The information contained in Chapter One: Drainage, dated August 2006, has been updated to reflect the **June 2026 Errata**. The errata addresses errors, changes in procedure, changes in NDOT department titles, changes in other Drainage Manual and Roadway 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. 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 roadway 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 (Ref. 1.1) and the **Federal Highway Administration (FHWA)** Hydraulic Design Series No. 2: Highway Hydrology (Highway Hydrology (HDS-2)). Additional **FHWA** Hydraulics publications may be found at (Publications - Hydraulics - Bridges & Structures - Federal Highway Administration).

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 Sediment Control of this manual).

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 of this chapter, the Glossary of this manual, and Chapter 5 of **AASHTO's** Highway Drainage Guidelines (Ref. 1.2) 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 a 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 small so 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 enroute 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 floodwaters are 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 (*RDM*) (Ref. 1.3), Chapter Thirteen: Planning and Project Development, Section 6, for additional information (https://dot.nebraska.gov/business-center/design-consultant/rd-manuals/).

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** (**Corps**), **Federal Emergency Management Administration** (**FEMA**), **Environmental Protection Agency** (**EPA**), **Department of Water, Energy, and Environment** (**DWEE NE**), and the **Natural Resource Districts** (**NRD**) have provisions that must be followed. **DWEE NE** deals with urban floodplain issues, roadway designers may consult with **DWEE NE** regarding drainage computations for urban areas. **DWEE NE** also has authority and primary responsibility in erosion control. See the *RDM*, Chapter Thirteen: Planning and Project Development (Ref. 1.3), Section 6, 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:

- Environmental impacts
- Cost of construction and right-of-way
- Delays, interruptions and inconvenience to traffic due to floods or roadway failures
- Injuries and hazards to people 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 clear 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 1:6 or flatter slopes within the lateral obstacle clear distance in the direction of oncoming traffic, 1:10 in depressed medians
- Headwalls shall not be located within the lateral obstacle clear zone unless they are shielded
- Cross drainage culverts greater than 36 in. diameter should 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 clear zone for pipe larger than 36 in. in diameter; these devices may also be used on smaller culverts presenting an 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 clear 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
- 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), the 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:

- 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** (**DE**) should be consulted for information about previous flooding experiences.

5.B Mapping

The roadway 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 roadway 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 the **Bridge Hydraulic Engineer**.

The Nebraska Administrative Code Title 455 (Ref. 1.4), Chapter 1, (https://dnr.nebraska.gov/sites/default/files/doc/desk-reference/legal-authority/Title_455_0708.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.

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 Appendix S: “Floodplain Policy”, in this manual.

