due to gravity (32.2 ft/s^2)

This equation assumes wet conditions with tailwater equaling 0.4D. This assumption should be used to design forebays for both wet and extended dry detention basins.

Step 3: Use D_{50} to dictate riprap apron length and depth (FHWA HEC14)

The following table can be used to size the riprap apron's length and depth based on D_{50}. Note that D_{50} is in inches in the table.

D_{50} (in)	***Apron Length (L_A)***	***Apron Depth (D_A)***
5	4D*	$3.5D_{50}$
6	4D	$3.3D_{50}$
10	5D	$2.4D_{50}$
14	6D	$2.2D_{50}$

*D is the culvert rise.

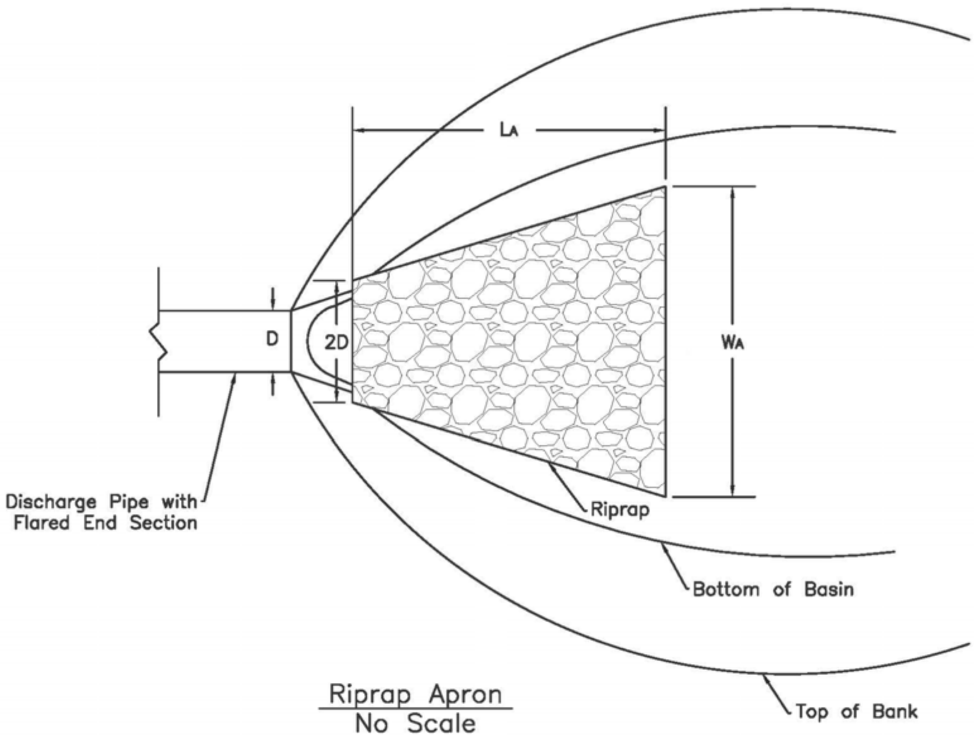
Table 1. Apron length and depth based on D_{50}

Step 4: Use D and L_A to determine the width at the downstream end of the apron

This width should be calculated based on a 1:3 flare:

$$W_A = 3D + 2/3\ L_A$$

Forebays



Riprap Apron
No Scale

Forebays

Step 5: Calculate discharge velocity to determine if the riprap apron is needed

If the velocity of stormwater leaving the discharge pipe is greater than 8 fps, then use a riprap apron or other energy dissipator in the forebay.

Design Flow Outlet Velocity	***Is an Energy Dissipator Required?***
< 8	No
8-15	Coordinate with Roadway Drainage Group.
>15	Yes

From NDOT Drainage and *Erosion Control Manual*, Chapter 2

Table 2: Requirements for Energy Dissipators

Step 6: If a riprap apron is needed, choose appropriate type

Based on D_{50}, select a suitable riprap type, as specified by NDOT, from the table below.

Riprap Type	***Median Diameter D_{50} (in.)***	***Maximum Diameter D_{100} (in.)***	***Minimum Depth Design / Thickness (in.)***
Rock Riprap Type A	9	15	18
Rock Riprap Type B	12	19	21
Broken Concrete Riprap	13	23	24
Rock Riprap Type C	15	25	27

Table 3: NDOT Riprap Properties

Step 7: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*.

Step 8: Calculate forebay volume ($V_{forebay}$)

The volume of the forebay is a percentage of the WQV; the percentage of WQV necessary is based on design depth. The more shallow the basin, the greater the design volume. Select the desired depth and calculate the forebay volume.

$V_{forebay}$= 10% × WQV (2 – 4 ft depth)
$V_{forebay}$= 15% × WQV (1.5 ft min depth)
$V_{forebay}$= 20% × WQV (1 ft min depth)

Step 9: Verify Forebay Area

Calculate the area (A) needed to maintain the desired depth of the forebay (D ≤ 4 ft).

$$A = V_{forebay} \div D$$

If riprap apron is necessary, compare A to the area of the apron from steps 3 and 4. If A exceeds the area of the apron, then design the forebay in accordance with A. If A is less than the area of the apron, then design the forebay in accordance with the area of the required apron and re-evaluate the forebay's volume and depth.

Step 10: For extended dry detention basin forebays only, size the stone weeper weir to pass larger design storms

A stone weeper weir will be used for the extended dry detention basin forebay. It can be designed by selecting a water depth above the crest and finding the effective weir length (L'), or by selecting an effective weir length and determining the water depth above the crest (h). Dry forebays will be designed with a minimum width of 4 ft at the top of the embankment. Corresponding discharge coefficients (C) for the minimum width with 1 ft of head (h) are typically 2.70.

Broad-crested weir: $Q = CL'h^{3/2}$

C = dimensionless weir discharge coefficient (2.70 typical)
L' = effective weir length, ft
h = water depth above the crest, ft

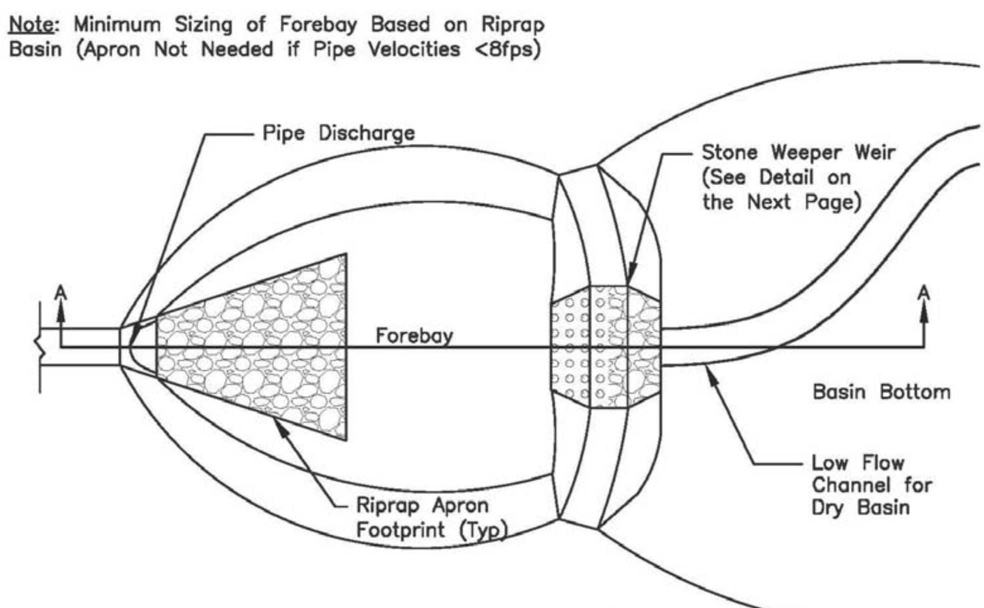
After establishing the effective weir length, Table 4 can be used to obtain the corresponding riprap length.

Effective Weir Length (L')	*Riprap Length (L)*
4 ft	16 ft
5 ft	17 ft
6 ft	18 ft
10 ft	22 ft
12 ft	24 ft

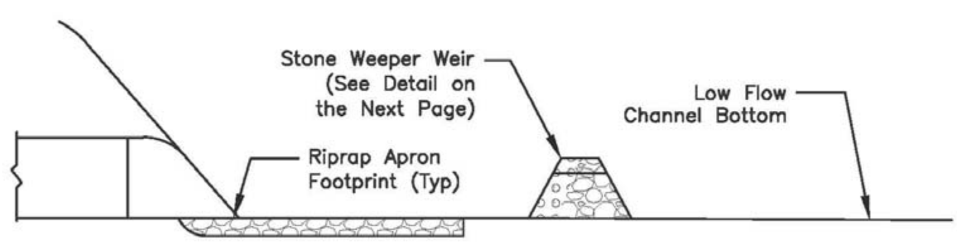
Table 4. Effective weir length (L') and corresponding minimum riprap (L) lengths.

Forebays

DESIGN EXAMPLES

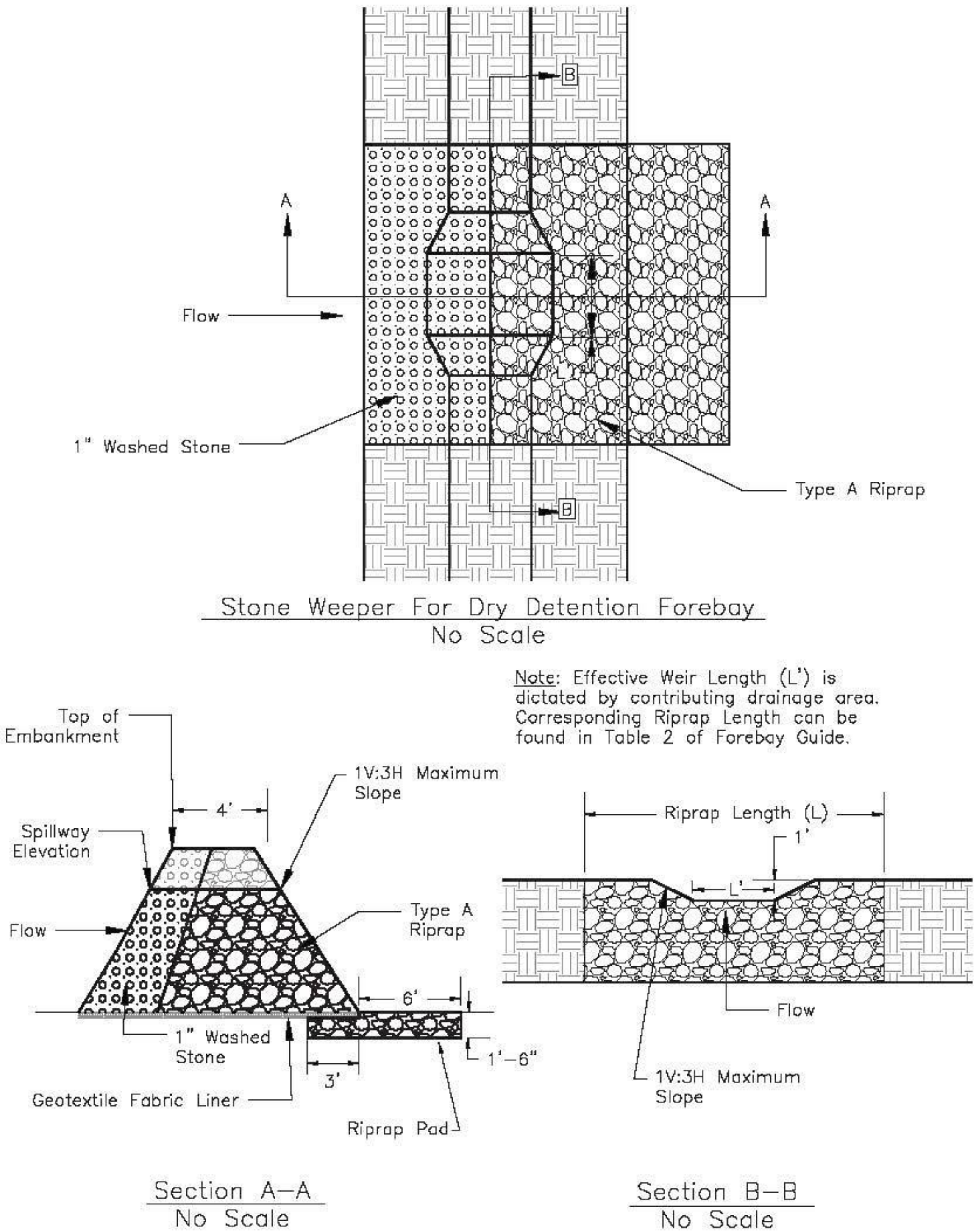


Forebay for Dry Basin
No Scale

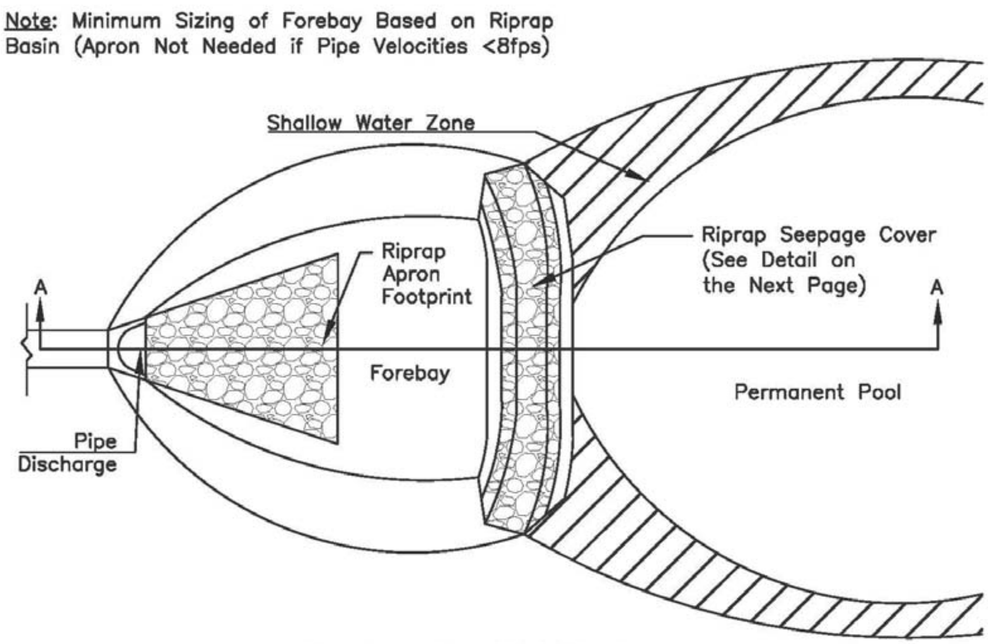


Section A–A
No Scale

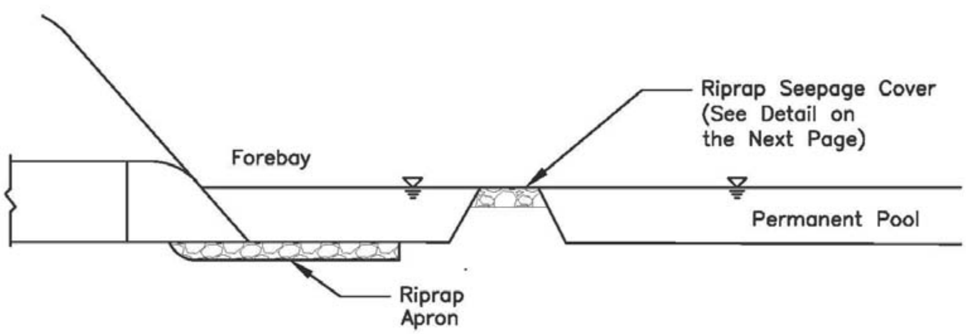
Forebays


B
A
A
Flow
1" Washed Stone
Type A Riprap
B
Stone Weeper For Dry Detention Forebay
No Scale
Note: Effective Weir Length (L') is dictated by contributing drainage area. Corresponding Riprap Length can be found in Table 2 of Forebay Guide.
Top of Embankment
4'
1V:3H Maximum Slope
Spillway Elevation
Flow
Type A Riprap
6'
1" Washed Stone
1'-6"
3'
Geotextile Fabric Liner
Riprap Pad
Section A-A
No Scale
Riprap Length (L)
1'
L'
Flow
1V:3H Maximum Slope
Section B-B
No Scale

Forebays

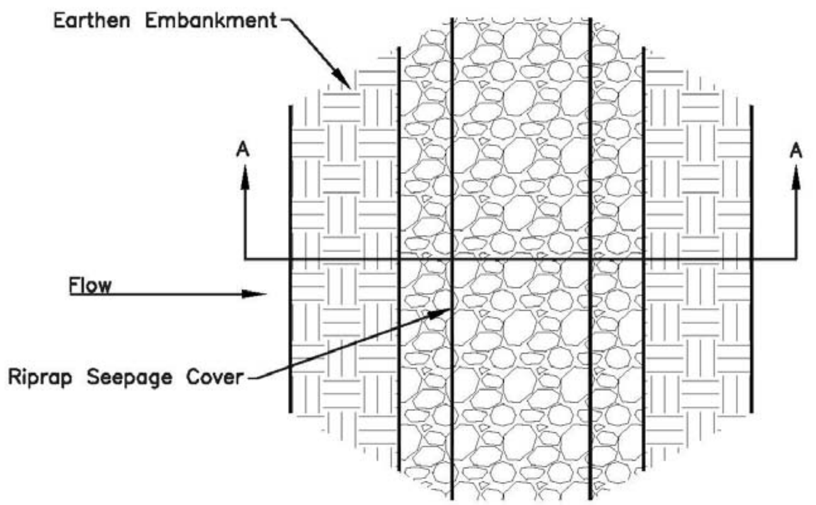


Forebay for Wet Basin
No Scale

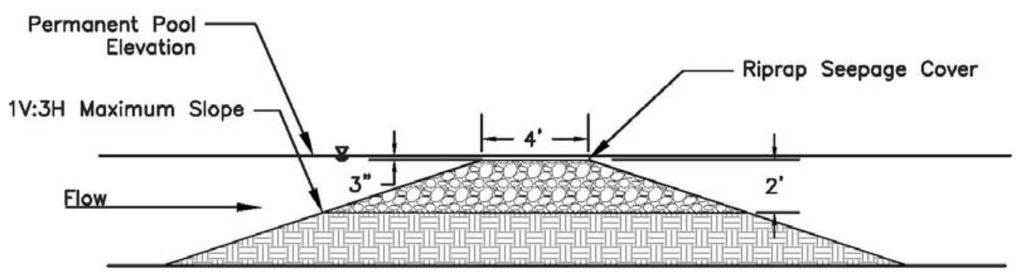


Section A–A
No Scale

Forebays



Embankment w/ Riprap Seepage Cover
For Wet Detention Basin Forebay
No Scale



Section A–A
No Scale

Forebays

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction.
- Set up a water diversion to prevent flows to the forebay during forebay construction if it is not being used for sediment control during site construction.
- It may be possible to use the forebay as a sediment trap during construction. However, the sediment may have to be removed periodically during construction and again once the site has stabilized.
- The Stone Weeper weir should be installed after construction is completed to prevent early clogging. A temporary earthen berm section may take its place during construction.
- The riprap seepage cover should be installed after construction is completed to prevent early clogging.

MAINTENANCE AND INSPECTION REQUIREMENTS

The maintenance objectives for a forebay include providing litter control, monitoring erosion and sedimentation, and maintaining stone weeper weirs and riprap seepage covers. Maintenance activities specific to wet forebays will be **bold** and those specific to dry forebays will be *italicized*.