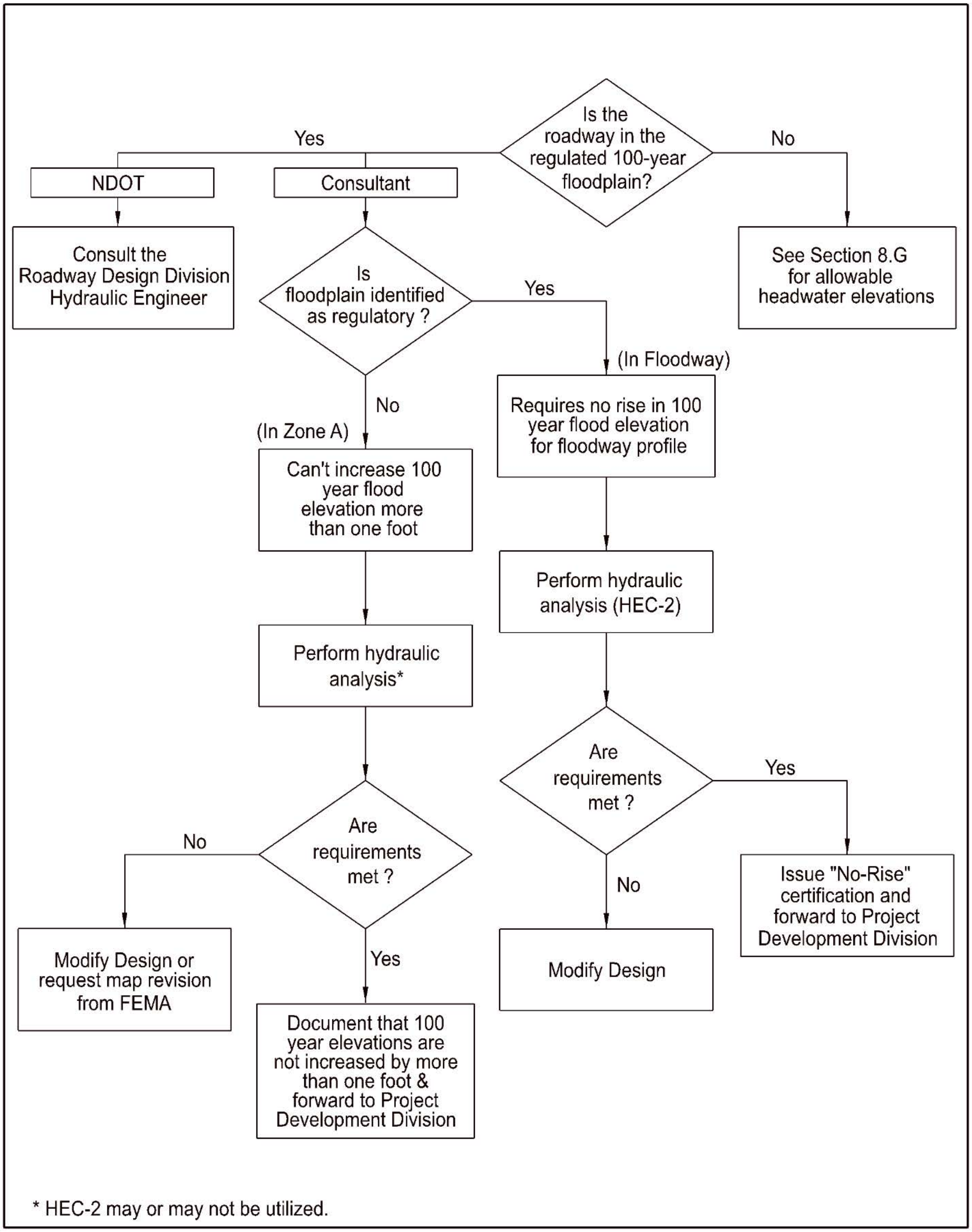


Exhibit 1.1 Floodplain Flow Chart

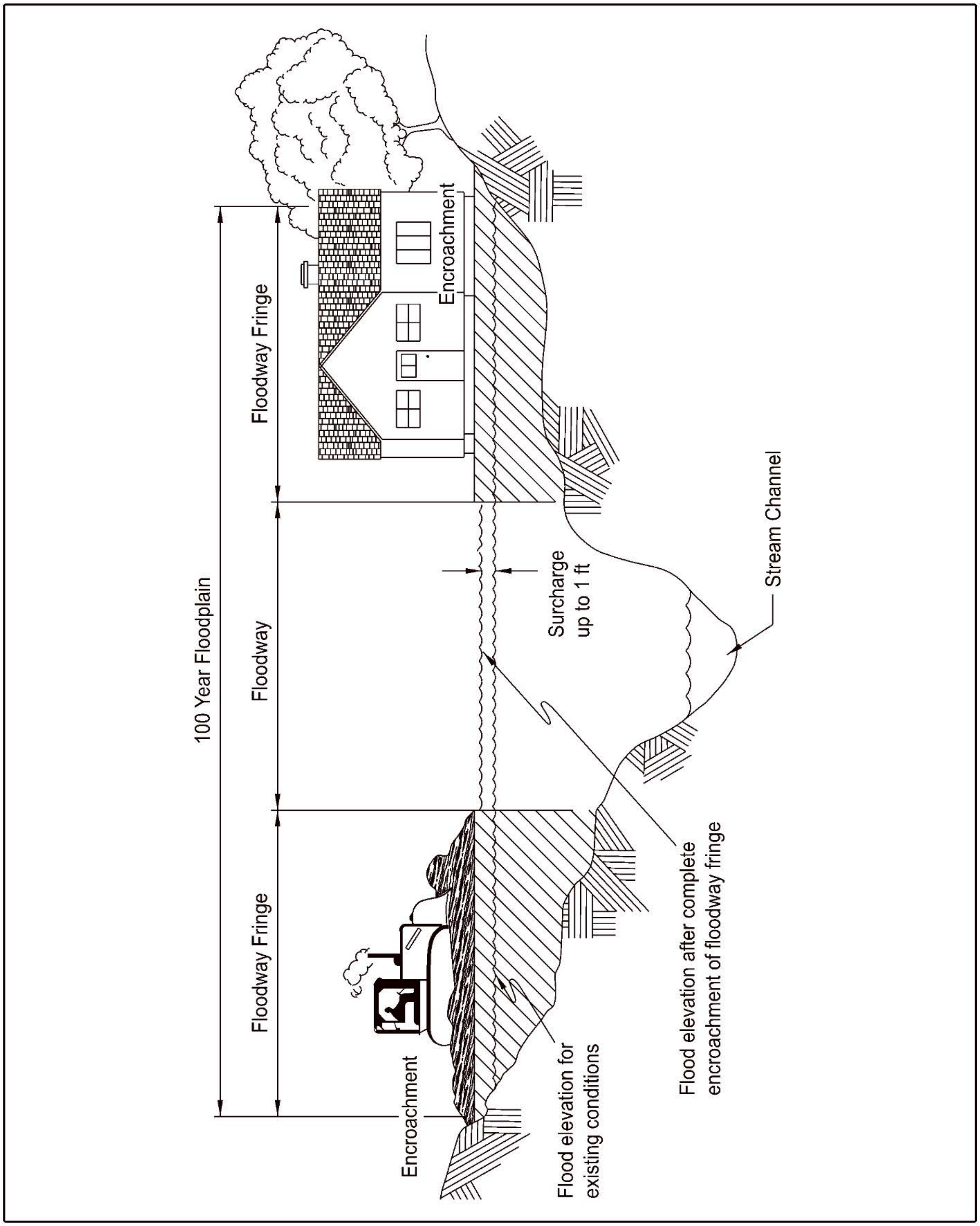


Exhibit 1.2 100-Year Floodplain Schematic
(Source: Ref. 1.4)

5.D <u>Utilities</u>

The preliminary survey will include the location of existing utilities (See the *RDM* (Ref. 1.3), Chapter Ten: <u>Miscellaneous Design Issues</u>, Section 12). Utility conflicts for both existing and proposed installations must be considered during drainage design.

5.E <u>Watershed Characteristics</u>

During preliminary drainage design the roadway 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 these documents.

5.F <u>Land Use</u>

The roadway 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 <u>Existing Drainage Facilities</u>

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 roadway 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 clear distance. In these instances the existing culvert may be extended and hydraulic analysis of the culvert will not be necessary.

5.H <u>Preliminary Layout</u>

Preliminary layouts or schematics, featuring the basic components of the intended design, are useful to the roadway 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 roadway designer to place existing utilities on the cross-section drawings. With this layout, the roadway designer can proceed with the detailed process of culvert and storm drainage design calculations, adjustments, and refinements.

5.I <u>Field Visit</u>

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

6. HYDROLOGY

Hydrology is the study of the movement and distribution of water. Runoff is the drainage that leaves an area as either 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 roadway designer should be familiar with the many factors or characteristics that affect peak runoff rates, including:

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, roadway designers use probability or frequency concepts with a specified rate or volume of flow which 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

NDOT 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 frequencies were 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, such as where a 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.

DWEE NE may designate some drainage ways as special flood prone areas. The use of the 100-year storm frequency is required for the design of all drainage structures in these specially designated areas (See Section 5.C of this chapter for additional information).

Drainage Structure	Design Storm Frequency by Facility Type/ADT		
	Interstate	2000 ADT & Over	1999 ADT & Under
Culvert	50 year	50 year	25 year
Storm Sewer	*	*	*
Storm Sewer on Depressed Roadways	50 year	50 year	50 year
Ditch Drop Pipe	50 year	50 year	25 year
Intercepting Dike/Backslope Drop Pipe	25 year	25 year	25 year
Temporary Facilities (Duration ≤ Two Years)**	2 year	2 year	2 year

* For High-Speed Roadways (≥ 50 mph) the design storm is 50-year. For Low-Speed Roadways (≤ 45 mph) the design storm is 10-year. See EXHIBIT 1.37.

** 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 discussed.

Exhibit 1.3 Design Storm Frequencies

6.C.2 Storm Sewer Design Storm

Urban storm sewers are generally designed for the 10-year storm; 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 roadway designer must determine where surface water not collected by the system will be carried. This uncollected surface flow cannot be allowed to either impact or be 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 roadway designer should verify that 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 **NDOT** with reasonable written assurances of a present plan for a future upgrade of its' municipal facilities. The **Municipality** shall provide **NDOT** with the details of its proposed improvements that will convey the design event determined by **NDOT**.
    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 **NDOT** 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 roadway designer may use methods other than the **NDOT** recommended methods for computing peak runoff to compare results (for other methods consult the **Bridge 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 following assumptions:

- The peak flow occurs only during the peak rainfall event
- The peak flow 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 of 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 of this chapter 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./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 of this chapter). 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 of this chapter 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: $Ra_i = (i_{15}-i_{30}) \div (Tc_{30}-Tc_{15}) = (4.9-3.45) \div (30-15) = 0.097$

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

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

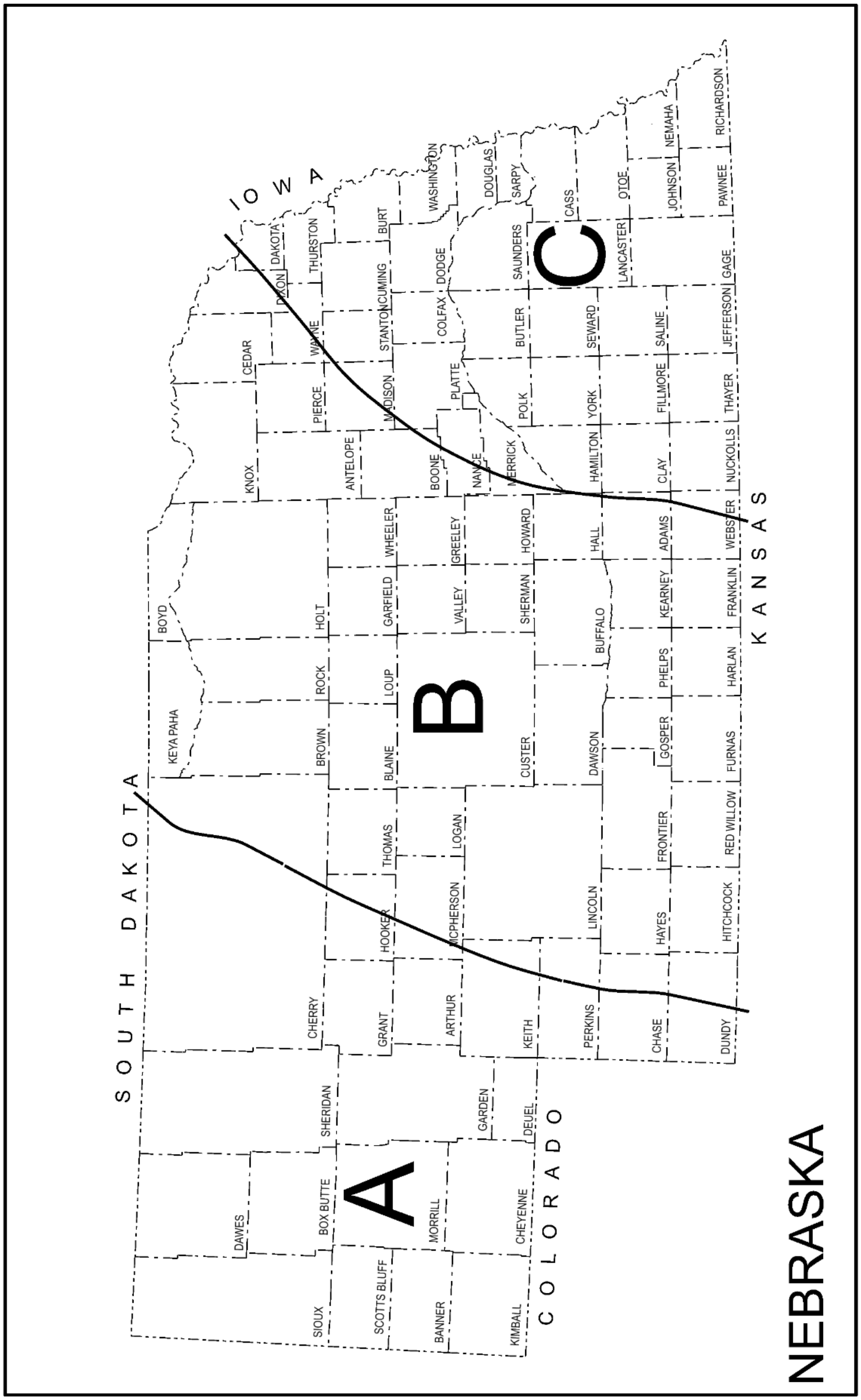
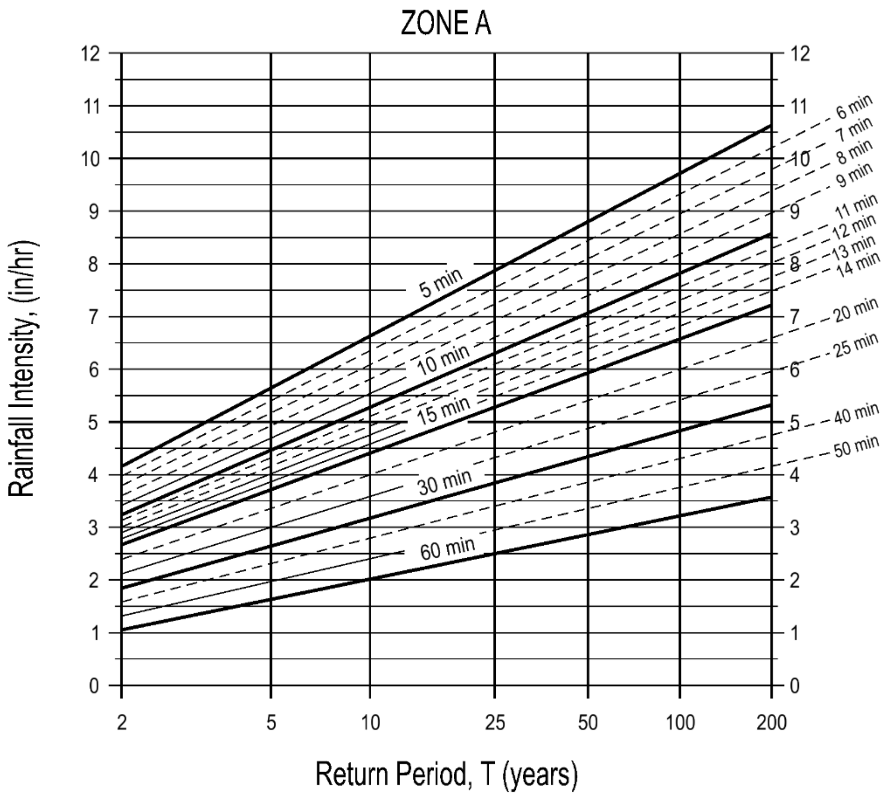