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• *Inspect forebay after any major rainfall event for any unintended surface ponding or indicators that water has ponded for an extended period of time.* • *Check forebay basin 3 days (72 hours) after a major rainfall event to ensure drainage of the basin.* • **Inspect forebay for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage).** • Inspect forebay basin to ensure the intended vegetation is establishing well. Consider reseeding if needed. • Inspect forebay basin for erosion and any damage by equipment or vehicles after every major rainfall event. Repair as needed. • Inspect forebay for sediment buildup in the basin or at outlet structures. Remove sediment when 50% of capacity is lost. • Inspect for trash and debris in the forebay and around any inlets and outlets. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

Forebays

Frequency	Inspection and Maintenance Activity
Establishment Status: As required in the Nebraska Construction Stormwater General Permit	• *Inspect forebay for any surface ponding or indicators that water has ponded for an extended period of time.* • *Check forebay 3 days (72 hours) after a major rainfall event to ensure drainage of the basin.* • **Inspect forebay for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage).** • Inspect forebay to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect forebay for erosion and damage by equipment or vehicles. Repair as needed. • Inspect forebay for sediment buildup on the bottom of the basin. Remove sediment when 50% of capacity is lost. • Inspect the forebay, inlets, and stone weepers/riprap covered embankments for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.
Annually (After NPDES Permit is closed).	• *Inspect forebay for any surface ponding or indicators that water has ponded for an extended period of time.* • **Inspect forebay for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage).** • Inspect forebay to ensure there is a dense, uniform stand of the intended vegetation. Consider reseeding if needed. • Mow grass to control weeds. • Inspect forebay for erosion and damage by equipment or vehicles. Repair as needed. • Inspect forebay for sediment buildup on the bottom of the basin. Remove sediment when 50% of capacity is lost. • Inspect the forebay, inlets, and stone weepers/riprap covered embankments for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain pretreatment STFs in accordance with their respective design guidelines.

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Principal Spillways

OVERVIEW

Source: Douglas County, Nebraska

Definition

The principal spillway is an outlet system that may have a number of orifice and weir combinations to control discharge rates from a collection basin.

Benefits

- Allows the WQV to be discharged in a controlled manner.
- Allows large storm events to pass in a controlled manner.
- Allows drawdown of the basin if needed for maintenance.

Overview Table

Associated Costs	L	M	H
Design		X	
Construction			X
Maintenance			X
Pollutant Removal	**L**	**M**	**H**
Suspended Solids	-		
Nutrients	-		
Heavy Metals	-		
Hydrocarbons	-		

Limitations

- Drawdowns and other orifices within the principal spillway can become clogged with sediment.
- Trash and debris can clog and potentially damage the spillway structure and embankment due to overflows.
- May require frequent maintenance depending on trash, debris, and sediment loading.

STORMWATER TREATMENT FACILITY (STF) DESCRIPTION

The principal spillway is the primary outlet for a stormwater detention facility; it is also known as the outlet works. Principal spillways can be comprised of drop inlets, pipes, weirs, orifices, chutes, channels, or any combination thereof. Principal spillway conduits are usually constructed through an embankment and carry a 10 yr. and/or lesser events in a controlled manner. To reduce the risk of seepage through the embankment along the pipe, anti-seepage collars should be used.

Emergency spillways are used to provide controlled flow for storms that exceed the maximum design storm for the principal spillway. This secondary structure is typically a weir which is separate from the primary outlet structure; it is constructed as part of the basin's embankment. The emergency spillway is sized to handle storms in excess of the principal spillway design storm with a minimum of 1 foot of freeboard between the water surface elevation and the top of the embankment.

STF Components

Berm (Embankment) - A berm is a compacted earthen ridge designed to capture and detain stormwater flows for a given design storm. Consideration should be given to the velocity of flow over the emergency spillway and at the toe. Protection or energy dissipation should be provided where velocities and turbulence are a concern.

Total Depth - The total depth of the basin from the lowest elevation of the bottom to the top of the berm or embankment. This depth includes the capacity required to handle the Water Quality Volume (WQV) and any additional storage for peak flow reduction. It includes a minimum of 1 foot of freeboard from the design water surface elevation in the emergency spillway to the top of the berm or embankment.

Outlet Structure - An outlet structure is a standpipe or structure designed to draw down and route excess stormwater that is stored in the basin. When extended detention is desired, a multi-stage structure is typically needed. For extended dry detention basins, a pipe or an orifice opening is typically provided at the bottom of the basin to draw down the WQV. For wet detention and stormwater wetlands the WQV opening sets the permanent pool elevation. Other controls (orifice or weir) are provided to draw down larger storm events (i.e., 2 yr. & 10 yr.) at predetermined maximum rates.

WQV Discharge Pipe - WQV, or the first ½ inch of runoff, will be discharged from the detention facility through the utilization of a perforated riser pipe and control valve. The perforated riser will be used to screen debris and trash and will be braced to restrict movement. 1”-3” limestone rock placed around the perforated pipe for the extended dry detention basin helps keep sediment contained to the basin. For wet detention basins, perforations will draw the WQV down to the permanent pool level. Whereas in extended dry detention basins the perforated riser will drawdown to the basin bottom dry condition. In either situation, the perforated risers will connect to control valves that regulate discharge rates.

Principal Spillways

Drawdown Pipe - Wet detention facilities will need an additional drawdown pipe in case the basin must be emptied for inspection and maintenance. The drawdown will be regulated by a control valve.

Anti-seep Collar - Water travels within a pipe, but it can also travel along the outside of a pipe. Seepage along a pipe can undermine the earthen embankment as well as cause washouts.

Anti-seep collars are used to prevent this piping along the outside of the pipe and therefore maintain structural integrity of the embankment. Anti-seep collars can be comprised of PVC, concrete, rubber, or plastics.

Anti-vortex Device - Water that enters the principal spillway outlet structure can be classified as orifice or weir flow. If orifice flow governs, then air-entrainment can occur, causing a loss of efficiency and potential cavitation. Anti-vortex devices, such as a baffle should be utilized in these situations to prevent a vortex, which will in turn prevent air-entrainment.

Principal Spillways

DESIGN CONSIDERATIONS

Questions to ask yourself…

Q. Is the spillway for a wet or an extended dry detention basin?

Q. What size is the design storm, or is the design based on the WQV?

Q. Will orifice hoods or trash racks be utilized to prevent debris from entering the spillway?

Q. Are anti-seepage or anti-vortex devices needed for this design?

Consider the ramifications of where the principal spillway is placed. It should be close to the shoreline and easily accessible for maintenance activities. It is typically positioned adjacent to the basin's embankment to minimize outlet pipe length. Consideration should be given to constructability, accessibility for maintenance and inspection, and the routing of storms during construction. Thought should be given to public safety and proper placement of safeguards to prevent the structure from tampering. The designer should also reference design guides for whatever STF is utilized (i.e., wet detention, extended dry detention, or stormwater wetland).

The WQV discharge pipe or structure is an essential component of the principal spillway system. This feature should be designed to allow the reservoir to be drained in a reasonable amount of time. 1”-3” limestone should be placed around the perforated pipe openings to reduce the risk of clogging. For wet detention basins and stormwater wetlands, the perforation will begin above the permanent pool volume to capture the WQV, and an additional valved drawdown pipe is needed for occasional pond drainage. For the extended dry detention basin, the perforated riser is designed to capture the WQV as well as function as a drawdown pipe. WQV drawdown for extended detention on both wet and dry detention basins will be regulated by valves that will be designed one standard opening size larger than needed and constructed to reduce drawdown time to the desired period. Additionally, both situations use oversized perforated pipes. These pipes are designed so that the minimum area of perforation openings is four times larger than the orifice opening size. This allows the valve to govern drawdown or drainage at a controlled rate even if some of the perforations are plugged.

Principal spillways can include a combination of weirs and orifice flows to convey the design floods in addition to conveying the WQV. Orifices may be hooded; this prevents surface debris from entering the orifice. It is an easy screening process that obtains water from below the water's surface. Hoods or shrouds, if used, should be secured to the structure and can be fabricated from stainless or galvanized steel. Plastic proprietary shrouds may also be used if approved by NDOT.

Trash racks and anti-vortex devices are utilized to maintain the structural integrity of the principal spillway. Trash racks must be used on weir openings to prevent debris from entering the spillway structure. Trash racks can be made of metal or plastic and have openings that are typically 6”-12”. Options can be found in NDOT's Approved Products List (APL). Anti- vortex devices may be used if orifice flow occurs in the principal spillway riser pipe. These devices prevent air-entrainment and cavitation in the pipe.

Principal Spillways

Principal spillway conduits are usually constructed through an embankment. Anti-seepage collars should be used for smooth pipe larger than 8 inches in diameter and for corrugated pipe larger than 12 inches in diameter. They are used to prevent seepage through the embankment along the outlet pipe and should extend at least 2 ft in all directions around the pipe. Collars should be placed a minimum of 2 ft away from pipe joints, unless flanged joints are used, and should be placed often enough to increase the seepage length by a minimum of 15%. After backfilling, hand powered compaction should be used near the collar to prevent any damage.

At the end of the principal spillway conduit there must be energy dissipation such as a riprap pad or plunge pool to prevent erosion on the downstream side of the embankment.

DESIGN CRITERIA

Description	Value
Water Quality Volume (WQV)	Reference Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual* Volume routed through perforated riser
Setback Distances	Reference NDOT's STF Guide for the Appropriate STF
WQV Drawdown Time	24 – 72 hours
Freeboard	Minimum of 1 ft above the emergency spillway water surface elevation (WSEL)
Trash Rack	6"-12" typical opening size Select from NDOT's Approved Products List (APL)
Anti-Vortex Device	Needed if orifice flow occurs. Dictated by ratio of hydraulic head (H) to riser pipe radius (R) H/R < 0.5 : Weir Flow 0.5 < H/R < 1 : Transitional Flow H/R > 1 : Orifice Flow
Anti-Seep Collar	Needed if the outlet pipe is a smooth pipe larger than 8 inches in diameter or a corrugated pipe larger than 12 inches in diameter Must extend 2 ft around pipe in all directions and be 3 ft thick for RCP
Perforated WQV/ Drawdown Pipe Opening Area	Four times the area required for the WQV orifice opening
Average Depth	**See additional guides: Wet Detention, Extended Dry Detention, and Stormwater Wetlands** Wet Detention (permanent pool) = 3-7 feet Extended Detention depth= <3 feet Stormwater Wetlands= 6 feet

Principal Spillways

DESIGN PROCEDURE

Step 1: Calculate Water Quality Volume (WQV)

Calculate WQV as defined in Chapter 3 of NDOT's *Drainage Design and Erosion Control Manual*. Reference other guides (i.e., wet detention, extended dry detention, stormwater wetlands) to size the reservoir based on WQV.

Step 2: Determine design peak discharge

Calculate discharge rates for each design storm as defined in Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual.* Provide additional storage as needed to reduce peak runoff in the detention facility.

Step 3: Size the orifice for WQV drawdown

Use the average discharge rate and the average hydraulic head to calculate the orifice size for the Minimum WQV Drawdown Time provided in the design criteria table above.

Find the average discharge rate:

$$Q = WQV/t/3600$$

Q = average orifice discharge rate (cfs)
t = WQV drawdown time (hours)

Find the orifice area:

$$A = Q/[C * \sqrt{2gh}]$$

A = orifice area (ft^2)
C = orifice discharge coefficient, dimensionless (0.60 typ.)
g = acceleration of gravity (32.2 ft/s^2)
h = average hydraulic head (ft)
(height measured from orifice invert to midpoint of extended detention depth – assumes orifice is small relative to total height)

Find the orifice diameter

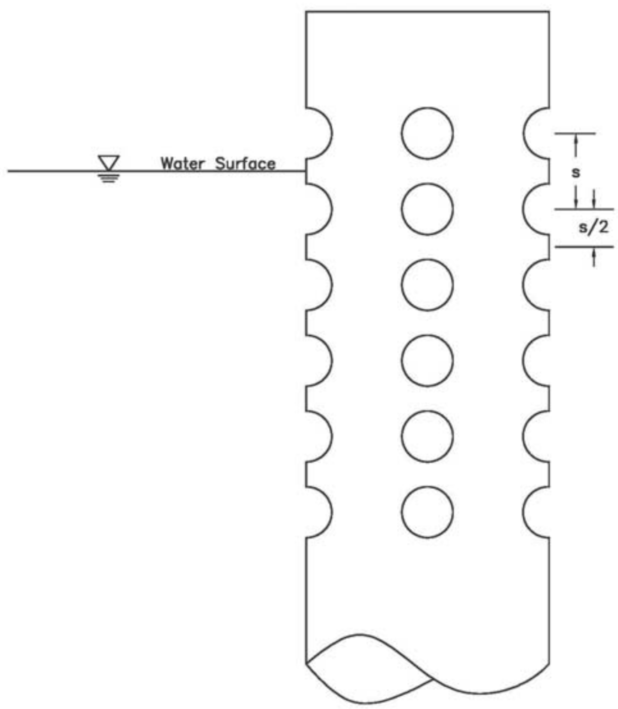
$$d = \sqrt{\frac{4A}{3.14}} * 12$$

d = orifice diameter (in)

For both dry and wet basins this perforated riser will be utilized to screen trash and debris, but a control valve will regulate drawdown. The total area of perforated openings should be oversized, to account for potential clogging over time. As a minimum, the total area of perforated opening (A_{design}), should be four times the orifice opening.

$$4 * A \leq A_{design}$$

Principal Spillways



Perforated Riser Pipe Detail
No Scale

Principal Spillways

The following tables can be used to determine how many perforations are needed:

Hole Diameter (in)	Area (in^2)
1/8	0.013
1/4	0.049
3/8	0.110
1/2	0.194
5/8	0.307
3/4	0.442
7/8	0.601
1	0.785

Table 1: Diameter and Area for Holes in Water Quality Drawdown Pipe

Riser Diameter (in)	Hole Diameter, inches ¼”	½”	¾”	1”
	# per row *(area per row - in^2)*	# per row *(area per row - in^2)*	# per row *(area per row - in^2)*	# per row *(area per row - in^2)*
4	8 *(0.392)*	8 *(1.552)*	--	--
6	12 *(0.588)*	12 *(2.328)*	9 *(3.978)*	--
8	16 *(0.784)*	16 *(3.104)*	12 *(5.304)*	8 *(6.28)*
10	20 *(0.980)*	20 *(3.88)*	14 *(6.188)*	10 *(7.85)*
12	24 *(1.176)*	24 *(4.656)*	18 *(7.956)*	12 *(9.42)*

Table 2: Maximum Number of Perforated Columns per Row

Principal Spillways

Buoyancy calculation:

Flotation of the riser structure can lead to a failed connection between the riser and the barrel, and even failure of the embankment. Buoyancy for both perforated and partially perforated pipes should be calculated and the size of the riser should be selected with this in mind.

Step 4: Size the orifice or weir for design storms

Outlets for design storms can be a combination of orifice and weir flow. Provide calculations or model inputs and outputs to ensure that the outlets and pipe for the spillway are sized in accordance with a design storm using Chapter 1 of NDOT's *Drainage Design and Erosion Control Manual.*

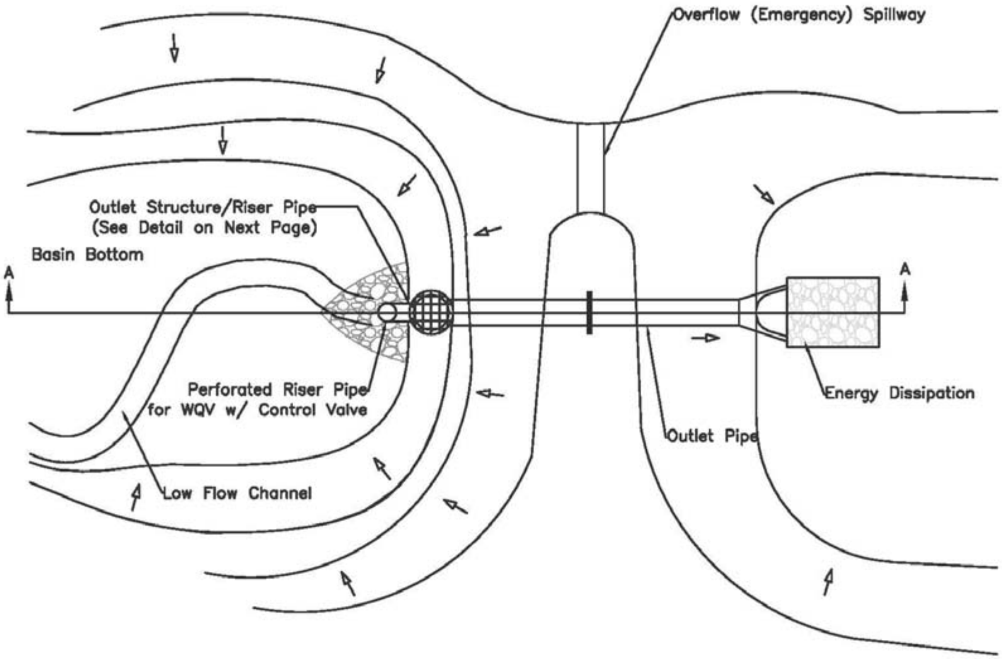
Step 5: Verify weir/orifice conditions to determine if anti-vortex device is needed

The 10-yr storm is typically the largest storm event routed through a riser pipe. Using the selected model determine H, the water depth above the weir (ft), and R, the radius of the riser (ft). The principal spillway is governed by weir flow if the H/R ratio does not exceed
0.5. If it is designed to allow for water depths that cause H/R ratio to exceed 0.5 then the flow is transitional. If the H/R ratio is 1 or greater than orifice flow dominates and anti- vortex devices are needed within the spillway.

Step 6: Size the emergency spillway for larger design storms

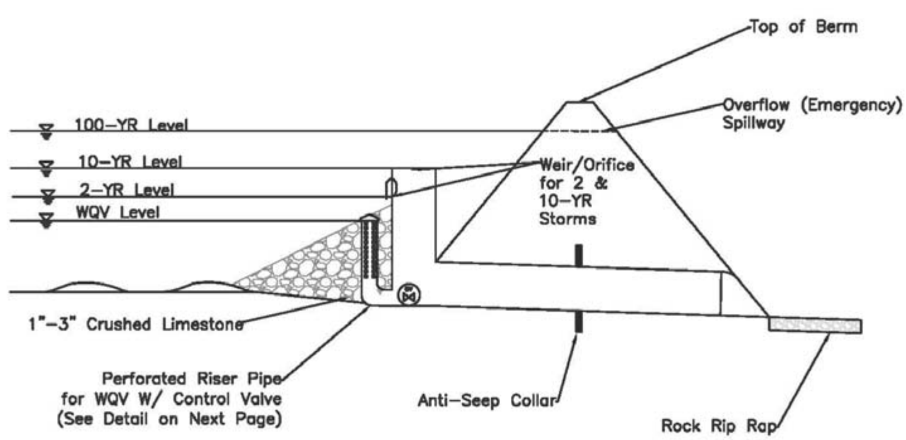
The emergency spillway earthen embankment should be sized for design storms that exceed capacity of the principal spillway. Typically they are designed for 100-yr storms with 1 ft of freeboard above that water surface elevation. The emergency spillway should be designed to withstand the anticipated shear stresses and erosion.