Exhibit 1.6 Nebraska Rainfall Zones for Use with Rainfall Intensities in Rational Method



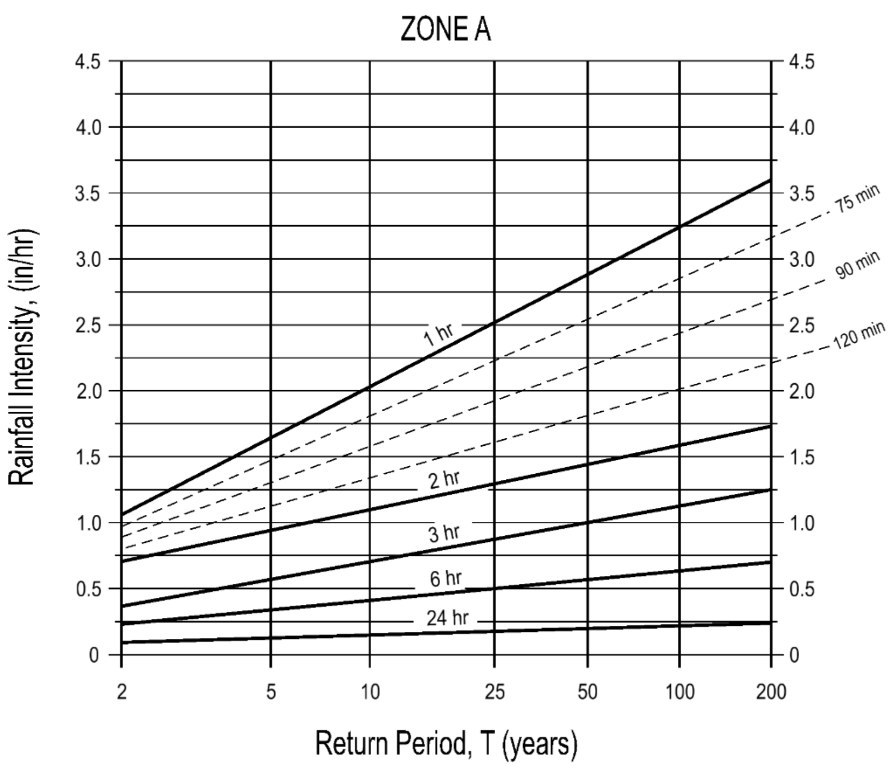


Exhibit 1.7 IDF Charts for Western Nebraska (Zone A)

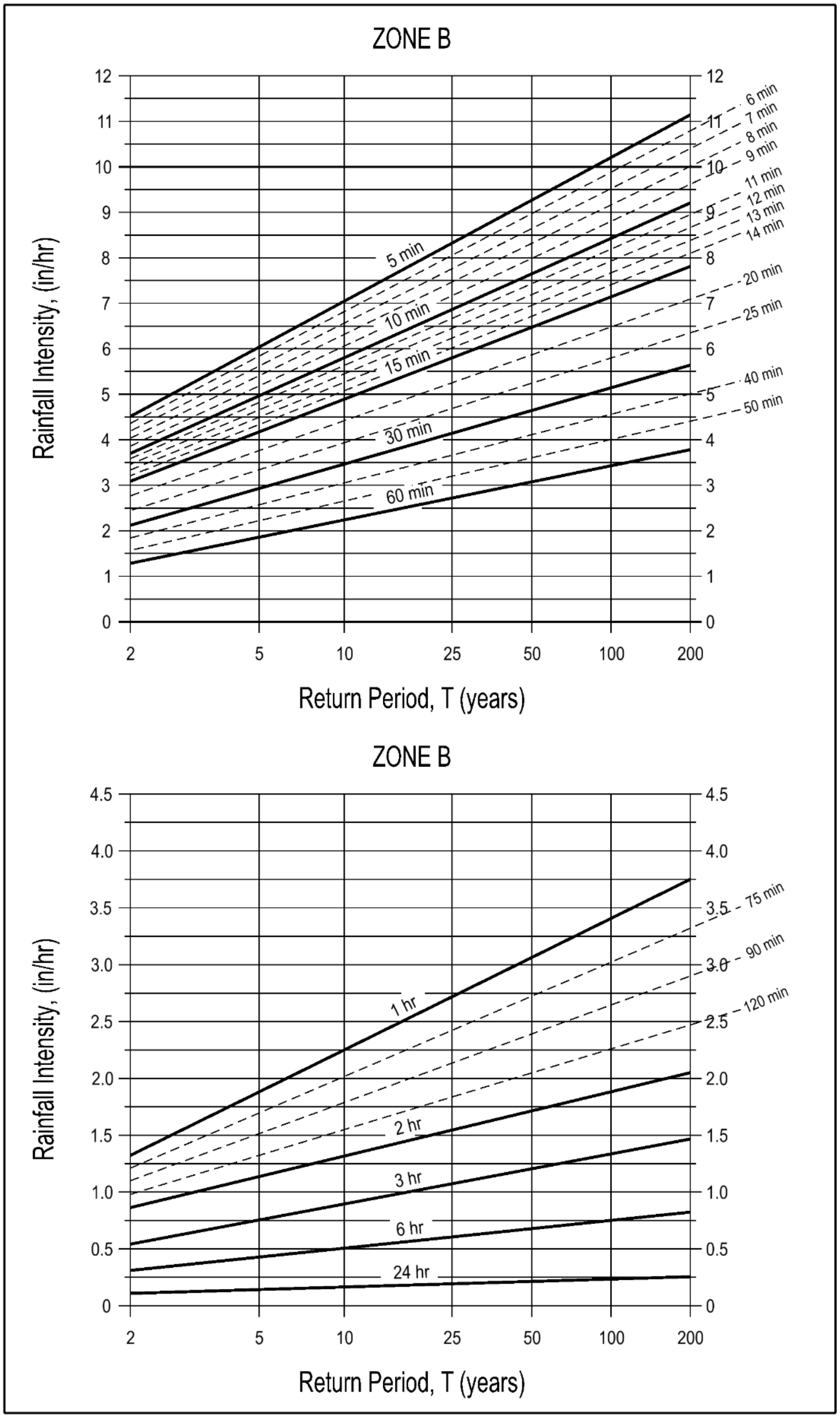


Exhibit 1.8 IDF Charts for Central Nebraska (Zone B)

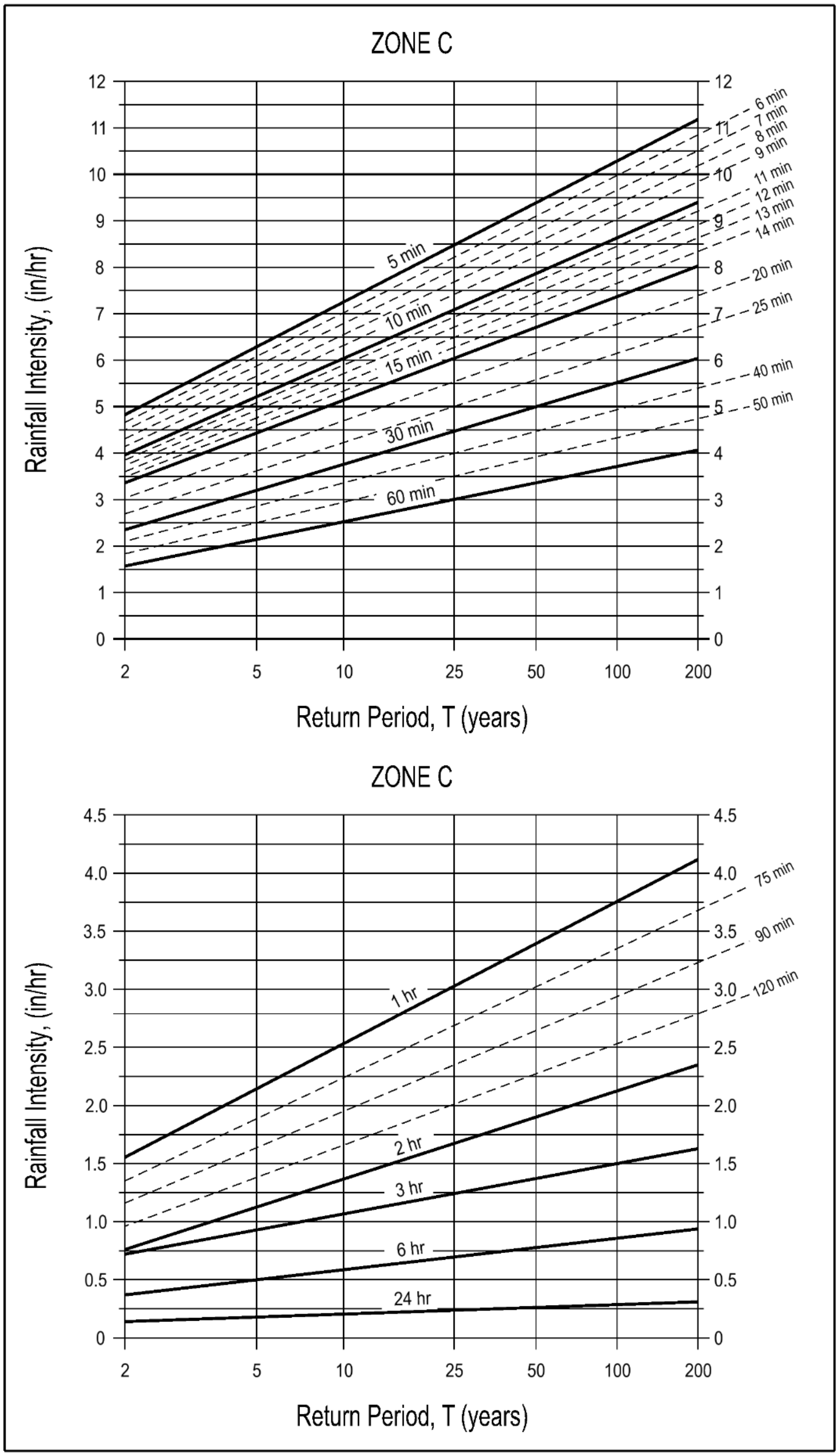


Exhibit 1.9 IDF Charts for Eastern Nebraska (Zone C)

6.D.1.c Time of Concentration

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 roadway 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 (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 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 roadway designer shall use 5 min. for the time of concentration.

The roadway 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:

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

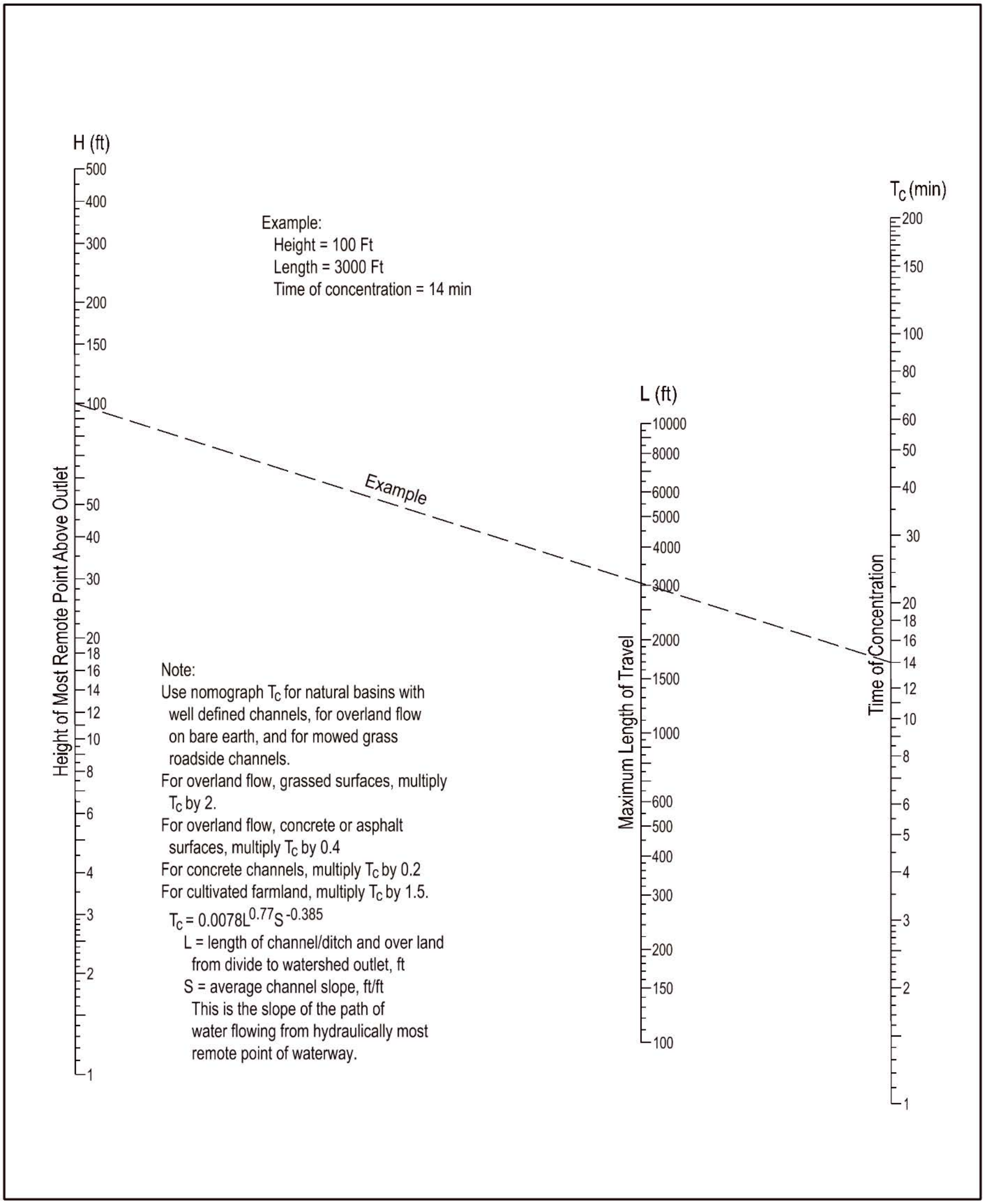


Exhibit 1.11 Time of Concentration of Small Drainage Basins (640 Ac. Or Less)
(Source: Ref. 1.5)

6.D.1.d Drainage Area

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

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 analysis, the State of Nebraska was divided into five regions. These regional divisions were selected to reduce the standard of error for the regression equations results. The five hydrologic regions of Nebraska (See EXHIBIT 1.12) are:

- 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 is basically 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 (the South Loup, Middle Loup, North Loup, and Loup Rivers and Cedar River and Beaver Creek)
- Region 5 is the Blue River basin

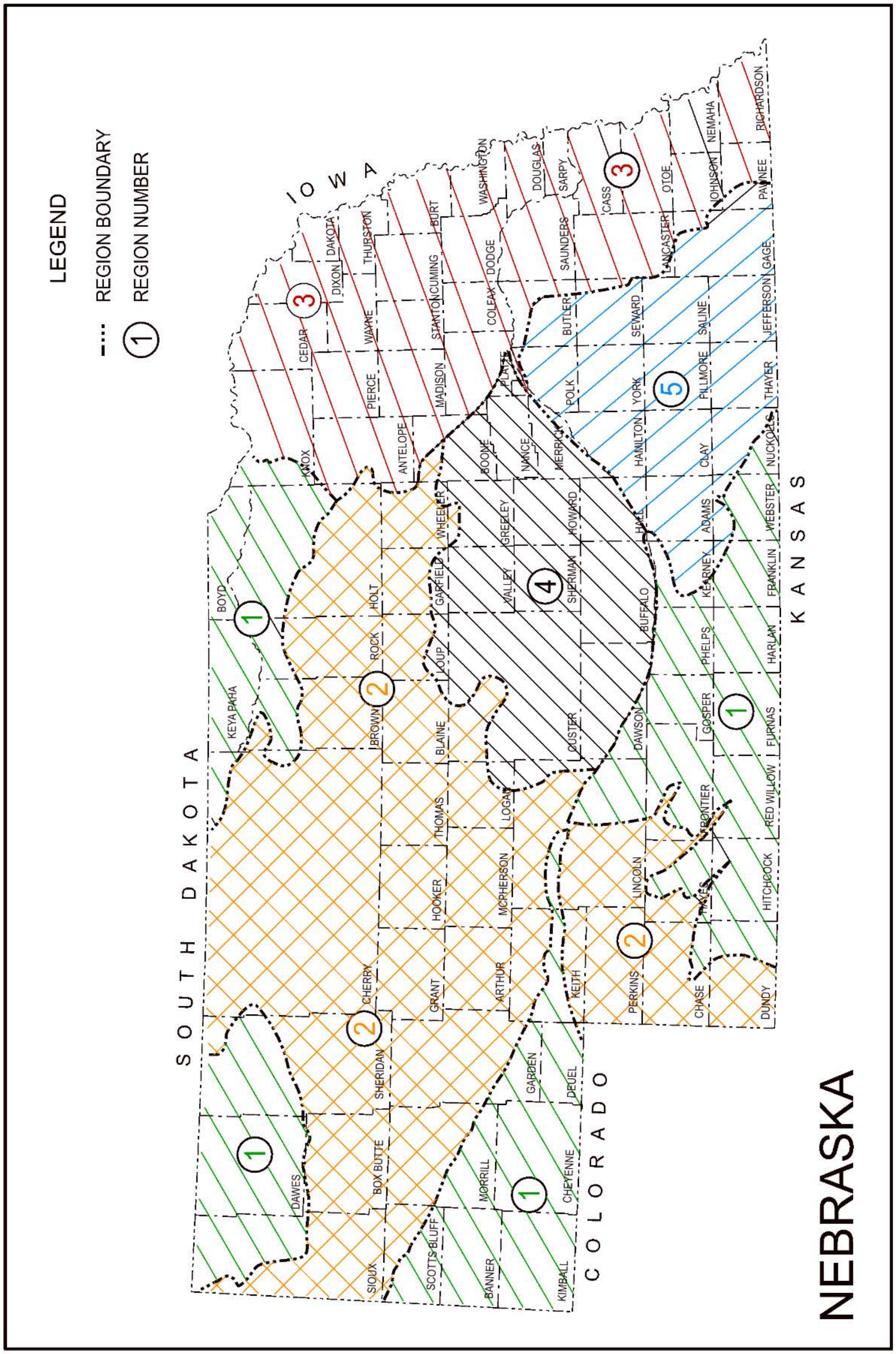


Exhibit 1.12 USGS Hydrologic Regions of Nebraska for Use with Regression Equation Method (640 Ac. or Greater) (Ref. 1.6)