DESIGN EXAMPLES



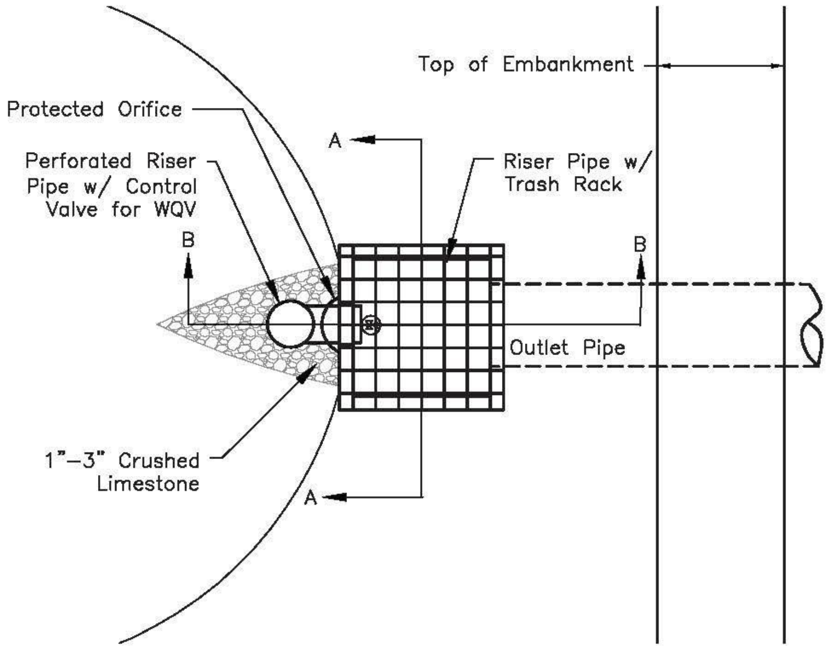
Principal Spillway for Extended Dry Detention
No Scale

Design storms are used for example only.
They may vary for each design.



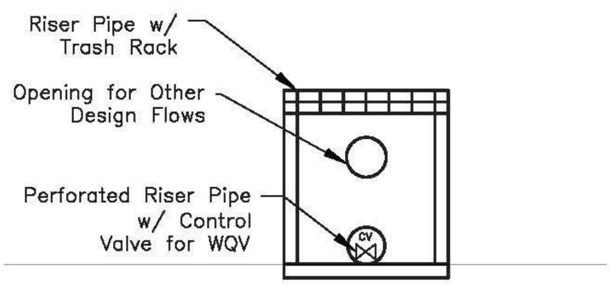
Section A–A
No Scale

Principal Spillways



Detail of Principal Spillway for Dry Basin

No Scale



Section A–A

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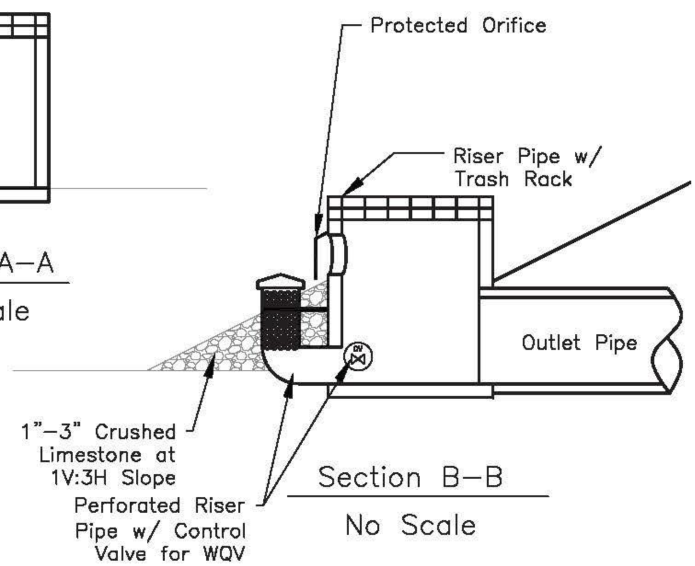
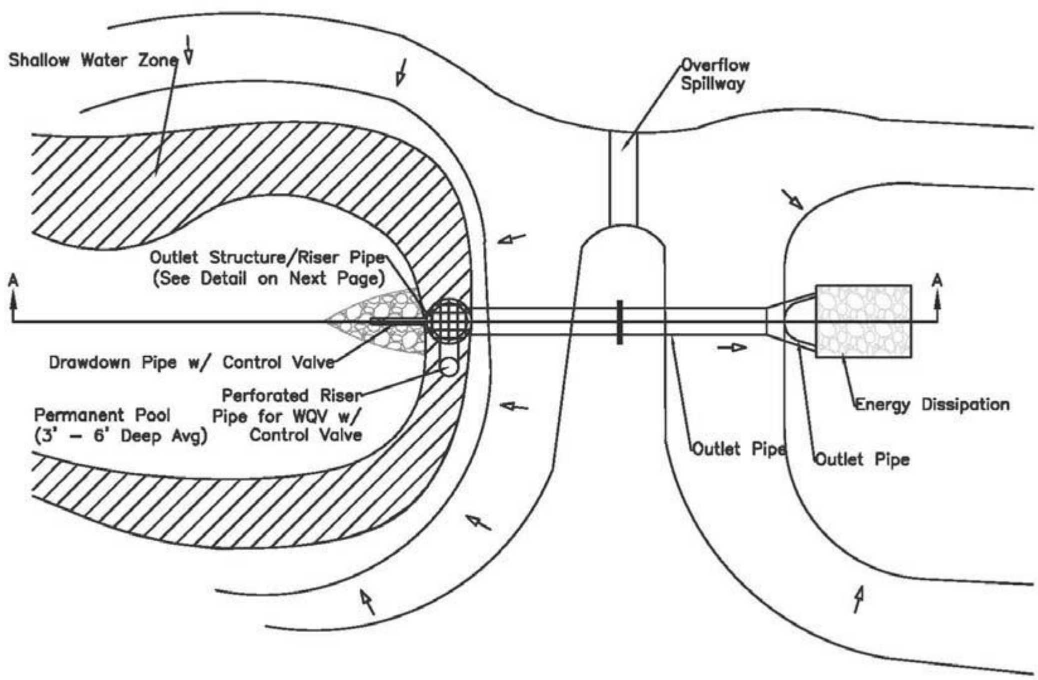


Section B–B

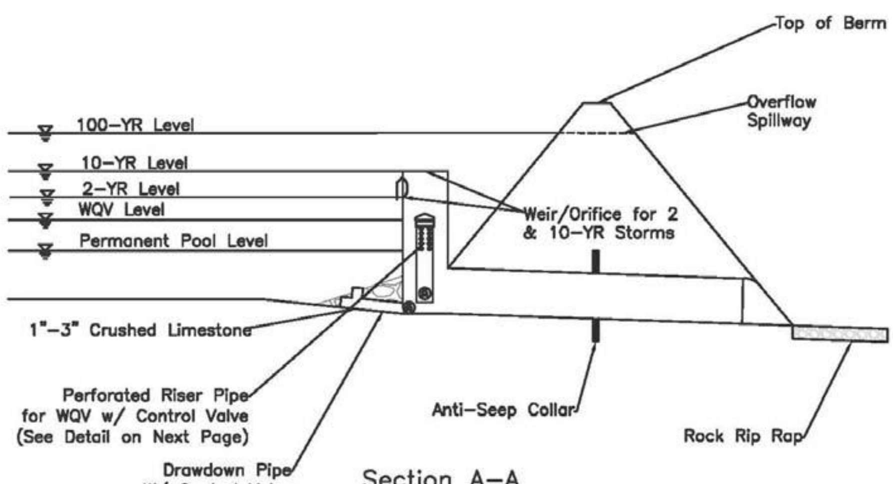
No Scale

Principal Spillways



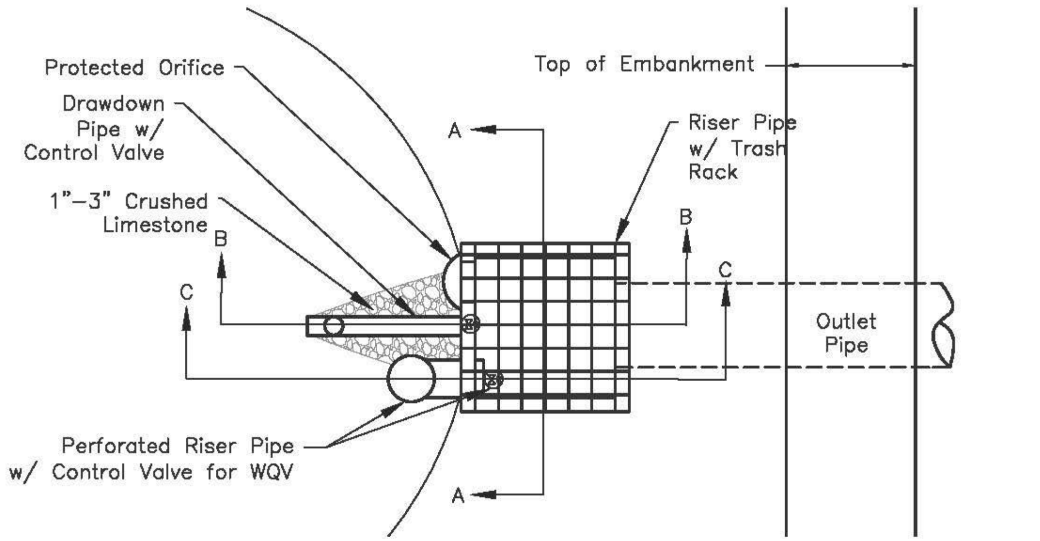
Principal Spillway for Wet Detention Basin/Stormwater Wetland
No Scale

Design storms are used for example only.
They may vary for each design.



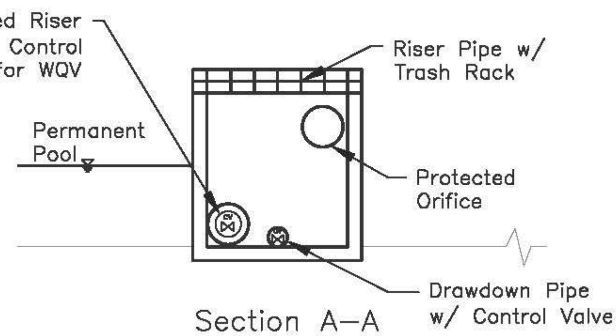
Section A–A
No Scale

Principal Spillways



Detail of Principal Spillway for Wet Basin/Stormwater Wetland

No Scale



Section A–A

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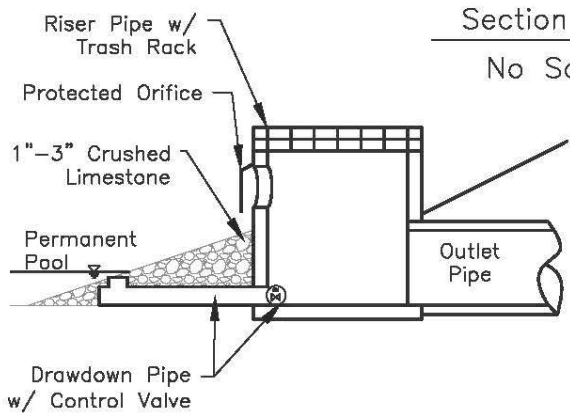

Section B–B

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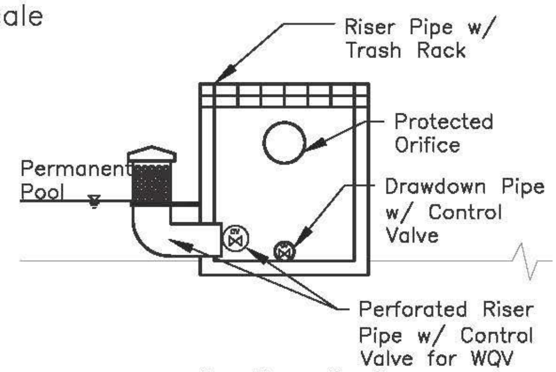

Section C–C

No Scale

CONSTRUCTION CONSIDERATIONS

- Consider in advance the construction sequencing and erosion and sediment control practices during site construction.
- The basin may be used to control sediment during construction. However, the sediment will have to be removed before completing the project unless additional sedimentation volume has been incorporated into the design and that design amount has not been exceeded.
- Basins used to control sediment during construction may need to be converted to a post-construction permanent structure. Site should be stabilized prior to conversion, and perforated riser pipe and 3" protective stone should be installed last.
- Ensure stability on the downstream side of the outlet pipe with riprap or another energy dissipating device to reduce erosion.
- Build the principal spillway while the system is offline. Make provisions to divert water while constructing spillway if constructed online.
- Silt traps can be converted to wet basins. To prevent silting in of the principal spillway the system should be offline or water diverted, and the perforated riser pipe and 3" protective stone should be installed last.

Principal Spillways

MAINTENANCE AND INSPECTIONS REQUIREMENTS

The maintenance objectives for a spillway include providing litter control, maintaining the spillway structure, and preventing erosion.

Maintaining Principal Spillway Structure

Principal spillway structures can be clogged with sediment, trash, or other debris. Trash racks can be utilized to prevent clogs within the riser pipe. The trash rack should be checked to make sure it fits securely to the outlet structure. Spillways plugged with debris or trash reduce the capacity of the spillway. Additionally, the plugged spillway may cause more frequent flows in the emergency spillway. Therefore, the trash rack and all openings should be checked for obstructions from trash, debris, and sediment. They should be inspected annually and cleaned out.

Rock protection drawdown pipes from sediment should be inspected, and sediment volumes in the basin should be checked.

Valves should be inspected for blockage, leaks, corrosion, or any sort of wear and repaired as soon as possible.

Conduits should be inspected thoroughly once a year for sagging, displacement at joints, cracks/leaks, signs of piping, any sort of surface wear or corrosion, and lastly, blockage.

Outlet structures should be inspected annually for blockage from trash or other debris; wear of the structure should be assessed and fixed. The inspection should also look for signs of seepage or piping along the outside of the pipe at or near the outlet. This might include erosion around the edges or top of the outlet, unexpected seepage during dry conditions, and unexplained sediment near the outlet. Energy dissipators such as riprap aprons shall be inspected annually and should be replaced if ineffective or silted in. Erosion issues should be assessed and repaired.

Embankment and Emergency Spillway

Seepage can also lead to erosion of soil and loss of stability on the downstream side of the embankment which may cause failure of the dam. Signs of embankment cracks or settling should be remedied. Cracking and settling can lead to structural failure of the embankment as well. Lastly, surface erosion can cause a structural failure and therefore it should be prevented.

Principal Spillways

Frequency	Inspection and Maintenance Activity
Construction Status: As required in the Nebraska Construction Stormwater General Permit.	• Check basin 3 days (72 hours) after a major rainfall event to ensure drainage of the basin to permanent pool elevation. • Inspect system after any major rainfall event for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect inlets, outlet pipe, and downstream side of outlets for erosion and any damage by equipment or vehicles after every major rainfall event. Repair as needed. • Inspect basin for excessive sediment buildup on the bottom of the basin and at any inlets, outlet pipes, and forebays. Remove sediment as needed. • Remove trash and debris from the basin and any inlets, outlet pipes, and forebays. In particular, remove trash and debris from trash racks and openings in the inlet structure. • Inspect and maintain STFs in accordance with their respective design guidelines.
Establishment Status: As required in the Nebraska Construction Stormwater General Permit.	• Check basin 3 days (72 hours) after a major rainfall event to ensure drainage of the basin to permanent pool elevation. • Inspect basin for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect inlets, outlet pipe, and downstream side of outlets for erosion and any damage by equipment or vehicles after every major rainfall event. Repair as needed. • Inspect basin for excessive sediment buildup on the bottom of the basin and at any inlets, outlet pipes, and forebays. Remove sediment as needed. • Inspect the basin and outlets for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain STFs in accordance with their respective design guidelines.

Principal Spillways

Frequency	Inspection and Maintenance Activity
Annually (After NDPES Permit is closed).	• Inspect to ensure drainage of the basin to permanent pool elevation. • Inspect wet detention basin for a water surface elevation that is consistently lower than design permanent pool elevation. If so, investigate to determine the cause (such as inflow, drought, or excessive seepage). • Inspect conduits and trash racks for clogging or any damage. Inspect the road on the crest of the embankment for any damage. • Inspect the principal spillway and embankment for any cracks, slides, sloughing, and settlement. • Inspect inlets, outlet pipe, and downstream side of outlets for erosion and any damage by equipment or vehicles. Repair as needed. • Inspect basin for excessive sediment buildup on the bottom of the basin and at any inlets, outlet pipes, and forebays. Remove sediment as needed. • Inspect the basin and outlets for trash and debris, erosion, sediment buildup, and structural damage. Repair as needed. • Inspect and maintain STFs in accordance with their respective design guidelines.

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The information contained in Appendix Q: Stormwater Treatment Form C - Maintenance, dated September 2013, has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

DRAFT FORM | 2013

NDOT Stormwater Treatment within MS4 Communities

Form C – Maintenance

Project Name: ______________________ **Project #:** ______________ **CN:** __________

Project Designer: ______________________

STF 1

STF Title: ______________________

Location: ______________________ Receiving Water: ______________________

General Maintenance Requirements:

Special Maintenance Requirements:

STF 2

STF Title: ______________________

Location: ______________________ Receiving Water: ______________________

General Maintenance Requirements:

Special Maintenance Requirements:

STF 3

STF Title: ______________________

Location: ______________________ Receiving Water: ______________________

General Maintenance Requirements:

Special Maintenance Requirements:

DRAFT FORM | 2013

STF 4

STF Title: ______________________________

Location: ______________________________ Receiving Water: ______________________________

General Maintenance Requirements:

____

____

Special Maintenance Requirements:

____

____

STF 5

STF Title: ______________________________

Location: ______________________________ Receiving Water: ______________________________

General Maintenance Requirements:

____

____

Special Maintenance Requirements:

____

____

STF 6

STF Title: ______________________________

Location: ______________________________ Receiving Water: ______________________________

General Maintenance Requirements:

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Special Maintenance Requirements:

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FORM C
Submitted to Operations Division (Name/Title)______________________________ (Date)__________

Attachments: Corresponding STF Design Guides, GIS Project Map of STF Locations

Nebraska Department of Transportation
Operating Instruction 45-5
January 17, 2001

AGREEMENTS

1. **Purpose:** To provide policy for the preparation, distribution, and disposition of agreements between the department and an outside party. This DOT-OI supersedes DOT-OI 45-5 dated November 2, 1992. The office of primary responsibility is the Project Development Division.

2. Due to the extensive number of agreements, the variable nature of technical performance, and the governmental requirements originating in many different areas, the Project Development Division (PDD) will prepare, coordinate, distribute, monitor, and maintain departmental agreements, excluding those agreements associated with bid lettings, right-of-way acquisition, purchasing, and the contracts and bonds for highway construction. Those divisions which normally prepare their own agreements, or use a standard form of agreement, will submit prepared agreements to the PDD for review prior to execution by outside parties.

3. When necessary, managers will request that agreements be prepared by the PDD and will submit a DR Form 65, "Request for Agreement."

4. Agreements will be reviewed "in-house" by the Controller Division and PDD prior to execution by any party.

5. Except for standard agreements which have had prior review, all agreements prepared outside the PDD should be submitted to the PDD for review before negotiations are begun. When a standard agreement is revised, it should also be submitted for review.

6. The originating office is responsible for obtaining the signatures of parties outside the department. Following execution by an outside party, agreements will be **hand-carried** to the PDD for internal coordination.

7. Internal coordination will be accomplished by using a RDP Form 656-A, "Agreement Monitoring System - Agreement File Update." The originating office will complete the description and coordination portion of the form. The computer portion will be completed by the PDD.

8. Individuals to whom agreements are routed for coordination will promptly coordinate and have the agreement hand-carried to the next office indicated on the coordination sheet and immediately advise the PDD, via telephone, if he/she disagrees with the contents of the agreement or believes that further coordination with other offices is required.

9. Signatory responsibility for agreements is defined by DOT-OI 45-6.

Operating Instruction 45-5

10. Following execution by the department and approval (when necessary) of the FHWA, the PDD will coordinate with the originating office for the distribution of the agreements. The department's original, or a copy if the department is not a party to the agreement, will be retained in the PDD files. The PDD will then enter the basic data into the computerized agreement monitoring system.

11. Agreements retained in the PDD files will be microfilmed after ten years from the execution date. At that time, most originals will be destroyed. Agreement microfilm cards and a reader are available in the PDD for use by others.

12. The agreement number, which is a descriptive number indicating the originating office, agreement type, year, and sequence will be used by the PDD for filing purposes. Cross-reference listing by the agreement number, control number, and project number will be available in the PDD.

13. **Requirements for Agreement Preparation and Review:**

    A. **Offer, acceptance, and a "meeting of the minds":**

       (1) The preliminary definition of performance specifications is normally established through personal contact and written proposals.