Return Period	Region 1	Region 2
2 10 50 100 200 500	$Q_2 = 1.965\ A_c^{0.493}\ (P-13)^{1.44}$ $Q_{10} = 211.7\ A_c^{0.324}\ (P-13)^{0.314}$ $Q_{50} = 6366\ A_c^{0.211}\ (P-13)^{-0.630}$ $Q_{100} = 23553\ A_c^{0.170}\ (P-13)^{-1.011}$ $Q_{200} = 82183\ A_c^{0.131}\ (P-13)^{-1.382}$ $Q_{500} = 400713\ A_c^{0.082}\ (P-13)^{-1.863}$	$Q_2 = 0.269\ A_c^{0.912}\ S^{0.967}\ SN10^{2.337}$ $Q_{10} = 0.109\ A_c^{0.9917}\ S^{1.653}\ SN10^{2.607}$ $Q_{50} = 0.0845\ A_c^{1.036}\ S^{2.005}\ SN10^{2.632}$ $Q_{100} = 0.0816\ A_c^{1.051}\ S^{2.119}\ SN10^{2.615}$ $Q_{200} = 0.0816\ A_c^{1.064}\ S^{2.216}\ SN10^{2.587}$ $Q_{500} = 0.0844\ A_c^{1.079}\ S^{2.326}\ SN10^{2.536}$
	Region 3	**Region 4**
2 10 50 100 200 500	$Q_2 = 7.57x10^{-10}\ A_c^{0.815}\ S^{0.599}\ P^{7.099}$ $Q_{10} = 2.55x10^{-8}\ A_c^{0.722}\ S^{0.505}\ P^{6.657}$ $Q_{50} = 8.19x10^{-7}\ A_c^{0.688}\ S^{0.492}\ P^{5.908}$ $Q_{100} = 3.26x10^{-6}\ A_c^{0.681}\ S^{0.497}\ P^{5.581}$ $Q_{200} = 1.37x10^{-5}\ A_c^{0.677}\ S^{0.504}\ P^{5.226}$ $Q_{500} = 9.20x10^{-5}\ A_c^{0.673}\ S^{0.516}\ P^{4.740}$	$Q_2 = 341.4\ A_c^{0.443}\ L^{0.126}\ (T_3-43)^{-2.062}$ $Q_{10} = 4741\ A_c^{0.914}\ L^{-0.783}\ (T_3-43)^{-1.960}$ $Q_{50} = 19516\ A_c^{1.285}\ L^{-1.411}\ (T_3-43)^{-1.903}$ $Q_{100} = 31008\ A_c^{1.433}\ L^{-1.648}\ (T_3-43)^{-1.876}$ $Q_{200} = 46677\ A_c^{1.573}\ L^{-1.871}\ (T_3-43)^{-1.850}$ $Q_{500} = 75811\ A_c^{1.752}\ L^{-2.148}\ (T_3-43)^{-1.819}$
	Region 5	
2 10 50 100 200 500	$Q_2 = 0.00137\ A_c^{0.790}\ S^{0.777}\ I_{24,2}^{8.036}$ $Q_{10} = 0.00126\ A_c^{0.687}\ S^{0.683}\ I_{24,2}^{10.037}$ $Q_{50} = 0.00240\ A_c^{0.632}\ S^{0.640}\ I_{24,2}^{10.467}$ $Q_{100} = 0.00335\ A_c^{0.615}\ S^{0.628}\ I_{24,2}^{10.491}$ $Q_{200} = 0.00464\ A_c^{0.599}\ S^{0.618}\ I_{24,2}^{10.490}$ $Q_{500} = 0.00755\ A_c^{0.581}\ S^{0.606}\ I_{24,2}^{10.393}$	

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 Ref. 1.7 (TechnicalPaper_No40.pdf)
* 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: Ref. 1.6)