       (2) Where federal financing is involved, it is imperative to caution against beginning work before receiving notice to proceed, since work performed before federal authorization is ineligible for federal reimbursement.

       (3) Failure of the contracting parties to interpret and understand all contract provisions identically tends to generate misunderstandings, disagreements, and legal problems. Accordingly, check each contract or agreement carefully for clarity and complete coverage of performance requirements as it affects each party to the contract.

       (4) A "closing conference" with all parties is highly recommended on agreements containing complex performance details to assure complete understanding before contract execution.

    B. **Cost Principles:** Managers will establish cost principles for use in determining the allowability of individual items of cost. These cost principles will be appropriately identified or referenced in each contractual document. If federal-aid is involved, cost principles will be those established by the applicable provisions of the governing Federal-Aid Highway Program Manual, grant agreement, Office of Management & Budget circular, other directives, and the contract cost principles and procedures set forth in the Federal Acquisition Regulations System (48 CFR, 1.31), as appropriate.

Operating Instruction 45-5

C. **Consideration:**

(1) Check each agreement to see if the following are clearly answered. What are the pay items? When will payment be made? How will the amount to be paid be determined? Who is to be paid and who pays? Form of payment, i.e., cash offset against cost sharing, etc.?

(2) In some cases, the requirement of consideration may be satisfied without monetary payment for work performance. An example of this would be obtaining covenants from counties, cities, and other political subdivisions to cause certain restrictions and to perform certain acts in consideration of the department making certain highway revisions either on its own behalf, or as an agent for the FHWA.

D. **Performance:**

(1) Extra care and attention in defining and describing the detailed work to be done will do much toward eliminating misunderstanding, extra correspondence, and the need for supplemental agreements. Check each agreement carefully to see if the following are clearly answered. What is to be done? When will it be done? Where will it be done? Who will do it? How will it be done?

(2) Attention should be directed to provisions in event of nonperformance.

(3) Where applicable, attention should be directed to provisions concerning the handling of credits for materials recovered.

E. **Authority:** Contracts and agreements should be thoroughly checked for accurate inclusion, reference, and compliance with applicable laws, rules, and regulations.

F. **Cost Sharing:** Contracts and agreements should be reviewed for clear definition of the participants, sharing formula, when participants contribute, and how participants contribute.

G. **Covenants:** Contracts and agreements should be reviewed for clear definition of stewardship, liability, inspection, audit permission, retention of records, Disadvantaged Business Enterprises, and nondiscrimination.

H. **Guidelines for Requesting Contract and Agreement Reviews from the Legal Counsel:**

(1) Agreements covering simple work performance and nominal amounts of consideration -- no review necessary.

(2) Standard agreements and contracts -- request a review once for form and legal sufficiency unless changed and after each state legislative session or issue/revision of applicable FHWA publications.

(3) Agreements and contracts involving complex provisions – review regardless of consideration amount.

(4) Formulation of policy is not a responsibility of the Legal Counsel. Questions involving policy will be taken to the applicable deputy director.

I. **Guidelines for Requesting Contract and Agreement Reviews by the Operational Analysis & Audit Division (OAAD):**

(1) Contracts exceeding $50,000 must be sent to the OAAD for a pre-award audit.

(2) Contracts not based on a firm, fixed price must go to the OAAD for a post-audit to determine the total allowable contract costs.

J. **General Guidelines:**

(1) Avoid indefinite or ambiguous language and be explicit. The terms "and/or" should never be used.

(2) Insure that each agreement includes all of the proper parties, but not more parties than necessary to perform the subject of the agreement.

(3) Do not automatically make the state a party to every agreement -- only when necessary.

(4) FHWA publications will be included by reference, as applicable.

Monty W. Fredrickson
Deputy Director-Engineering

The Nebraska Division of the FHWA approved this appendix for use on the National Highway System and other federal projects on December 19, 2018.

Floodplain Management Guidelines – Compliance for Impacts to Floodplains

A.1 Introduction

This guidance pertains to floodplain considerations in the design of federal-aid projects, along with cross-references to the National Environmental Policy Act (NEPA) (https://ceq.doe.gov/) documentation requirements satisfied by the **Nebraska Department of Transportation** (**NDOT**) **Roadway Design Division** (**Roadway**) or **Bridge Division** (**Bridge**) in coordination with the **NDOT Environmental Section**. Additional information regarding floodplains may be found in other **NDOT** manuals and guidance documents, for example the Environmental Procedures Manual (https://dot.nebraska.gov/projects/environment/pubs/docs/), the Nebraska Categorical Exclusion Guidance(https://dot.nebraska.gov/projects/environment/pubs/docs/), and **NDOT's** Public Involvement Procedure (located in the Environmental Procedures Manual, Chapter 9).

The purpose of this guidance is to provide **NDOT's** process for the location and hydraulic design of highway encroachments on floodplains for **NDOT** and local projects using federal-aid funds. These guidelines contain an introduction to floodplains and floodways, an overview of the laws applicable to **NDOT** regarding floodplains and floodways, the Professionally Qualified Staff (PQS) memo procedure for documenting location hydraulic studies required by 23 Code of Federal Regulations (CFR) 650.111 (23 CFR §650 Bridges, Structures, And Hydraulics - Code of Federal Regulations (ecfr.io)), and the floodplain certification process completed by **NDOT Hydraulics Staff** for the submittal of the floodplain development permit.

It is the policy of **NDOT** to follow **Federal Highway Administration** (**FHWA**) floodplain regulations as set forth in 23 CFR 650A whenever federal-aid funds are involved. However, issues relating to floodplains quickly become complex due to the existence of other applicable laws, regulations and guidance documents, including those provided by the **Federal Emergency Management Agency** (**FEMA**) and the **Nebraska Department of Natural Resources** (**NeDNR**). For the benefit of the design practitioner, this guidance provides reference to **FEMA** and **NeDNR** regulatory considerations, in addition to **FHWA** requirements, when applicable.

A.2 Floodplain/Floodway Discussion

As noted above, various statutes, regulations and guidance documents exist from different state and federal agencies pertaining to floodplains, each having their own definitions of terms relevant to floodplain analysis. What follows is **NDOT's** synopsis of various terms, and brief explanations regarding the interplay between the agencies' regulatory schemes.

Floodplains are hydrologically important, environmentally sensitive, and ecologically productive areas that perform many natural functions. Flooding occurs naturally along rivers and coastal areas. Flood waters can carry nutrient-rich sediments which contribute to a fertile environment for vegetation. Floodplains are beneficial for wildlife by creating a variety of habitats for fish and other animals. In addition, floodplains are important in providing storage and conveyance for flood water, protection of water quality, and recharge of groundwater.

A **floodplain** is defined by **FEMA** to be *any land area susceptible to be inundated by water from any source* (44 CFR 59.1, 44 CFR §59 General Provisions - Code of Federal Regulations (ecfr.io)). In more general terms, a floodplain has been described as an area of land adjacent to a stream or river or low lying area which experiences flooding during periods of high discharge.

The National Flood Insurance Program (NFIP) (Flood Insurance | FEMA.gov) is a **FEMA** program of flood insurance coverage and floodplain management administered pursuant to CFR Title 44/Part-60/Subpart-B.

FHWA's 23 CFR 650A, **FEMA's** NFIP, and **NeDNR's** minimum standards regulate the one percent annual chance (100 year storm event) floodplain, which is usually mapped as the Special Flood Hazard Areas (SFHAs) on **FEMA's** Flood Insurance Rate Maps (FIRMs) (or, in older (pre-1986) studies, Flood Hazard Boundary Maps (FHBMs)). **NeDNR** also maps the one percent annual chance floodplain on their Work Maps and calls them Flood Awareness Areas (FAAs). FAAs are used if no FIRM maps are available, and if there are previously unmapped areas that are shown as mapped in the FAA which are pending **FEMA** approval.

The **base flood** is *the flood having a one percent chance of being equaled or exceeded in magnitude in any given year*, and the **base floodplain** is *the area that is subject to the base flood* (23 CFR 650.105). **FEMA** defines the base flood elevation (BFE) as the elevation to which floodwater is anticipated to rise during the base flood. The BFEs are shown on FIRMs and on the flood profiles.

On both FIRMs and the older FHBMs, FEMA designates SFHAs without base flood elevations using the term "Zone A.". In FIRMs, FEMA designates SFHAs with associated base flood elevations or depths as either Zone AE, A zones, A1-30, AH, and AO. FIRMs designate a shaded "Zone X" as the area, including the base floodplain, which is subject to inundation from a flood having a 0.2 percent chance of being equaled or exceeded in any given year (some older FIRMs used the term "Zone B" to reflect this "Zone X" area). **FEMA** also occasionally designates areas of one percent annual chance flood with average depths less than one-foot or with drainage areas less than one square mile as Zone X floodplains as well. A summary of the types of Zones that may be found on the FIRMS is shown in the definitions.

Development in floodplains, typically Zone A, AH, and AO, is typically allowed as long as demonstrated that the cumulative effect of the proposed development, when combined with all other existing and anticipated new construction or substantial improvement, will not increase the water surface elevation of the base flood more than one-foot at any location along the watercourse. For more specific information, see 44 CFR 60.3(d)(2) (44 CFR §60 Criteria For Land Management And Use - Code of Federal Regulations (ecfr.io)). **FHWA's** floodplain encroachment policy requires longitudinal encroachments to be avoided where practicable. Generally, any increase in the 100-year water-surface elevation produced by a longitudinal encroachment on an NFIP floodplain should not exceed the one-foot allowed by the Federal NFIP standards. Transverse encroachments shall be supported by analysis of design alternatives. Design alternatives should include consideration of a design that is consistent with the Federal NFIP standard, which allows a one-foot rise in the 100-year water surface elevation. Additional guidance can be found in **FHWA's** "Guidance for Implementing the One-foot Standard for Encroachments on NFIP Floodplains".

Most Zone AE's also include a **regulatory floodway**. The regulatory floodway is the *portion of the floodplain that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height* (23 CFR 650.105(m)). The floodway limits are determined through a hydrologic and hydraulic engineering analysis (H&H) that applies encroachments in the flood plain to a surcharge of no more than one-foot per **FEMA** regulations (44 CFR 60.3(d)(3); 23 CFR 650.111). An **encroachment** *is an action within the limits of the base floodplain* (23 CFR 650.105(e)). Typically, the floodway is the most hazardous portion of the floodplain where the fastest flow of water occurs. Development in the regulatory floodway is only allowed if it is demonstrated that no rise in the base flood elevation will occur anywhere along the base flood profile. However, an Memorandum of Understanding (MOU) exists between **FHWA** and **FEMA** which allows an exception for piers, as having "a very minor effect on the floodway water surface elevation". See MOU, *Procedures for Coordinating Highway Encroachments on Floodplains with FEMA* and *Additional Guidance on 23 CFR 650A* (Attachment 2 - Additional Guidance on 23 CFR 650A - Hydrology & Floodplains - Hydraulics - Bridges & Structures - Federal Highway Administration (dot.gov)).

The **flood fringe** is *the area within the SFHA, outside of the floodway, that usually contains slow-moving or standing water during a base flood event* (**FEMA**, *Managing Floodplain Development through NFIP)*. Because the floodway has been calculated to pass the base flood, NFIP minimum standards allow development in the flood fringe without the need for further assessment of the impact on flood heights, unless it is required by local regulation. Development in the flood fringe must also be in compliance with the State Minimum Standards (Nebraska Administrative Code (NAC) Title 455, Chapter 1, Microsoft Word - MinStds-Title_455 (nebraska.gov)) or more restrictive local standards, and must be permitted by local floodplain administrators. These approved local floodplain management regulations may contain more restrictive standards that can take precedence over the NFIP minimum criteria for NFIP administration. 44 CFR 60.1(d).

For federal-aid projects, **FHWA** regulations apply to all base floodplains including **FEMA** regulated floodplains. The **FEMA** FIRM maps and **NeDNR** work maps are used to determine if any portion of a proposed project is within the SFHA and floodway, and any action within the SFHA is subject to the provisions of local, state, and federal floodplain management regulations. Please see Section A.2.4 for guidance regarding circumstances when irreconcilable conflicts arise between **NDOT** and **NeDNR** or local floodplain management agencies regarding the application of a state or local floodplain standard to a federal-aid highway project.

See **Figure 1.1** for a floodplain schematic.

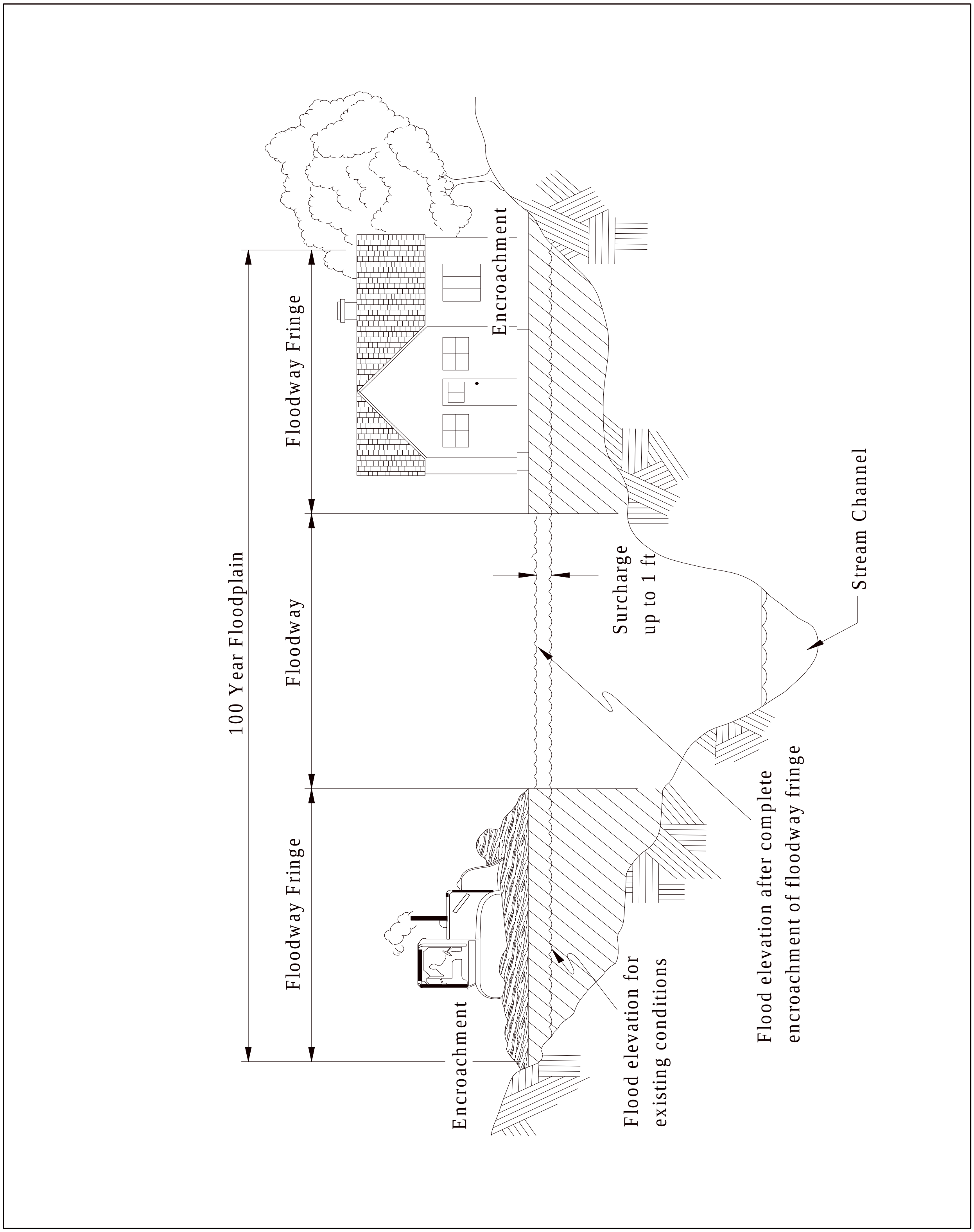

Figure 1.1 100-Year Floodplain Schematic

B.1 Statutes and Regulations

Each federal agency is required by Executive Order (EO) 11988 (Executive Order 11988 | FEMA.gov) to take action to reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare, and to restore and preserve the natural and beneficial values served by floodplains in carrying out its responsibilities.

B.1.1. Executive Order 11988 Summary

EO 11988 requires federal agencies to avoid to the extent possible the long and short-term adverse impacts associated with the occupancy and modification of floodplains and to avoid direct and indirect support of floodplain development wherever there is a practicable alternative. In accomplishing this objective, "each agency shall provide leadership and shall take action to reduce the risk of flood loss, to minimize the impact of floods on human safety, health, and welfare, and to restore and preserve the natural and beneficial values served by flood plains in carrying out its responsibilities" for the following actions:

- acquiring, managing, and disposing of federal lands and facilities
- providing federally-undertaken, financed, or assisted construction and improvements
- conducting federal activities and programs affecting land use, including but not limited to water and related land resources planning, regulation, and licensing activities

This mandate applies to all federally-approved actions, including but not limited to highway construction, reconstruction, rehabilitation, repair, or improvement projects.

B.1.2. FHWA Implementation of Executive Order 11988

The implementation of EO 11988 in transportation projects is addressed by 23 CFR 650 Subpart A, entitled “Location and Hydraulic Design of Encroachment on Floodplains.” (23 CFR 650A). **FHWA** floodplain regulations pertain to planning, NEPA documentation, design, construction, and other aspects of program and project delivery on **NDOT** and local federally funded transportation projects.

B.1.3. Location Hydraulic Studies (23 CFR 650.111)

For federally funded or administered actions, 23 CFR 650.111 provides for the completion of a location hydraulics study, including analysis and the discussion of the practicability of alternatives to longitudinal encroachments.

NFIP maps (https://www.fema.gov/flood-insurance), or information developed by the highway agency if NFIP maps are not available, are used to determine whether a highway location alternative will include an encroachment. In addition, local, state, and federal water resources and floodplain management agencies may be consulted to determine if the proposed highway action is consistent with existing watershed and floodplain management programs and to obtain current information on development and proposed actions in the affected watersheds.

A longitudinal encroachment is an action within the limits of the base floodplain that is parallel to the direction of flow and floodwaters are being conveyed parallel to the highway (e.g. a highway that runs alongside a river or stream and the highway fill slope extends laterally into the floodplain). Longitudinal encroachment should be avoided where practicable, where encroachments are parallel or nearly parallel to the base flood elevation (23 CFR 650.103(b)). However, from a hydraulics perspective, the term parallel/longitudinal encroachment is generally not applied to highway crossings of floodplains where the depth of flow is governed by the crossing structure, typically a culvert or bridge, which is known as a transverse encroachment.

On federal-aid projects involving an alternative alignment, location studies will include evaluation and discussion of the practicability of alternative to any longitudinal encroachments. The study will evaluate alternative locations that would result in lesser impacts to the floodplain or reduce or eliminate a longitudinal encroachment.

For all alternatives containing encroachments, the location hydraulic study must also include a discussion, *commensurate with the risk or environmental impact*, of:

23 CFR 650.111(c)

1. *The risks associated with implementation of the action,*
2. *The impacts on natural and beneficial flood-plain values,*
3. *The support of probable incompatible flood-plain development,*
4. *The measures to minimize floodplain impacts associated with the action, and*
5. *The measures to restore and preserve the natural and beneficial floodplain values impacted by the action.*

23 CFR 650.111(c)

FHWA definitions for the terms "Action", "Encroachment", "Natural and Beneficial Floodplain Values", "Risk" and other relevant terms are provided in 23 CFR 650.105. As defined in 23 CFR 650.105(q), *a **"significant encroachment"** shall mean a highway encroachment and any direct support of likely base flood-plain development that would involve one or more of the following construction-or flood-related impacts*:

(1) A significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route.
(2) A significant risk, or
(3) A significant adverse impact on natural and beneficial flood-plain values.

Location hydraulic studies will also include discussion of the practicability of alternatives to significant encroachments or support of incompatible floodplain development (23 CFR 650.111(d)). Additional guidance is available for reference in the *Significant Encroachment Minute Memo*. Significant encroachments are addressed in more detail in Section C.1.6 below.

Location Hydraulics Studies are documented by a PQS Floodplain Memo, which is also included in environmental review documents prepared pursuant to 23 CFR Part 771 (23 CFR §771 Environmental Impact And Related Procedures - Code of Federal Regulations (ecfr.io)). The PQS Floodplain Memo is described in more detail in Section A.3 below.

B.1.4. National Flood Insurance Program (NFIP) Regulations

Congress established the NFIP as part of the National Flood Insurance Act of 1968 (42 USC 4001, 42 U.S.C. § 4001 - U.S. Code Title 42. The Public Health and Welfare § 4001 | FindLaw). The NFIP was established to designate SFHAs to help reduce flood losses and disaster relief costs by guiding development away from the SFHAs. It also provided flood insurance to those living in the SFHAs, and required communities to implement flood regulations. Since then it has been refined by congress several times:

- Flood Insurance Protection Act of 1973
- National Flood Insurance Reform Act of 1994
- Bunning-Bereuter-Blumenauer Flood Insurance Reform Act of 2004
- Biggert-Waters Flood Insurance Reform Act of 2012
- Homeowners Flood Insurance Affordability Act (HFIAA) of 2014

National Flood Insurance Program Regulations are published in Title 44 of the Code of Federal Regulations.

As noted above, **NDOT** must comply with 23 CFR 650A on state and local highway projects that utilize federal-aid funds. These include designing encroachments in a manner consistent with standards established by **FEMA**, **State**, and **Local** governmental agencies for the administration of the NFIP (23 CFR 650.103(g)). The NFIP requires that all development undertaken by **NDOT** (federally funded or state funded) in a federally identified SFHA within a given jurisdiction (county or municipality) comply with the respective locally-adopted floodplain regulations (44 CFR 60.12). **NDOT** is required to meet federal minimum NFIP standards if local floodplain standards do not exist. Local floodplain management regulations are reviewed by both **NeDNR** and **FEMA** to ensure they meet State and Federal requirements before being adopted. These approved local floodplain management regulations may contain more restrictive standards that can take precedence over the NFIP minimum criteria for NFIP administration (44 CFR 60.1(d)). **NDOT** will coordinate with **FHWA Nebraska Division** in situations where there is an irreconcilable conflict between a **NDOT** and **NeDNR** or **Local** floodplain management agencies regarding the application of a local floodplain standard to a federal-aid highway project.

The **State of Nebraska** has communities that participate in the NFIP, as well as non-participating communities. **Nebraska** also has mapped and unmapped communities. A "participating" community means the community participates in the NFIP program and has completed the application, adopted a resolution of intent to participate and cooperate with **FEMA**, and has adopted and submitted a floodplain management ordinance that meets or exceeds the minimum NFIP and State criteria. Within participating communities, the Federal government makes flood insurance available throughout the community. Communities that participate in the NFIP can be found in the Community Status Book, which can be found on **FEMA's** website (Home | FEMA.gov).

A "mapped" area means that **FEMA** has published a flood map for the area, whereas "unmapped" areas do not have a **FEMA** published flood map. Currently, Nebraska has 16 counties that are unmapped by **FEMA**. **NeDNR's** online floodplain interactive map (https://dnr.nebraska.gov/floodplain/interactive-maps) includes **FEMA** and **NeDNR** mapped locations (paper and digital), it also includes the incorporation of Letter of Map Revisions (LOMR).

B.1.5. State of Nebraska Floodplain Statutes and Regulations

The Nebraska Legislature passed the Nebraska Floodplain Regulations Act in 1967 (updated in 1983 and 1993) which can be found in *Nebraska Revised Statutes*, Chapter 31,Sections 1001 to 1023 (as amended) (Nebraska Legislature - Revised Statutes Chapter 31). In response to statutory directive, **NeDNR** implemented the Nebraska Minimum Standards for Floodplain Management Program (the State Minimum Standards), found at NAC Title 455, Chapter 1. **NeDNR** looks to **FEMA** for its primary regulatory interpretations and guidance (44 CFR 60.1(d)) in addition to requirements provided by State law.

In the State of Nebraska, if local regulations are not in place, any development on state-owned land, by a state agency, or being state-financed must also meet the State Minimum Standards adopted and implemented by **NeDNR**. **NDOT** should also be mindful of the potential for additional restrictions that may be set forth in locally adopted floodplain regulations.

The purpose of the “Nebraska Floodplain Regulations Act” is set forth in Nebraska Revised Statutes, Section 31-1001:

> *“(1)The Legislature finds that recurrent flooding in various areas of the state presents serious hazards to the health, safety, welfare, and property of the people of the state, both within and outside such areas. The hazards include loss of life, loss of and damage to private and public property, disruption of lives and of livelihoods, interruption of commerce, transportation, communication, and governmental services, and unsanitary and unhealthy living and environmental conditions. The wise use of land subject to flooding is a matter of state concern. The Legislature further finds that the establishment of improved floodplain management practices and the availability of financial assistance to citizens of the state whose property is damaged during times of flooding are essential to the health, safety, and general welfare of the people of Nebraska*”

NeDNR’s duties as described by statute include mapping floodplains, providing technical assistance, and coordinating the NFIP at the state level. More specifically, Nebraska Revised Statutes Section 31-1017 authorizes **NeDNR** to:

- *Coordinate floodplain management activities of local, state, and federal agencies;*
- *Receive federal funds intended to accomplish flood plain management objectives;*
- *Prepare and distribute information and conduct educational activities which will aid the public and local units of government in complying with the purposes of sections 31-1001 to 31-1023*
- *Provide local governments having jurisdiction over flood-prone lands with technical data and maps adequate to develop or support reasonable flood plain management regulation;*
- *Adopt and promulgate rules and regulations establishing minimum standards for local flood plain management regulation. In addition to the public notice requirement in the Administrative Procedure Act, the department shall, at least twenty days in advance, notify by mail the clerks of all cities, villages, and counties which might be affected of any hearing to consider the adoption, amendment, or repeal of such minimum standards. Such minimum standards shall be designed to protect human life, health, and property and to preserve the capacity of the flood plain to discharge the waters of the base flood and shall take into consideration (a) the danger to life and property by water which may be backed up or diverted by proposed obstructions and land uses, (b) the danger that proposed obstructions or land uses will be swept downstream to the injury of others, (c)*

the availability of alternate locations for proposed obstructions and land uses, (d) the opportunities for construction or alteration of proposed obstructions in such a manner as to lessen the danger, (e) the permanence of proposed obstructions or land uses, (f) the anticipated development in the foreseeable future of areas which may be affected by proposed obstructions or land uses, (g) hardship factors which may result from approval or denial of proposed obstructions or land uses, and (h) such other factors as are in harmony with the purposes of sections 31-1001 to 31-1023. Such minimum standards may, when required by law, distinguish between farm and nonfarm activities and shall provide for anticipated developments and gradations in flood hazards. If deemed necessary by the department to adequately accomplish the purposes of such sections, such standards may be more restrictive than those contained in the national flood insurance program standards, except that the department shall not adopt standards which conflict with those of the national flood insurance program in such a way that compliance with both sets of standards is not possible;

- *Provide local governments and other state and local agencies with technical assistance, engineering assistance, model ordinances, assistance in evaluating permit applications and possible violations of flood plain management regulations, assistance in personnel training, and assistance in monitoring administration and enforcement activities;*
- *Serve as a repository for all known flood data within the state;*
- *Assist federal, state, or local agencies in the planning and implementation of flood plain management activities, such as flood warning systems, land acquisition programs, and relocation programs;*
- *Enter upon any lands and waters in the state for the purpose of making any investigation or survey or as otherwise necessary to carry out the purposes of such sections. Such right of entry shall extend to all employees, surveyors, or other agents of the department in the official performance of their duties, and such persons shall not be liable to prosecution for trespass when performing their official duties;*
- *Enter into contracts or other arrangements with any state or federal agency or person as defined in section 49-801 (Nebraska Legislature - Revised Statutes Chapter 49) as necessary to carry out the purposes of sections 31-1001 to 31-1023; and*
- *Adopt and enforce such rules and regulations as are necessary to carry out the duties and responsibilities of such sections.*

Nebraska Revised Statutes, Sections 31-1019 to 31-1023, describe **NDOT's** responsibilities with regard to floodplains, as well as the interplay between **NeDNR** and local floodplain administrators. Section 31-1023 requires state agencies, boards and commissions to take preventive action to minimize flood hazards and losses in connection with state-owned and state-financed buildings, roads, and other facilities, and to take steps necessary to insure compliance with the Minimum Standards adopted by **NeDNR** when such facilities are being located or constructed in areas where no local government is enforcing floodplain management regulations. When local governments are enforcing floodplain management regulations, **NDOT** complies with the respective locally adopted floodplain regulations, again seeking assistance from **FHWA** in the event of a major disagreement affecting a federally funded transportation project. A list of local floodplain managers can be found on **NeDNR's** website (Welcome | Department of Natural Resources (nebraska.gov)).

As noted above, NAC Title 455, Chapter 1, Section 004 provides minimum standards (State Minimum Standards) governing location of obstructions and substantial improvements in the floodplain of a base flood. NAC, Title 455, Chapter 1, Section 004.01 states that no new construction, substantial improvements, or other obstruction (including fill) shall be permitted unless it is demonstrated that the cumulative effect of the proposed new construction, when combined with all other existing and anticipated new construction or substantial improvement, will not increase the water surface elevation of the base flood more than one-foot at any location.

According to NAC, Title 455, Chapter 1, Section 002.16, an "Obstruction" shall mean any wall, wharf, embankment, levee, dike, pile, abutment, projection, excavation (including the alteration or relocation of a watercourse or drainway), channel rectification, bridge, conduit, culvert, building, stored equipment or material, wire, fence, rock, gravel, refuse, fill, or other analogous structure or matter which may impede, retard, or change the direction of the flow of water, either in itself or by catching or collecting debris carried by such water, or that is placed where the natural flow of the water would carry such structure or matter downstream to the damage or detriment of either life or property.

NAC, Title 455, Chapter 1, Section 005 provides State Minimum Standards governing location of obstructions or substantial improvements in the floodway. NAC, Title 455, Chapter 1, Section 005.01 states that no new construction, substantial improvements, or other obstruction (including fill) shall be permitted within the floodway unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice that the proposed new construction would not result in any increase in the water surface elevations along the floodway profile during occurrence of the base flood.

B.1.6. Floodplains and Clean Water Act Permitting

Undeveloped floodplain land provides many natural resources and functions of considerable economic, social and environmental value. Floodplains often contain wetlands and other important ecological areas as part of a total functioning system that benefits the quality of the local environment. The movement of water through ground and surface systems, floodplains, wetland, and watersheds is perhaps the greatest indicator of the interaction of natural processes in the environment.

When jurisdictional waterways will be affected by construction activities, the **Technical Resources Unit** (**TRU**) in the **Environmental Section** of the **Project Development Division** is consulted to determine **US Army Corps of Engineers** (**USACE**) permitting requirements. Floodplain impacts are also reviewed for projects that require **USACE** permit to meet Section 404 (Overview of Clean Water Act Section 404 | Section 404 of the Clean Water Act: Permitting Discharges of Dredge or Fill Material | US EPA) or Section 401 (The Clean Water Act Section 401 Certification Rule | Overview of Certification under Section 401 of the Clean Water Act | US EPA) requirements, as the **USACE** is also required to consider EO 11988 as part of its public interest review when an application is received requesting authorization to impact waters of the U.S. that also has the potential to increase the BFE.

For projects requiring Section 401, 404 or 408 (Section 408 (army.mil)) approvals, the Roadway Designer should contact **NDOT's TRU** for obtaining necessary permits and approvals.

A Section 9 bridge permit (US Section 9 Coast Guard Bridge Permit | Agency of Transportation (vermont.gov)) can be required for new bridges over commercially navigable waters, but these types of permits are uncommon. In Nebraska, the only navigable water identified is the Missouri River. In these cases, the **U.S. Coast Guard** regulates Section 9 and would also require completion of floodplain coordination prior to issuing a permit. Again, contact **NDOT's TRU** for assistance with Section 9 permits.

C.1. Floodplain Analysis/PQS Floodplain Memo (23 CFR 650.111)

When the proposed action for a federally funded transportation improvement project lies within a base floodplain, a PQS Floodplain Memo is prepared, which includes a location hydraulic study in accordance with 23 CFR 650.111. The 650.111 minimum documentation requirements, including the review requirements for NEPA found at 23 CFR 771, are met by the preparation of a PQS Floodplain Memo (or Floodplain Memo).

The PQS Floodplain Memo (Exhibit L of the Design Process Outline, (DPO) https://dot.nebraska.gov/business-center/design-consultant/) addresses each of the criteria set forth in 23 CFR 650.111 for Location Hydraulic Studies (see Sections C.1.1 – C.1.6). In addition, local, state, and federal floodplain management agencies may be consulted to determine if the proposed highway action is consistent with existing watershed and floodplain management programs and to obtain current information on development and proposed actions in the affected watershed.

The PQS Floodplain Memo is prepared by designers/engineers based on site specific information. It is reviewed by the **Roadway Design/Local Project Unit Head**, and approved by **NDOT PQS**. The **PQS** must be a registered Professional Engineer with hydraulic expertise. Exhibit L in the DPO provides additional information.

The PQS Floodplain Memo is submitted to the **Environmental Section** (**NEPA Specialist**), who incorporates the information into the environmental review documentation in accordance with 23 CFR part 771 (23 CFR §771 Environmental Impact And Related Procedures - Code of Federal Regulations (ecfr.io)). In addition, the **Public Involvement Unit** in the **Communications Division** is notified when a project has been identified as having adverse impacts to floodplains, defined as the anticipation of either a rise greater than one-foot in the base flood elevation or a LOMR. The PQS Floodplain Memo documenting the 23 CFR 650.111 Location Hydraulic Study provides the minimum documentation necessary regarding analysis of floodplain encroachment impacts, and is also utilized as part of the Plan-in-Hand process.

C.1.1. Documentation of Floodplain and/or Floodway Encroachments

The first section of the Floodplain Memo documents whether the project has an encroachment. Seven different scenarios may occur. The designer/engineer identifies which scenario applies for each project. The seven scenarios are as follows:

- Project is located within a Mapped and Participating Community and crosses or overlaps upon Base Floodplains
- Project is located in a Mapped and Participating Community and crosses or overlaps upon Base Floodplains and Regulatory Floodways
- Project is located in a Mapped and Participating Community and does not overlap upon any Base Floodplain or Regulatory Floodway
- Project is located in a Mapped but Non-Participating Community and crosses or overlaps upon Base Floodplains
- Project is located in a Non-Mapped and Non-Participating Community and crosses or overlaps upon Potential Base Floodplains
- Project is located in a Non-Mapped and Non-Participating Community and does not overlap upon Potential Base Floodplain
- Project entails scope that does not meet the criteria for Development – See the **FEMA** definitions for definition of Development

C.1.2. Documentation of Floodplain and/or Floodway Impacts

For each project, the Floodplain Memo also documents whether the project will cause a rise in the BFE greater than one-foot, an increase in the potential for property loss and hazard to life, or any rise in a regulatory floodway. Based on the scope of the project, three determinations may be made. They are as follows:

- Based on the project scope, no floodplain certification or permit will be required.
- Based on the project scope, the project will be certified to meet floodplain regulations. It is not anticipated to cause greater than one-foot of rise in the BFE within a Base Floodplain, increase the potential for property loss and hazard to life, or any rise in the BFE within a Regulatory Floodway. or
- Based on the project scope, it is anticipated that the project will require a conditional letter of map revision (CLOMR) and a letter of map revision (LOMR) following construction and will require further coordination with **FEMA**. Notify the **NDOT Public Involvement Unit**.

Projects may require the issuance of a LOMR or CLOMR. The project **PQS** with hydraulic expertise will provide more guidance if/when that process is required. Coordination with **FEMA** must occur when:

- Floodplain studies indicate that a proposed encroachment on a regulatory floodway (BFE increase) would require an amendment to the floodway map (however, it is not likely that any increase would be allowed by **FEMA**), or
- A proposed encroachment on a floodplain where a detailed study has been performed but no floodway designated and the maximum one-foot increase in the base flood elevation would be exceeded.

Map revisions are a complex process. It is recommended that **NeDNR** and/or **FEMA** be contacted for technical assistance at the outset and throughout this process. A basic outline of the information needed is listed below:

- Flood Insurance Study (FIS) backup data (hydrology, hydraulics and mapping) from **FEMA** for pre-project hydraulic model.
- Pre-project survey of existing cross-sections of the stream or watercourse at the proposed project site.
- Design must meet local, state, and **FEMA** criteria and all other permit requirements.
- Original FIS models (HEC-2, HEC-RAS or WSPRO, etc.) must be rerun with new data to reflect the new base floodplain boundaries.

Map revision data (LOMR) must be submitted to **FEMA** within 6 months of project completion. However, the coordination/consultation for the revisions should begin during the design phase of a project (CLOMR).

C.1.3. Documentation of floodplain encroachments other than functionally dependent use

The project designer/engineer will determine if the project has a base floodplain that overlaps the project at locations other than culverts and/or bridges. It will be determined whether the project scope results in a floodplain encroachment other than functionally dependent uses.

For transportation projects, functionally dependent use has been described as bridges, or any water conveyance structures or actions that facilitate the use of open space use (e.g. recreational trails, bicycle and pedestrian paths). Functionally dependent uses also include embankment, culverts, grading and guardrails, and other associated appurtenances or required work to support or protect a bridge or culvert.

If there are no base floodplains that overlap the project, then documentation will reflect that there are no base floodplain encroachments.

If there are no locations along the project that possibly or potentially overlap a floodplain outside of culverts, bridges, and adjacent embankment or other activities listed above, and all overlapping areas are located at culverts or bridges and are considered a functionally dependent use of the floodplain, then documentation will reflect that there are no floodplain encroachments other than functionally dependent use.

If there are locations along the project that overlap a floodplain and are not considered functionally dependent, then it will be documented that there are floodplain encroachments other than functionally dependent use. Source information will also be documented and will contain the Panel Number and the Effective Date of the map (for example, FIRM, FHBM). If a digital work map from **NeDNR** was used, this will also be documented.

C.1.4. Documentation of evaluation and discussion of practicability of alternatives to any longitudinal encroachments (23 CFR 650.111(b))

The designer/engineer evaluates and discusses the practicability of alternatives to any longitudinal encroachments. For all projects, the longitudinal encroachment will be described, and consideration will be given to whether there are alternatives to the encroachment. For example, a project on existing alignment or maintenance project that typically has an asset preservation scope, there would be no alternative that would have less impact on the longitudinal encroachment.

If there are no longitudinal (parallel) encroachments located along the project, there are no longitudinal encroachments to evaluate or discuss.

For reconstruction or new construction on existing alignment, language about replacement on alignment will be included. For reconstruction or new construction on any portion of the project on new alignment, a discussion regarding the alternative analysis and selection of the alignment will be included.

C.1.5. Documentation of the discussion of risks associated with implementation of the action, the impacts on natural and beneficial flood-plain values, the support of probable incompatible flood-plain development, the measures to minimize flood-plain impacts associated with the action, and the measures to restore and preserve the natural and beneficial flood-plain values impacted by the action for all alternatives containing encroachments and for those actions which would support base floodplain development (23 CFR 650.111 (c))

The designer/engineer will include discussion of the following items, commensurate with the significance of the risk or environmental impact, for all alternatives containing encroachments and for those actions which would support base flood-plain development (from 23 CFR 650.111(c)):

(1) The risks associated with implementation of the action,
(2) The impacts on natural and beneficial flood-plain values,
(3) The support of probable incompatible flood-plain development,
(4) The measures to minimize flood-plain impacts associated with the action, and
(5) The measures to restore and preserve the natural and beneficial flood-plain values impacted by the action.

Risk is defined as "*the consequences associated with the probability of flooding attributable to an encroachment. It shall include the potential for property loss and hazard to life during the service life of the highway*." (23 CFR 650.105(o)). Therefore, from the reverse perspective, if a project will not, or will minimally increase the potential for loss of life or property, and will not or will minimally increase the consequences associated with the probability of flooding attributable to the encroachment, it would not be considered a significant risk. Please see C.1.6 for additional information.

Natural and beneficial floodplain values are defined in 23 CFR 650.105(i), and include, but are not limited to, *fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.* When analyzing the significance of the risk or environmental impacts on natural and beneficial floodplain values, discussion should occur with relevant subject matter experts from the **NDOT Environmental Section**. The Floodplain Memo documents the discussion of risk or environmental impacts to natural and beneficial floodplain values *commensurate with the significance of the risk or environmental impact* (23 CFR 650.111(c)).

It is the policy of **FHWA** to avoid support of incompatible floodplain development (23 CFR 650.103(f)). Support of base floodplain development means to encourage, allow or otherwise facilitate additional base floodplain development. Direct support results from an encroachment, while indirect support results from an action out of the base floodplain, (23 CFR 650.105(r)). As such, the Floodplain Memo documents whether the proposed improvements for a project will maintain local and regional access to existing rural and agricultural areas, or whether it will create new access to undeveloped lands.

The Floodplain Memo documents what *measures were taken to minimize flood-plain impacts associated with the action, commensurate with the significance of the risk or environmental impact, for all alternatives containing encroachments and for those actions which would support base flood-plain development* (from 23 CFR 650.111(c)).

The Floodplain Memo documents what measures were taken to restore the natural and beneficial flood-plain values along the project. For example, for projects that have temporary soil disturbance activities during construction, sediment and erosion control best management practices will be utilized during construction and disturbed areas will be seeded following construction.

C.1.6 Documentation of an evaluation and discussion of the practicability of alternatives to any significant encroachments or any support of incompatible flood-plain development. (23 CFR 650.111(d))

As defined in 23 CFR 650.105, a *"significant encroachment" shall mean a highway encroachment and any direct support of likely base floodplain development that would involve one or more of the following construction-or flood-related impacts:*

(1) A significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route.
(2) A significant risk. or
(3) A significant adverse impact on natural and beneficial flood-plain values.

Documentation will reflect if a project results in a significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or a community's only evacuation route. Documentation will also reflect whether the project scope results in a significant risk of increase in potential for loss of life or property, a substantial adverse impact on natural and beneficial flood-plain values, and whether the project supports any incompatible floodplain development.

According to 23 CFR 650.113, *a proposed action which includes a significant encroachment shall not be approved unless FHWA finds that the proposed significant encroachment is the "only practicable alternative". This finding shall be included in the final environmental document (final environmental impact statement or finding of no significant impact) and shall be supported by the following information:*

(1) The reasons why the proposed action must be located in the flood plain,
(2) The alternatives considered and why they were not practicable, and
(3) A statement indicating whether the action conforms to applicable State or local flood-plain protection standards.

D. Design Standards (23 CFR 650.115)

NDOT follows general design standards consistent with State and Federal law. Design standards established by 23 CFR 650.115 are as follows:

(a) The design selected for an encroachment shall be supported by analyses of design alternatives with consideration given to capital costs and risks, and to other economic, engineering, social and environmental concerns.
  (1) Consideration of capital costs and risks shall include, as appropriate, a risk analysis or assessment which includes:
    i. The overtopping flood or the base flood, whichever is greater, or
    ii. The greatest flood which must flow through the highway drainage structure(s), where overtopping is not practicable. The greatest flood used in the analysis is subject to state-of-the-art capability to estimate the exceedance probability.

(2) The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2 percent chance[1] of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways.
(3) Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failure.
(4) The effect of existing flood control channels, levees, and reservoirs shall be considered in estimating the peak discharge and stage for all floods considered in the design.
(5) The design of encroachments shall be consistent with standards established by the FEMA, State, and local governmental agencies for the administration of the National Flood Insurance Program for:
i. All direct Federal highway actions, unless the standards are demonstrably inappropriate, and
ii. Federal-aid highway actions where a regulatory floodway has been designated or where studies are underway to establish a regulatory floodway.

(b) Rest area buildings and related water supply and waste treatment facilities shall be located outside the base flood plain, where practicable. Rest area buildings which are located in the base flood plain shall be floodproofed against damage from the base flood.

(c) *Where highway fills are to be used as dams to permanently impound water more than 50 acre-feet[2] in volume or 25 feet deep[2], the hydrologic, hydraulic, and structural design of the fill and appurtenant spillways shall have the approval of the State or Federal agency responsible for the safety of dams or like structures within the State, prior to authorization by the Division Administrator to advertise for bids for construction.*

Although the minimum design for Interstate highway ramps and frontage roads or for other highways is not given in 23 CFR 650.115, design guidance can be found in EXHIBIT 1.3 in Chapter One: Drainage of **NDOT's** Drainage Design and Erosion Control Manual (http://dot.nebraska.gov/business-center/design-consultant/rd-manuals/). Also, see 23 CFR 650.117 for the content of design studies. *Design Standards for Highways in National Flood Insurance Program Mapped Floodplains* is also available as guidance from **FHWA**.

E. Floodplain Considerations during the Phases of an NDOT Project

During the initial project programming and scoping of an **NDOT** Project (Program Phase), Project Development determines the presence of a floodplain and/or floodway within the project area. Floodplain identification is accomplished by referencing the **NeDNR** floodplain interactive map and/or the **FEMA** Flood Map Service Center (FEMA Flood Map Service Center | Welcome!); these sources provide the best available current information since they are updated as necessary by those agencies. **Project Development** documents the findings within the NDOT Form 73 – Highway Improvement Programming Request and Scoping Report.

Following the project scoping phase, the project enters the Planning Phase, where it becomes the responsibility of the Roadway Designer/Engineer to determine whether the extent of the project footprint results in an encroachment on a floodplain or floodway. At the end of the Planning Phase, the presence of floodplains/floodways is documented within the meeting minutes of the initial Project Coordination Meeting (PCM). For more information regarding Project Coordination Meetings, see Exhibit A of the DPO.

[1] 2 percent chance is often referred to as a 50-year design storm
[2] Metrics removed

Following the Planning Phase, is the (preliminary) Design Phase. The PQS Floodplain Memo will be completed as part of the Plan-in-Hand process during this phase. As noted above, the PQS Floodplain Memo is included with the environmental documentation for the next project phase, the Environmental Approval Phase.

The Plan Details phase follows the Environmental Approval Phase, and it is here that the floodplain certification is completed by **Hydraulics Staff** for submittal to the local floodplain administrators for the floodplain development permit. (See Section F regarding floodplain certification)

When a project is located in a floodplain, it is the responsibility of the designer and their **Unit Head** to be aware of the restrictions on increases to the base flood elevation. Changes in culvert pipe sizes, culvert flow lines, embankment fill, channel shape, bridges, and pavement elevation increase can cause increases in the base flood elevation of a floodplain. Designers should also be mindful of the fact that local governments occasionally have more restrictive requirements for development in a floodplain that may need to be taken into consideration during design.

The **NDOT Hydraulics Staff** will work to support and provide training to Roadway Designers on understanding the program, use of resources for identifying floodplains and floodways, and identifying encroachments.

F. NDOT Floodplain Certification Process

Projects which may affect base flood elevations are investigated, analyzed, and certified to meet floodplain regulations prior to requesting a permit from the local floodplain administrator. Once the final limits of construction have been determined, the limits are compared to the limits of the floodplain/floodway to start the certification process.

Upon identification of floodplain/floodway encroachment by a project, **Roadway Design Hydraulics** and **Bridge Hydraulics** engage in additional evaluation and analysis of impacts, and ultimately certify that the improvement meets requirements for work within a floodplain/floodway. Projects being accomplished under a consultant contract require the consultant to follow the same process as internal projects when evaluating floodplain/floodway impacts and providing certification. Certifications completed by consultants are reviewed by **NDOT PQS/Hydraulics Staff**, and documentation is placed in the project file.

Projects with construction occurring in a floodplain, whether crossing or parallel to that floodplain, require **NDOT** certify that:

- Where construction occurs in base floodplains, it does not increase, cumulatively, the floodplain base flood elevation more than one-foot (1’), and
- Where construction occurs in regulatory floodways, there is no increase to the base flood elevation.

The certification will document that the project meets State regulations, and causes neither an increase in the floodway nor an increase in the base flood elevation by more than one-foot in a floodplain. In the case of a community with stricter standards, **NDOT** will certify that the project meets those stricter local regulations.

Certification are provided by **PQS Hydraulics Staff**. Designers (or their **Unit Head**) should request a certification once limits of construction and final design have been completed. This occurs prior to the submittal of the Roadway Design Details, Clarity task 5508. The responsibility for completing certification of compliance with floodplain regulations rests with:

- **Bridge Hydraulics** for structures (bridge or box culvert), with at least a 20-foot span measured along the centerline of the road, or
- **Roadway Design Hydraulics** for all other structures (bridge, box culvert or pipe culvert(s)) with less than a 20-foot span measured along the centerline of the road, encroachments into a floodplain by the highway embankment, and other obstructions.
- **Consultants**, if they meet PQS requirements as a registered Professional Engineer with hydraulic expertise.

NDOT produces a project memo for the local floodplain administrator that describes the project location/bounds (including Section, Township, and Range), work being done, a short paragraph explaining impacts for each floodplain encroachment and the work's effect on the BFE or water surface elevations, and a statement that the work meets the floodplain requirements. A certification for each floodplain encroachment (with engineer's stamp and signature) is provided, as well as a set of floodplain maps showing encroachment locations and a general location map. **NDOT** retains and stores H&H analysis with the design files and submits the memo, certifications, and mapping to the **NDOT TRU** for inclusion in the Floodplain Development Permit Application. The technical data, if available, will be provided to local floodplain administrators upon request.

G. Certification Guidelines

The level of analysis required for floodplain certifications can differ. Below are several scenarios that **NDOT** commonly encounters.

G.1. Case 1. No Changes to Roadway Elevations or Culverts

On cases where the project will result in no changes to the roadway elevation and no culvert replacements or extensions occur, floodplain documentation typically involves a "Letter of No Impact" to the local floodplain administrator(s). These types of projects include, but are not limited to, mill and fill projects with no grade raise and no grading outside of the shoulder hinge point, concrete pavement repair, armor coat, fog seal, microsurfacing, diamond grinding, bridge painting, applying surface sealers, joint sealing, curb ramps, etc.

These types of work fall within the category of "minor projects" described in **FEMA's** "Managing Floodplain Development through NFIP" guidance document, which states that some projects are too small to warrant an engineering study and the encroachment certification. A determination has been made that these projects will not block flood flows.

The "Letter of No Impact" describes the project location and work involved. The letter further states that the project will have no impact on the floodplain/floodway and **NDOT** does not believe a floodplain permit is required; no H&H analysis is required. If the local administrator so chooses, she/he may contact **NDOT** and request a permit application, and in that situation **NDOT** will provide permitting information as requested. See the Appendix for discussion on crack-filling and striping projects, which are activities that fall outside of the definition of development as defined by NFIP.

G.2 Case 2. Changes to Roadway Elevation, Bridges, or Culverts in Floodplains/Floodways

In cases where the project will result in changes to the roadway elevation or embankments (e.g. an overlay with a grade raise or a widening project involving changes to the ditch or backslope) and/or culvert replacements or extensions, the effort required to evaluate the impacts and for certification increases. For these projects, the **NDOT Roadway Design Hydraulics Unit** completes an investigation and H&H evaluation (see description below) for each non-bridge size structure and for roadway embankment floodplain impacts. Bridge size structures are typically analyzed and certified by **NDOT's Bridge Hydraulics Section**. The H&H analysis, when a hydraulic model is required, focuses on the 100-year event and, for Zone A, compares the pre-project conditions to the post-project conditions; for Zone AE, the comparison is between the current NFIP BFE vs. the post project BFE. The profile of the base flood elevations is reviewed upstream and downstream of the highway.

The investigation also includes checking for risks to improved properties, even if the analysis shows the project meets floodplain regulations. When the project work increases the BFE(s) over the acceptable limits or investigation shows risk to improved property, adjustments to the culvert/bridge/roadway design are warranted and coordinated with **Roadway Design** prior to certification.

The H&H analysis will follow standard **NDOT** procedures. If a **FEMA** study exists, then it will be used to determine impacts on the base floodplain and regulated floodway as appropriate. **NDOT** uses the Rational Method to compute peak discharges for drainage areas less than 640 acres or 1 sq. mile and the NRCS curve number method for drainage areas from 1 to 10 sq. miles. Regression equations are used to determine peak discharges for drainage areas greater than 10 sq. miles. For most floodplain certifications, **NDOT** uses basic hydraulic procedures for determining the highway project's impacts on the floodplain encroachment(s) (i.e. HY-8 for the culvert crossings and normal depth Manning's Equation calculations for channel impacts and parallel encroachments). **NDOT** typically reserves the use of HEC-RAS, or other **FEMA**-approved models, for analysis of bridge replacements within the floodplain or for cases where backwater from changes due to the highway project could reasonably impact upstream buildings. Investigation of encroachments includes checking for impacts to improved real estate, even if analysis shows the requirements of the floodplain regulations are met.

For more information regarding H&H calculation methods, please see Chapter One: Drainage in **NDOT's** Drainage Design & Erosion Control Manual and **NDOT's** Bridge Hydraulic Analysis Guidelines (https://dot.nebraska.gov/business-center/bridge/hydraulic/).

G.3 Case 3. No Floodplains Encroached

On projects where no floodplains are encroached, **NDOT** generates a courtesy email to **NDOT** project stakeholders (wetland biologist, NEPA practitioner, for example) stating that no floodplains are encroached upon. The information is also documented in Clarity within the floodplain comments. Official documentation of the absence of floodplain encroachment is captured by the PQS memo, which is kept in the project file.

G.4 Case 4. Non-Mapped Communities

On projects where the community does not have **FEMA** floodplain mapping, **NDOT** certifies floodplains to meet State Minimum Standards. The existence of State Minimum Standards Potential Base Floodplains is determined by analysis of drainage basins using USGS quad maps and aerials. The work effort on State Minimum Standards Potential Base Floodplains, once their existence is established, is similar to the cases stated above in cases 1 and 2.

In currently unmapped communities, **NDOT** will provide floodplain certifications for projects that drain more than 1 square mile (640 acres). Watersheds greater than 1 square mile will be treated as Potential Base floodplains and the project area will then be evaluated based on the requirements listed above for Base floodplains. They will be certified but no permit is requested from these non-participating communities.

G.5 Case 5. Non-Mapped Counties located with Nebraska Sandhills

There are 16 counties in Nebraska that do not have **FEMA** Floodplain mapping and do not participate in the NFIP. There may be communities within these counties that are mapped. For counties that are completely unmapped, floodplain development permits are not required for these counties, however certifications may still be required as explained in more detail below. It is best to verify participating and/or mapped communities by looking it up through **FEMA**.

As a topographic region, this area is characterized by well-watered, grass-stabilized dunes of predominantly sandy soils. The combination of highly permeable soils (which limit surface runoff and enhance infiltration) and thick, permeable, subsurface deposits (which facilitate percolation and recharge the groundwater supply) have infiltration rates a hundred times higher than in the clayey loams of the Platte Basin and other like river basins around the state. Sandhill streams are fed by groundwater and streamflow in the region is predominantly from subsurface percolation instead of surface runoff (Atlas of the Sand Hills, Bleed and Flowerday, 1998). Traditional methods of determining drainage areas by topographic maps and determining peak flows of surface runoff are not applicable in these areas; precipitation in the region rarely results in overland runoff that reaches streams. Instead, it quickly infiltrates porous soils and moves according to subsurface groundwater gradients. Aerial inspection and contour maps also reveal that the typical patterns of sheet flow runoff which coalesces into concentrated overland flow and channels are not present in this area.

Perennial or intermittent streams located within this region are characterized by nearly constant flows that are typical of streams fed almost entirely by groundwater seepage. These streams do not experience the extreme high discharges or low discharges that are typical of streams being fed by surface runoff. Thus floodplain maps that are associated with a present flooding risk have not been completed for these areas and potential Base floodplains cannot be identified.

When a project is located within an unmapped county with unmapped communities, does not involve a stream crossing, and lies within the Sandhills topographic regions, roadway embankment and culvert locations (non-bridge sized structures) will be identified as areas where there are no potential Base floodplains. Therefore no certifications are required for these locations. If there is a stream crossing, see the paragraph below.

For roadway embankments, culverts, and bridges located at stream crossings, the project will be certified based on information available. Methods can include calculations from regression equations, flow gauge data, **NDOT** developed data for Nebraska Counties, flood flow frequency prediction equations developed for the Sandhills region on Nebraska, historical high water data, **NDOT's** Bridge Hydraulic Analysis Guidelines, or any other data available. Hydrology and hydraulics calculations within the Sandhills vary and best engineering judgement will be used to determine 100-year flows at a stream crossing located at a bridge structure.

H. Floodplain Permitting

Projects that meet the definition of development and which encroach within a mapped SFHA as shown on a FIRM will be submitted to the local floodplain administrators for a floodplain development permit. Development is defined by **FEMA** *as any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation or drilling operations or storage of equipment or materials* (44 CFR 59.1). However, according to **FHWA**, *support base flood-plain development shall mean to encourage, allow, serve, or otherwise facilitate additional base flood-plain development. Direct support results from an encroachment, while indirect support results from an action out of the base flood plain* (23 CFR 650.109(r)). Currently, **NDOT** has a letter from **NeDNR** that states that crack sealing and paint striping are activities that fall outside of the definition of development and do not require a Floodplain Development Permit. As noted above, **NDOT** will coordinate with **FHWA Nebraska Division** in situations where there is an irreconcilable conflict between **NDOT** and **NeDNR** or **Local Floodplain Management Agencies** regarding the application of a local floodplain standard to a federal-aid highway project.

Certifications of compliance with floodplain regulations, once completed, will be placed in the project file and forwarded to the **Project Development Technical Resources Unit** (**TRU**) so a floodplain permit can be requested from the regulating community (county or city). Certifications of compliance for projects located in communities not regulating floodplains and based on Minimum State Standards are provided to the **TRU** and placed in the project file to show compliance with environmental regulations.

I. Floodplain Mitigation

According to **FEMA**, flood mitigation is defined as "any sustained action that reduces or eliminates long-term risk to people and property from the effects of floods." Flood mitigation reduces the overall risk of a structure experiencing flood damage, and also reduces the severity of flood damage when it occurs.

There are two types of basic flood mitigation: structural and nonstructural. As the name implies, structural techniques seek to build structures in order to change or "control" the physical environment; thus, common techniques are dams, levees, floodwalls, jetties, or retention ponds. The purpose of non-structural flood mitigation is to change the way that people interact with the floodplain, flood risk, and also aim to move people away from flood-prone areas. Floodplain mitigation outside of engineered structural mitigation are not addressed in this manual.

Structural floodplain mitigation is more likely to occur when projects cannot be avoided within a floodplain and have impacts on lives, property, and natural and beneficial floodplain values. If there are no practicable alternative sites, then **NDOT** must develop measures to minimize the adverse impacts, restore, and preserve the floodplain. The methods used to minimize, restore, and preserve vary in context and intensity depending on the project. **NDOT** coordinates and communicates with **NeDNR** and other state and federal agencies with regard to structural floodplain mitigation strategies.

APPENDIX – ACRONYMS and DEFINITIONS

BFE	Base Flood Elevation
CLOMR	Conditional Letter of Map Revisions
EO	Executive Order
FAA	Flood Awareness Area
FEMA	Federal Emergence Management Agency
FHBM	Federal Hazard Boundary Map
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
H&H	Hydraulic Engineering Analysis
LOMR	Letter of Map Revision
NAC	Nebraska Administrative Code
NDOT	Nebraska Department of Transportation
NeDNR	Nebraska Department of Natural Resources
NEPA	National Environmental Policy Act
NFIP	National Flood Insurance Program
PQS	Professionally Qualified Staff
SFHA	Special Flood Hazard Area
TRU	Technical Resources Unit in the Environmental Section of the NDOT Planning and Project Development Division
USACE	United States Army Corps of Engineers

FHWA - FAPG 23 CFR 650A, Location and Hydraulic Design of Encroachments on Flood Plains Definitions

The following definitions of terms are made for the purpose of uniform application in the documentation and preparation of location hydraulic studies. Refer to 23 CFR 650.105 for a complete list of definitions.

"Action" shall mean any highway construction, reconstruction, rehabilitation, repair, or improvement undertaken with Federal or Federal-aid highway funds or FHWA approval.

“Base Flood” - The flood or tide having a one percent (1%) chance of being exceeded in any given year (100-year flood).

"Base flood plain" shall mean the area subject to flooding by the base flood.

“Design Flood” - The peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the state of the design flood.

"Encroachment" shall mean an action within the limits of the base floodplain. 23 CFR

“Floodproof” - To design and construct a project to keep floodwaters out or to reduce the effects of floodwaters.

“Natural and beneficial Floodplain Values” shall include but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, groundwater discharge.

"Practicable" shall mean capable of being done within reasonable natural, social, or economic constraints.

"Preserve" shall mean to avoid modification to the functions of the natural flood-plain environment or to maintain it as closely as practicable in its natural state.

"Regulatory floodway" shall mean the flood-plain area that is reserved in an open manner by Federal, State or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program). (Since the 1 foot is already accounted for, no increase more than 0.00 feet is allowed.) 23 CFR 650.109(m)

“Restore” shall mean to reestablish a setting or environment in which the functions of the natural and beneficial flood-plain values adversely impacted by the highway agency action can again operate.

“Risk” shall mean the consequences associated with the probability of flooding attributable to an encroachment. It shall include the potential for property loss and hazard to life during the service life of the highway.

Risk Analysis - An economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least expected cost to the public. It shall include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway-aggravated flood damage to other property, and for additional or interrupted highway travel.

“Significant encroachment” shall mean a highway encroachment and any direct support of likely base floodplain development that would involve one or more of the following construction-or flood-related impacts:

1) *A significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community’s only evaluation route.*
2) *A significant risk, or*
3) *A significant adverse impact on natural and beneficial flood-plain values.*

“Support base floodplain development” shall mean to encourage, allow, serve, or otherwise facilitate additional base flood-plain development. Direct support results from an encroachment, while indirect support results form an action out of the base flood plain.

Federal Emergency Management Agency (FEMA) Definitions. See 44 CFR 59 for a complete list of definitions. And see Home | FEMA.gov **for additional interpretations and guidance:**

Backwater is the effect of downstream flow on the water surface profile. The rise in water surface elevation due to encroachment.

The **Base Flood** is *a flood having a 1% chance of being equaled or exceeded in any given year*.

The **Base Flood Depth (BFD)** is the depth shown on the Flood Insurance Rate Map (FIRM) for Zone AO that indicates the depth of water above the highest adjacent grade resulting from a flood that has a 1% chance of equaling or exceeding that level in any given year.

The **Base Flood Elevation (BFE)** is the computed elevation to which floodwater is anticipated to rise during the base flood. BFEs are shown on Flood Insurance Rate Maps (FIRMs) and on the flood profiles. The BFE is the regulatory requirement for the elevation or floodproofing of structures. The relationship between the BFE and a structure's elevation determines the flood insurance premium. The BFE is the elevation of surface water resulting from a flood that has a 1% chance of equaling or exceeding that level in any given year. The BFE is shown on the Flood Insurance Rate Map (FIRM) for zones AE, AH, A1–A30, AR, AR/A, AR/AE, AR/A1– A30, AR/AH, AR/AO, V1–V30 and VE.

A **Community** is a political entity that *has the authority to adopt and enforce floodplain management regulations for the area under its jurisdiction*.

Development means *any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation or drilling operations or storage of equipment or materials.*

Encroachments are activities or construction within the floodway including fill, new construction, substantial improvements, and other development. These activities are prohibited within the adopted regulatory floodway unless it has been demonstrated through hydrologic and hydraulic analyses that the proposed encroachment would not result in any increase in flood levels.

Federal Emergency Management Agency (**FEMA**) is the Federal agency under which the NFIP is administered. In March 2003, **FEMA** became part of the newly created **U.S. Department of Homeland Security**.

A **Flood Boundary and Floodway Map (FBFM)** is a pre-Map initiatives floodplain management map delineates the 100-year (1% annual chance) and 500-year (0.2% annual chance) floodplains, floodway, and cross sections.

A **Flood Hazard Boundary Map (FHBM)** *means an official map of a community, issued by the Federal Insurance Administrator, where the boundaries of the flood, mudslide (i.e., mudflow) related erosion areas having special hazards have been designated as Zones A, M, and/or E.*

A **Flood Insurance Rate Map (FIRM)** *means an official map of a community, on which the Federal Insurance Administrator has delineated both the special hazard areas and the risk premium zones applicable to the community. A FIRM that has been made available digitally is called a Digital Flood Insurance Rate Map (DFIRM).* A FIRM is an official map of a community on which FEMA has delineated the Special Flood Hazard Areas (SFHAs), the Base Flood Elevations (BFEs) and the risk premium zones applicable to the community.

A **Flood Insurance Study (FIS)** *means an examination, evaluation and determination of flood hazards and, if appropriate, corresponding water surface elevations, or an examination, evaluation and determination of mudslide (i.e., mudflow) and/or flood-related erosion hazards.* A FIS is a compilation and presentation of flood risk data for specific watercourses, lakes, and coastal flood hazard areas within a community. When a flood study is completed for the NFIP, the information and maps are assembled into an FIS. The FIS report contains detailed flood elevation data in flood profiles and data tables.

Flood mitigation is defined as any sustained action that reduces or eliminates long-term risk to people and property from the effects of floods.

A **Floodplain** is *any land area susceptible to being inundated by floodwaters from any source.*

The **Floodway Fringe** (sometimes referred to as the flood fringe) is the area within the SFHA, are the portions of the floodplain beyond the floodway, which usually contains slow-moving or standing water during a base flood event.

Frequency Analysis (also Flood Frequency Analysis) are statistical techniques that estimate the probabilities of a flood event occurring. Flood Frequency isthe statistical number of years that takes place before the recurrence of a flood of the same magnitude. (10-year flood, 50-year flood, 100-year flood, etc.)

A **Functionally Dependent Use** *is a use which cannot perform its intended purpose unless it is located or carried out in close proximity to water. This term includes only docking facilities, port facilities that are necessary for the loading and unloading of cargo or passengers, and shipbuilding and ship repair facilities, but does not include long-term storage or related manufacturing facilities.* – FHWA has used it as the following: Functionally dependent use has been described as bridges, or any water conveyance structures or actions that facilitate the use of open space use (e.g. recreational trails, bicycle and pedestrian paths). Functionally dependent uses also include embankment, culverts, grading and guardrails, and other associated or required work that are required to support or protect the bridge or culvert.

A **Letter of Map Amendment (LOMA)** is an amendment to the currently effective FEMA map which establishes that a property is not located in a Special Flood Hazard Area (SFHA). A LOMA is issued only by FEMA.

A **Letter of Map Revision (LOMR)** is an official amendment to the currently effective FEMA map. It is issued by FEMA and changes flood zones, delineations and elevations.

A **Map Revision** is a change in the Flood Hazard Boundary Map (FHBM) or Flood Insurance Rate Map (FIRM) for a community which reflects revised zone, base flood or other information.

A **Mapped Community** is a Community (County, City or Village) which has Floodplain Mapping (FHM, FHBM, FIRM, or work maps) (see definition above).

A **Non-Mapped Community** is a Community (County, City or Village) which does not have Floodplain Mapping (see definition above). State Minimum Standards apply within these Communities.

A **Non-Participating Community** is a Community (County, City or Village) which does not participate in the National Flood Insurance Program (NFIP). A non-participating community does not regulate development activities that occur in floodplains (mapped or potential) within in its jurisdiction.

A list of Non-Participating Communities is maintained on the FEMA web site http://www.fema.gov/national-flood-insurance-program/national-flood-insurance-program-community-status-book.

The **National Flood Insurance Program (NFIP)** is the program of flood insurance coverage and floodplain management administered under the Act and applicable federal regulations promulgated in CFR Title 44/Part-60/Subpart-B.

A **Participating Community** is a Community for which FEMA has authorized the sale of flood insurance under the NFIP. A Participating Community regulates development activities, via ordinances and permits, which occur in floodplains (mapped or potential) within in its jurisdiction. A list of Participating Communities is maintained on the FEMA web site http://www.fema.gov/national-flood-insurance-program/national-flood-insurance-program-community-status-book.

A **Permit for Floodplain Development** is a permit is required before construction or development begins within any Special Flood Hazard Area (SFHA). Permits are required to ensure that proposed development projects meet the requirements of the NFIP and the community's floodplain management ordinance. A community must also review all proposed developments to assure that all necessary permits have been received from those governmental agencies from which approval is required by Federal or State law.

A **Regulatory Floodway** means the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. Communities must regulate development in these floodways to ensure that there are no increases in upstream flood elevations. For streams and other watercourses where FEMA has provided Base Flood Elevations (BFEs), but no floodway has been designated, the community must review floodplain development on a case-by-case basis to ensure that increases in water surface elevations do not occur, or identify the need to adopt a floodway if adequate information is available.

A **Special Flood Hazard Area (SFHA)** is an area having special flood, mudflow or flood-related erosion hazards and shown on a Flood Hazard Boundary Map (FHBM) or a Flood Insurance Rate Map (FIRM) Zone A, AO, A1-A30, AE, A99, AH, AR, AR/A, AR/AE, AR/AH, AR/AO, AR/A1-A30, V1-V30, VE or V. SFHAs are delineated on an NFIP map as being subject to inundation by the base (100-year) flood.

A **Zone** is a geographical area shown on a Flood Hazard Boundary Map (FHBM) or a Flood Insurance Rate Map (FIRM) that reflects the severity or type of flooding in the area.

Zone	Description
Moderate to Low Risk Areas – In communities that participate in the NFIP, flood insurance is available to all property owners and renters in these zones:	
B and X (shaded)	Area of moderate flood hazard, usually the area between the limits of the 100-year and 500-year floods. B Zones are also used to designate the base floodplains of lesser hazards, such as areas protected by levees from 100-year flood, or shallow flooding areas with average depths of less than one foot or drainage areas less than 1 square mile.
C and X (unshaded)	Area of minimal flood hazard, usually depicted on FIRMs as above the 500-year flood level, Zone C may have ponding and local drainage problems that don't warrant a detailed study or designation as base floodplain. Zone X is the area determined to be outside the 500-year flood and protected by levee from 100-year flood.
High Risk Areas – In communities that participate in the NFIP, mandatory flood insurance purchase requirements apply to all of these zones:	
A	Areas with a 1% annual chance of flooding. Because detailed analyses are not performed for such areas; no depths or base flood elevations are shown within these zones.
AE	The base floodplain where base flood elevations are provided. AE Zones are now used on new format FIRMs instead of A1-A30 Zones.
A1-A30	These are known as numbered A Zones (e.g. A14 or A 20). This is the base floodplain where the FIRM shows a BFE (old FIRM format).
AH	Areas with a 1% annual chance of shallow flooding, usually in the form of a pond, with an average depth ranging from 1 to 3 feet. Base flood elevations derived from detailed analyses are shown at selected intervals within these zones.
AO	River or stream flood hazard area, and areas with a 1% or greater chance of shallow flooding each year, usually in the form of sheet flow, with an average depth ranging from 1 to 3 feet.
AR	Areas with a temporarily increased flood risk due to the building or restoration of a flood control system (such as a levee or a dam).
A99	Areas with a 1% annual chance of flooding that will be protected by a Federal flood control system where construction has reach specified legal requirements. No depths or base flood elevations are shown within these zones.
High Risk – Coastal Areas – In communities that participate in the NFIP, mandatory flood insurance purchase requirements apply to all of these zones.	
V	Coastal areas with a 1% or greater chance of flooding and an additional hazard associated with storm waves. No base flood elevations are shown within these zones.
Zone	Description
VE, V1-V30	Coastal areas with a 1% or greater chance of flooding and an additional hazard associated with storm waves. Base flood elevations derived from detailed analyses are shown at selected intervals within these zones.
Undetermined Risk Areas	
D	Areas with possible but undetermined flood hazards. No flood hazard analysis has been conducted.

*Reference – FEMA Map Service Center

Nebraska Administrative Code Title 455 (NeDNR) Chapter 1 – Minimum Standards for Floodplain Management Programs Definitions:

"Base flood" shall mean the flood having a one per cent chance of being equaled or exceeded in magnitude in any given year.

"Flood" shall mean the water of any watercourse or drainway which is above the bank or outside the channel and banks of such watercourse or drainway.

"Floodway" shall mean the channel of a watercourse or drainway and the adjacent land areas that are necessary to be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than one foot.

"Flood fringe" shall mean that portion of the floodplain of the base flood which is outside of the floodway.

"Floodplain" shall mean the area adjoining a watercourse or drainway which has been or may be covered by floodwaters.

"Floodplain management regulations" shall mean and include zoning ordinances, subdivision regulations, building codes, and Title 455 Chapter 1 - 3 - other applications of the police power which are authorized by law to secure safety from floods and provide for the reasonable and prudent use of floodplains.

"New construction" shall mean obstructions for which the "start of construction" commenced on or after the effective date of the floodplain management regulation adopted by a community and includes any subsequent improvements to such obstructions.

"Obstruction" shall mean any wall, wharf, embankment, levee, dike, pile, abutment, projection, excavation (including the alteration or relocation of a watercourse or drainway), channel rectification, bridge, conduit, culvert, building, stored equipment or material, wire, fence, rock, gravel, refuse, fill, or other analogous structure or matter which may impede, retard, or change the direction of the flow of water, either in itself or by catching or collecting debris carried by such water, or that is placed where the natural flow of the water would carry such structure or matter downstream to the damage or detriment of either life or property. Dams designed to store or divert water are not obstructions if permission for the construction thereof is obtained from the Department of Natural Resources pursuant to The Safety of Dams and Reservoirs Act (Sections 46-1601 to 46-1670 R.R.S., 1943 as amended.)

"Potential Base Floodplain" shall refer to NDOT's policy, developed in partnership with NeDNR, to meet State Minimum Standards in unmapped communities by identifying watersheds greater than 640 acres as Potential Base Floodplains and evaluating the project's impact to the Base Flood Elevation at that location.

"Structure" shall mean a walled and roofed building that is principally above ground, as well as a manufactured home, and a gas or liquid storage tank that is principally above ground.

"Substantial improvement" shall mean any reconstruction, rehabilitation, addition, or other improvement of an obstruction, the cost of which equals or exceeds 50 percent of the market value of the obstruction before "start of construction" of the improvement. This includes obstructions which have incurred "substantial damage," regardless of the actual repair work performed. The term does not, however, include either (1) any project for improvement of a structure or other obstruction to correct existing violations of state or local health, sanitary, or safety code specifications which have been identified by the local code enforcement official and which are the minimum necessary to assure safe living conditions, or (2) any alteration of an "historic structure," provided that the alteration will not preclude the structure's continued designation as an "historic structure."

"Watercourse" shall mean any depression two feet or more below the surrounding land which serves to give direction to a current of water at least nine months of the year and which has a bed and well-defined banks.

Other General NDOT Terminology:

"Floodway fringe" - The portion of the 100-year floodplain that is not within the floodway and in which development and other forms of encroachment may be permitted under certain circumstances.

"Location Hydraulic Study" - The preliminary investigative study to be made of base floodplain encroachments by a proposed highway action. (This study must be performed by a registered engineer with hydraulic expertise.) For specific requirements, see 23 CFR 650.111.

"Longitudinal Encroachment" - An encroachment that is parallel to the direction of flow. Example: A highway that runs along the edge of a river is, usually considered a longitudinal encroachment.

Risk Assessment - An economic and/or non-economic assessment of the impacts associated with the floodplain encroachment(s). It is meant to be more general in detail than a risk analysis. The format and content of the Summary Floodplain Encroachment Report form is the minimum required for a risk assessment.

REFERENCES

23 CFR 650 Subpart A – Location and Hydraulic Design of Encroachments on Flood Plains

23 CFR 771 – Environmental Impact and Related Procedures

44 CFR 60.3 – Floodplain Management Criteria for Flood-Prone Areas

Additional Guidance on 23 CFR 650A, FHWA, September 20, 1992

Atlas of the Sand Hills, Bleed and Flowerday, 1998

Design Standards for Highways in National Flood Insurance Program Mapped Floodplains, FHWA, April 21, 1992

Executive Order 11988 – Floodplain Management

Flood Disaster Protection Act of 1973

Flood Hazard Mitigation Plan, Nebraska Department of Natural Resources, 2013

Guidance for Implementing the One-foot Standard for Encroachments on NFIP Floodplains, FHWA, December 22, 1985

Instructions and Guidance for Completing the Nebraska Categorical Exclusion Determination Form for Federal-Aid Projects June 2, 2015

Managing Floodplain Development through NFIP, Federal Emergency Management Agency

National Flood Insurance Act (42 USC 4001)

National Flood Insurance Reform Act in 1994

NDOT's Drainage Design and Erosion Control Manual

Nebraska Administrative Code Title 455

Nebraska Revised Statute Chapter 31 Drainage, Section 1017 Department; floodplain management

Procedures for Coordinating Highway Encroachments on Floodplains with the Federal Emergency Management Agency (FEMA), June 25, 1982

Programmatic Categorical Exclusion Agreement Between the Federal Highway Administration and the Nebraska Department of Roads Regarding the Processing of Highway Projects Categorically Excluded from the Requirements to Prepare either an Environmental Assessment or Environmental Impact Statement, April 2015

Significant Encroachments Minute Memo, FHWA, April 2, 1985

USACE Omaha District & NDOT's Flood Flow Frequency Prediction Equations Sand Hills Region of Nebraska, dated July 2003

Nebraska Administrative Code Title 455, Chapter 1, (Reference 1.16)

LINKS

23 CFR 650
23 CFR §650 Bridges, Structures, And Hydraulics - Code of Federal Regulations (ecfr.io)

23 CFR 771
23 CFR §771 Environmental Impact And Related Procedures - Code of Federal Regulations (ecfr.io)

42 USC 4001
42 U.S.C. § 4001 - U.S. Code Title 42. The Public Health and Welfare § 4001 | FindLaw

44 CFR 59
44 CFR §59 General Provisions - Code of Federal Regulations (ecfr.io)

44 CFR 60
44 CFR §60 Criteria For Land Management And Use - Code of Federal Regulations (ecfr.io)

Clean Water Act Section 401
The Clean Water Act Section 401 Certification Rule | Overview of Certification under Section 401 of the Clean Water Act | US EPA

Clean Water Act Section 404
Overview of Clean Water Act Section 404 | Section 404 of the Clean Water Act: Permitting Discharges of Dredge or Fill Material | US EPA

Corps of Engineers Section 408
Section 408 (army.mil)

DPO
https://dot.nebraska.gov/business-center/design-consultant/

Drainage Manual
http://dot.nebraska.gov/business-center/design-consultant/rd-manuals/

Executive Order 11988
Executive Order 11988 | FEMA.gov

FEMA
Home | FEMA.gov

FEMA Flood Map Service Center
FEMA Flood Map Service Center | Welcome!

National Flood Insurance Program (NFIP)
Flood Insurance | FEMA.gov

National Flood Insurance Program Maps
https://www.fema.gov/flood-insurance

NDOT Bridge Hydraulic Analysis Guidelines
https://dot.nebraska.gov/business-center/bridge/hydraulic/

NDOT Environmental Procedures Manual
https://dot.nebraska.gov/projects/environment/pubs/docs/

Nebraska Administrative Code Title 455, Chapter 1
Microsoft Word - MinStds-Title 455 (nebraska.gov)

Nebraska Categorical Exclusion Guidance
https://dot.nebraska.gov/projects/environment/pubs/docs/

Nebraska Department of Natural Resources
Welcome | Department of Natural Resources (nebraska.gov)

Department of Natural Resources Floodplain Interactive Map
https://dnr.nebraska.gov/floodplain/interactive-maps

Nebraska Revised Statutes Chapter 31
Nebraska Legislature - Revised Statutes Chapter 31

Nebraska Revised Statutes Chapter 49
Nebraska Legislature - Revised Statutes Chapter 49

NEPA
https://ceq.doe.gov/

USCG Section 9 Bridge Permit
US Section 9 Coast Guard Bridge Permit | Agency of Transportation (vermont.gov)

The information contained in the Glossary, dated August 2006, has been updated to reflect the August 2018 Errata. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design and Drainage Manual chapters and other reference material citations occurring since the latest publication of the Glossary.

1. GLOSSARY

A. Acronyms, Abbreviations and Symbols

A	Area of cross section
A	Watershed area
a	Depth of depression
C	Runoff coefficient or coefficient
d	Depth of gutter flow at the curb line
D	Diameter of pipe
E_o	Ratio of frontal flow to total gutter flow, Q_w/Q
h	Height of curb opening inlet
H	Head loss
I	Rainfall intensity
K	Coefficient
L	Length of curb opening inlet
L	Pipe length
L	Pavement width
L	Length of runoff travel
n	Roughness coefficient in Manning formula
P	Perimeter of grate opening, neglecting bars and side against curb
P	Tire pressure
Q	Rate of discharge in gutter
Q_i	Intercepted flow
Q_s	Gutter capacity above the depressed section
Q_T	Total flow
R	Hydraulic radius
S or S_x	Cross slope

S	Crown slope of pavement
S or S_L	Longitudinal slope of pavement
S_w	Depression section slope
T	Top width of water surface (spread on pavement)
t_c	Time of concentration
T_s	Spread above depressed section
V	Vehicle speed
V	Velocity of flow
W	Width of depression for curb opening inlets
W_d	Rotational velocity on dry surface
WD	Water depth
W_w	Rotational velocity on flooded surface
y	Depth of flow in approach gutter
Z	T/d, reciprocal of the cross slope

B. <u>Terms and Definitions</u>

Asphaltic curb and flume A permanent or temporary erosion control measure.

Baffle piers See dragon teeth.

Beneficial Uses Streams, lakes, rivers, and other water bodies, have uses to humans and other life; these uses are referred to as the Beneficial Uses of a water body. Beneficial uses are assigned to surface waters within or bordering upon the State of Nebraska. Some uses require higher quality water than others. When multiple uses are assigned to the same waters, all assigned uses will be protected. The beneficial uses defined by these standards are:

- Primary Contact Recreation
- Aquatic Life
  - Cold water (Class A and B)
  - Warm water (Class A and B)
- Water Supply
  - Public Drinking Water
  - Agricultural
  - Industrial
- Aesthetics

Bypass	Flow that bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets can be designed to allow a certain amount of bypass. Also, inlets may be designed to allow a certain amount of bypass for one design storm and larger or smaller amounts for other design storms.
Cellular confinement system	Three-dimensional cells constructed of heavy-duty polyethylene filled with various materials.
Clean Water Act	A federal law that controls the discharge of pollutants into surface water in a number of ways, including discharge permits.
Combination inlet	A drainage inlet usually composed of a curb-opening inlet and a grate inlet.
Common Plan of Development	A contiguous area where multiple separate and distinct land disturbing activities may be taking place at different times, on different schedules, but under one proposed plan. One plan is broadly defined to include design, permit application, advertisement or physical demarcation indicating that land-disturbing activities may occur.
Concrete ditch lining	Ditch floors and banks paved with cast-in-place reinforced concrete.
Concrete slope protection	The placement of concrete slabs on bridge embankments for erosion control.
Covercrop seeding	The establishment of temporary vegetative cover on disturbed areas with appropriate rapidly growing annual plants.
Culvert protection	A temporary sediment filter located at culvert inlets to prevent sediment from entering, accumulating in and being transferred by the drainage system prior to permanent stabilization.
Curb-opening inlet	A drainage inlet consisting of an opening in a roadway curb.
Diffused surface waters	Waters which have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel. They appear as puddles, sheet or overland flow and rills; and continue to be surface waters until they disappear from the surface by infiltration or evaporation, or until by overland or vagrant flow, they reach well-defined watercourses or standing bodies of water like lakes or seas.
Dragon teeth	Alternating rows of rectangular-shaped objects that are used to dissipate energy at the outlet of culverts.
Drop inlet	A drainage inlet with a horizontal or nearly horizontal opening.
Drop structures	Structures designed to transport stormwater runoff down a highway embankment.

Endangered Species Act	The Endangered Species Act (ESA) is an environmental law that protects threatened and endangered plants and animals and the habitats in which they are found. The lead federal agencies for implementing ESA are the **U.S. Fish and Wildlife Service** (**FWS**) and the **U.S. National Oceanic and Atmospheric Administration** (**NOAA**) **Fisheries Service**. The **FWS** maintains a worldwide list of endangered species. Species include birds, insects, fish, reptiles, mammals, crustaceans, flowers, grasses, and trees.
Environmental Protection Agency (EPA)	The **U.S. Environmental Protection Agency** (**EPA** or sometimes **USEPA**) is an agency of the federal government of the United States charged with protecting human health and the environment, by writing and enforcing regulations based on laws passed by Congress.
Ephemeral Stream	Stream that flows only during and immediately after precipitation events.
Equivalent cross slope	An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.
Erosion	A natural process whereby soil particles are dislodged by rainfall and carried away by runoff.
Erosion checks	Hay or straw barriers placed in ditches at predetermined intervals to slow the velocity of water and cause silt deposition.
Erosion control	Techniques and measures utilized to provide direct protection to the soil surface and prevent erosion of soil particles.
Erosion control netting	Photodegradeable lightweight flexible netting used over slope protection.
Erosion control products	Erosion control blankets or mats used to control erosion on critical areas by providing a microclimate which protects young vegetation and promotes its establishment.
Flanking inlets	Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.
Flood waters	Former stream waters which have escaped from a watercourse (and its overflow channel) and flow or stand over adjoining land; they remain as such until they disappear from the surface by infiltration, evaporation or return to a natural watercourse. They do not become surface waters by mingling with such waters; nor do they become stream waters by eroding a temporary channel.

Foreign waters	Those waters imported by a user from one watershed into another.
Frontal flow	The portion of the flow that passes over the upstream side of a grate.
Gabions	Rectangular, rock-filled wire baskets suitable for use as lining of high steep channel banks; channel drop structures; and energy dissipation at the outlet of culverts.
Grate inlet	A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale or channel.
Grate perimeter	The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.
Ground cover	Plants used on embankments to eliminate mowing; not considered adequate erosion protection.
Ground waters	Waters situated below the surface of the land, irrespective of their source and transient status.
Gutter	That portion of the roadway section adjacent to the curb, which is utilized to convey stormwater runoff. If may include a portion, or all, of a traveled lane, shoulder or parking lane, and a limited width, adjacent to the curb, may be of different materials and have a different cross slope.
Hydraulic grade line	The locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).
Impervious Surface	A hard surface area that prevents or retards the entry of water into the soil, thus causing water to run off the surface in greater quantities and at an increased rate of flow.
Inlet efficiency	The ratio of flow intercepted by an inlet to total flow in the gutter.
Intercepting dike	A temporary or permanent ridge of soil constructed at the top or base of a sloping disturbed area used to divert storm runoff from upstream drainage areas away from unprotected disturbed areas.
Intercepting ditch	A channel constructed either across, at the top, at the midpoint, or at the toe of a slope to intercept and convey water at non-erosive velocities to an adequate and stable outlet.
Intermittent Stream	An intermittent or seasonal stream is one that has a consistent base flow, but only for part of the year.
Lakes and ponds	Relatively permanent bodies of water substantially at rest in depressions of natural origin.

Land Disturbance	Areas of exposed, erodible soil, including stockpiles, that are within the limits of construction and that result from construction activities.
Linear Facility	Highway, Local Road.
Local Public Agency (LPA)	Any local political subdivision, board, commission, governmental entity, or civic organization sponsoring a federally funded transportation project and determined to be qualified to assume the administrative responsibilities for such projects by **NDOT**.
Marshes	Lands saturated by waters flowing over the surface in excess of infiltration capacity, as in sloughs of rivers and tidal channels.
Maximum Extent Practicable (MEP)	MEP is the process of evaluating the selected STFs based on legal and institutional constraints, technical feasibility, relative effectiveness, and cost/benefit ratio. For the purpose of the **NDOT's** Stormwater Management Program, implementation of STFs consistent with Chapter 3 shall constitute MEP.
MS4 Community	An Urbanized Area with a population of 10,000 or greater and a population density of at least 1,000 people/square mile. See Appendix O, "Regulated MS4s in Nebraska", for the official maps of MS4 communities.
MS4 Permit	An MS4 Permit (NPDES Permit) is EPA's program to control the discharge of pollutants to waters of the United States. NPDES is a part of the federal CWA, which requires point and non-point source dischargers to obtain permits. These permits are referred to as NPDES permits. ➢ Phase I, issued in 1990, required medium and large cities or certain counties with populations of 100,000 or more to obtain NPDES permit coverage for their stormwater discharges. ➢ Phase II, issued in 1999, required regulated small MS4s in urbanized areas, as well as small MS4s outside the urbanized areas that are designated by the permitting authority, to obtain NPDES permit coverage for their stormwater discharges.
Mulching	Application of plant residues or other suitable materials to the soil surface to protect the surface from raindrop impact and to reduce the velocity of overland flow.

Municipal Separate Storm Sewer System (MS4)	A conveyance or system of conveyances (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, man-made channels, or storm drains) which: ➢ Are owned and operated by a state, city, town, borough, county, parish, district, association, or other public body (created by... or pursuant to state law) having jurisdiction over disposal of sewage, industrial wastes, stormwater, or other wastes, including special districts under state law such as a sewer district, flood control district or drainage district, or similar entity, or an Indian tribe or an authorized Indian tribal organization, or a designated and approved management agency under section 208 of the Clean Water Act (CWA) that discharges to waters of the United States; ➢ Are designed or used for collecting or conveying stormwater; ➢ Is not a combined sewer; and ➢ Which is not part of a publicly owned treatment works (POTW). [40 CFR 122.26(b)(8)].
National Pollutant Discharge Elimination System (NPDES)	The National Pollutant Discharge Elimination System (NPDES) Stormwater Program regulates stormwater discharges from three potential sources: municipal separate storm sewer systems (MS4s), construction activities, and industrial activities. Most stormwater discharges are considered point sources, and operators of these sources may be required to receive an NPDES permit before they can discharge. This permitting mechanism is designed to prevent stormwater runoff from washing harmful pollutants into local surface waters such as streams, rivers, lakes or coastal waters.
Nebraska Department of Environmental Quality (NDEQ)	The **Nebraska Department of Environmental Quality** was created pursuant to passage of the Nebraska Environmental Protection Act in 1971. This State Agency is responsible for the protection of Nebraska's air, land and water resources.
New Pavement	New Pavement is defined as an impervious surface which is placed in an area currently devoid of such surfacing, or the complete removal and replacement of existing surfacing with modification of the base and/or subgrade.
Non-Linear Facilities	Rest Areas, Maintenance Yard, Offices, etc.

Off Line Treatment	An Offline STF is one where only a selected amount of stormwater runoff, frequently the WQV, is diverted through the STF and all additional runoff bypasses the STF.
Off-Site Mitigation	Construction of STF(s) in a location not immediately adjacent to (but within the MS4 of) a given project, in lieu of construction of STF(s) on the given project. When stormwater treatment is not feasible in a given project, there may be an option to mitigate the Water Quality Volume on a separate **NDOT** project within the same MS4 community. Currently, this is not an acceptable treatment option. The **NDOT** will first need to develop an agreement with the **Nebraska Department of Environmental Quality** on how this would be administered before this practice will be allowed.
On Line Treatment	An Online STF is one where all stormwater runoff generated by the project is conveyed through the STF.
Percolating waters	Waters which have infiltrated the surface of the land and moved slowly downward 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. Percolating waters confined below impermeable formations with sufficient pressure to spring or well up to the surface are termed artesian waters; those detained or retained above an impermeable formation, so as to stand above and detached from the main body of ground water, are called perched waters.
Perennial Stream	Stream or river (channel) that has continuous flow in parts of its bed all year round during years of normal rainfall.
Permanent slope protection	Spreading and crimping of hay on bare soil in conjunction with seeding.
Plunge basin	A type of energy dissipator that may be used where flows issue from a freely discharging pipe where the water jet subsequently discharges into the air and then plunges downward into the basin.
Pollutant	Any constituent present in sufficient quantity to impair the beneficial uses of a receiving waterbody. The most common contaminants in highway runoff are suspended solids, heavy metals, inorganic salts, and aromatic hydrocarbons that accumulate on the road surface as a result of regular highway operation and maintenance activities.
Post Project	Refers to the time period after a construction project is completed (represented by the full establishment of vegetation).
Pressure head	The height of a column of water that would exert a unit pressure equal to the pressure of the water.

Priority Stormwater Outfall	Concentrated stormwater flow locations from areas with a net increase of at least 5,000 square feet of New Pavement (including bridge surfaces) directly discharging from State ROW to the following locations within the MS4 boundary: • Streams – Perennial and Intermittent, • Lakes and Ponds, • Wetlands, • Municipal Separate Storm Sewer System, • Ephemeral drainage that directly discharges to one of the above located beyond the ROW line and within the distance identified in Appendix N.
Receiving Water	Creeks, streams, rivers, lakes, estuaries, or other surface water bodies into which stormwater is discharged.
Revet mattress	A special type of gabion with a large surface area-to-thickness ratio.
Riprap	A layer, facing or protective lining of stones over filter fabric placed to prevent erosion, scour or sloughing.
Scour hole	An energy dissipator consisting of a preformed excavated hole or depression that is lined with riprap of a stable size to prevent scouring.
Scupper	A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.
Sediment	Eroded soil deposited by gravity in streams, lakes and reservoirs.
Sediment basin	A temporary barrier or dam with a controlled stormwater release structure formed by constructing an embankment of compacted soil across a drainage way.
Sediment control	Techniques and measures utilized to remove sediment from waters by filtering or slowing the velocity of the water.
Sedimentation	The natural process of deposition of eroded soil.
Seeding	Permanent placement of seed on unsurfaced foreslopes, ditches, backslopes, shoulders, medians, and other areas, as specified, once the finish grade is established
Side-flow interception	Flow that is intercepted along the side of a grate inlet, as opposed to frontal interception.
Silt fence	A temporary or permanent sedimentation barrier consisting of synthetic or natural fabric.
Silt trap	A temporary ponding area formed by excavating a ditch along the path of water.

Slope drain	A pipe installed above grade, extending from the top to the bottom of a cut or fill slope to temporarily transport concentrated stormwater runoff safely down the face of a cut or fill slope.
Slotted drain inlet	A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect, and transport the flow.
Sodding	Transplanting of ready to grow grasses, done mostly in urban areas limited to occupied residential property and business sites.
Splashover	Portion of the frontal flow at a grate, which skips or splashes over the grate and is not intercepted.
Spread	The width of flow in the gutter measured laterally from the roadway curb.
Springs	Percolating waters issued by natural forces from the earth.
Storage or excess supply	Mainly project storage, where waters are retained from investigation, residential, municipal or industrial use.
Storm drain	That portion of the storm drainage facility that receives runoff from the inlets and conveys the runoff to an adequate outfall. Culverts discharging to the storm drainage system are considered part of the system.
Stormwater Outfall	A point source at the point where a facility and/or municipal separate storm sewer discharges to waters of the state. For **NDOT** this means anywhere that intentionally collected stormwater flow exits the right-of-way and discharges to a water of the state.
Stormwater Run-on	Stormwater run-on is defined as any stormwater which intermingles with the Treatment Drainage Area runoff prior to treatment. This can occur as either overland flow or underground flow via culvert or storm sewer pipe. **NDOT** projects can receive stormwater run-on from both adjacent properties and other parts of the highway or right-of-way.
Stormwater Runoff	Rainfall, snowmelt, and other surface water drainage; that does not evaporate or infiltrate the ground because of impervious land surfaces or a soil infiltration rate lower than rainfall intensity, but instead flows onto adjacent land or into waterbodies or is routed into a drain or sewer system.
Stormwater Treatment Facility (STF)	A STF is a measure that is implemented to protect water quality and reduce potential for pollution associated with stormwater runoff and includes any program, technology, process, siting criteria, operating method, or device that controls, prevents, removes, or reduces pollution. STFs are a combination of permanent structural and/or non-structural facilities used to improve stormwater quality throughout the functional life of the roadway.

Stream waters	Former diffused surface waters that have entered and now flow in a well-defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in the bed or banks of the watercourse. They continue as stream waters as long as they flow in the watercourse, and include overflow and multiple channels as well as the ordinary or low-water channel.
Surface waters	Waters commonly held to be those above the rock or soil surface of the earth.
Swamps	Lands saturated by ground water standing at or near the surface.
Temporary seeding	The establishment of permanent vegetation using perennial grasses for a short duration, usually two years or less; generally used in staged construction. May also consist of covercrop seed with hydraulically applied heavy mulch for temporary roads.
Temporary slope protection	Spreading and anchoring of hay, straw or rushes without seeding. Typically used on temporary roads in or near sandhills, but may be used other places.
Total Suspended Solids (TSS)	TSS is the weight of particles that are suspended in water. Suspended solids in water reduce light penetration in the water column, can clog the gills of fish and invertebrates, and are often associated with toxic contaminants because organics and metals tend to bind to particles.
Treatment Drainage Area	The Treatment Drainage Area is defined as the area of New Pavement on the project.
Underground streams	Flows of ground waters parallel to and adjoining stream waters and usually determined to be integral parts of the visible streams.
Velocity head	A quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.
Waste and artificial waters	Waters due to escape or seepage from constructed works.

Watercourse	A definite channel with bed and banks within which water flows, either continuously or in season. A watercourse is continuous in the direction of flow and may extend laterally beyond the definite banks to include artificial channels such as canals and drains, except when these are natural channels lawfully trained or restrained by the works of man. It does not include all depressions or swales through which surface or errant waters pass.
Waters of the State	Waters within the jurisdiction of the state including all streams, lakes, ponds, impounding reservoirs , marshes, wetlands, water courses, waterways, wells, springs, irrigation systems, drainage systems, and all other bodies or accumulation of water, surface and underground, natural or artificial, public or private, situated wholly or partly within or bordering upon the state.
Water Quality Volume (WQV)	WQV is defined as the amount of storm water runoff from a given storm that should be captured and treated in order to remove a majority of storm water pollutants on an average annual basis. This is equal to one-half inch of runoff from the Treatment Drainage Area.
Water Quality Volume Discharge Rate	The WQV Discharge Rate is the peak stormwater discharge generated by the water quality volume rainfall event using the **Natural Resources Conservation Service** (**NRCS**) Curve Number (CN) procedure.
Weighted Q Method	A calculation method that determines the peak stormwater runoff under the NRCS Curve Number procedure based on totaling the peak runoff values calculated from each individual land use type.
Weighted CN Method	A calculation method that determines the peak stormwater runoff under the NRCS Curve Number procedure based on averaging the curve numbers of individual land use types before calculating peak runoff.
Wetland	Areas that are inundated or saturated by surface or ground water at a frequency or duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs and similar areas. Wetlands are regulated by either State or Federal Agencies.

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