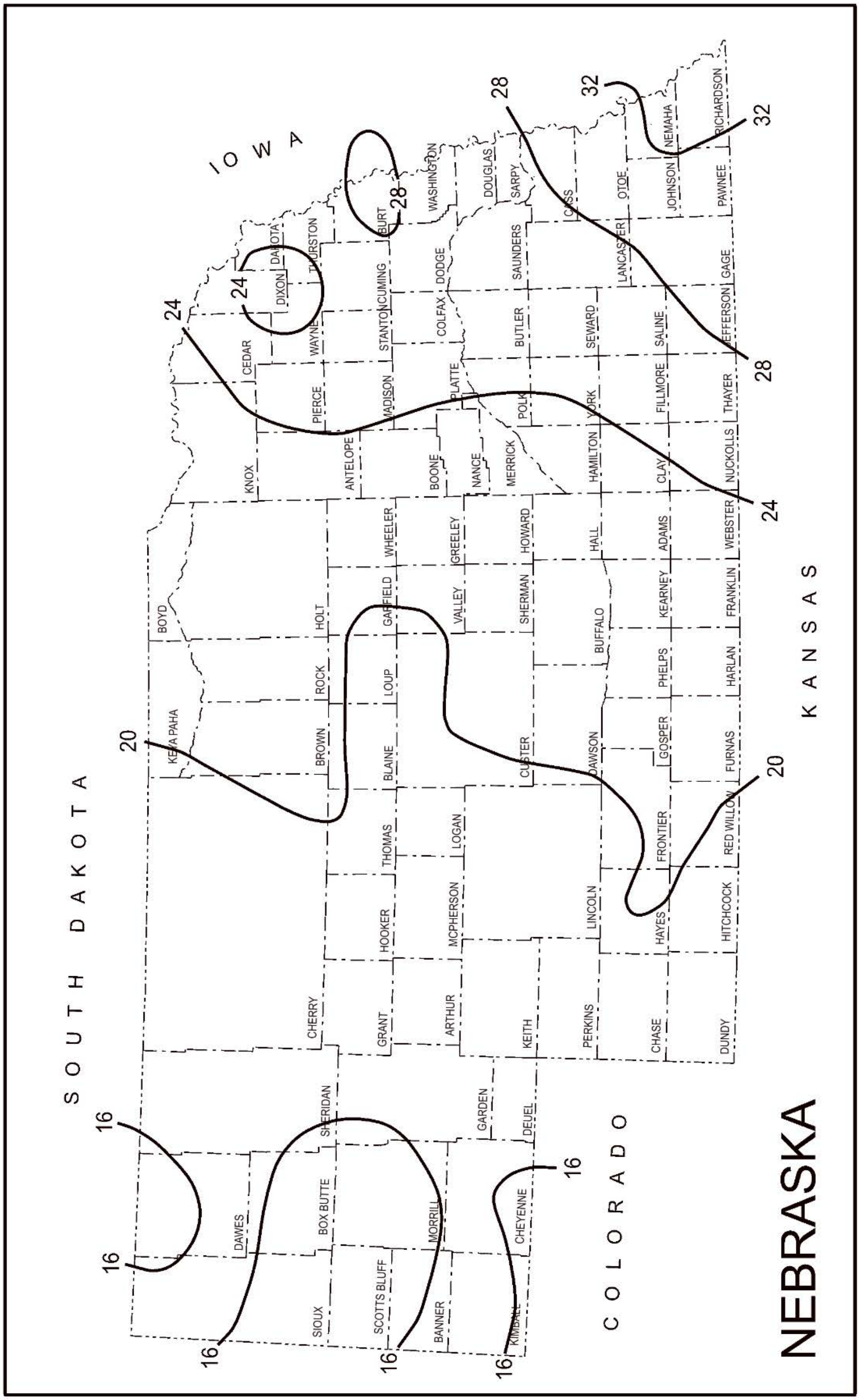


Exhibit 1.14 Mean Annual Precipitation (inches) (640 Ac. or Greater) (Ref. 1.7)

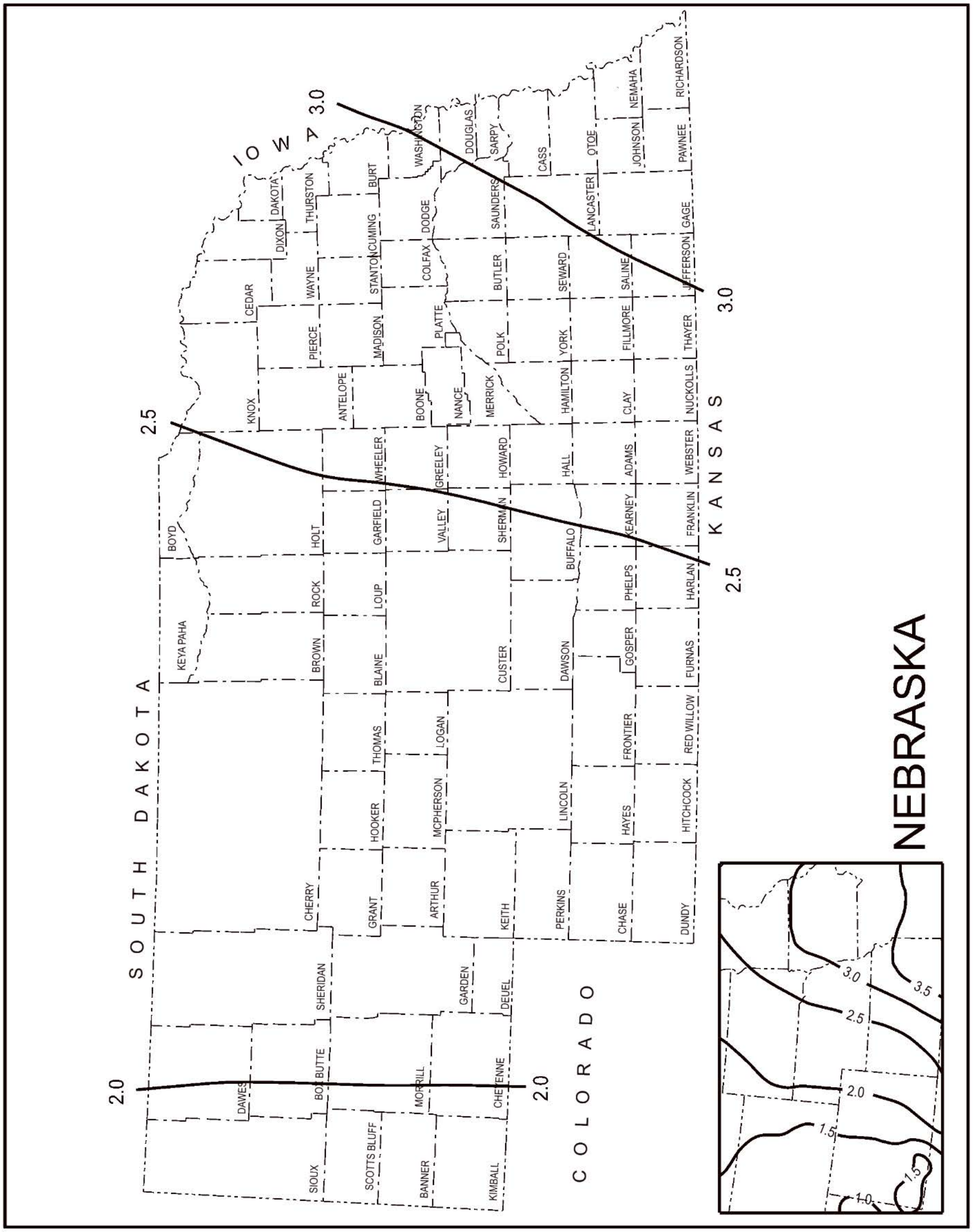


Exhibit 1.15 2-Year, 24-Hour Rainfall (inches) (640 Ac. or Greater)
(Ref. 1.7)

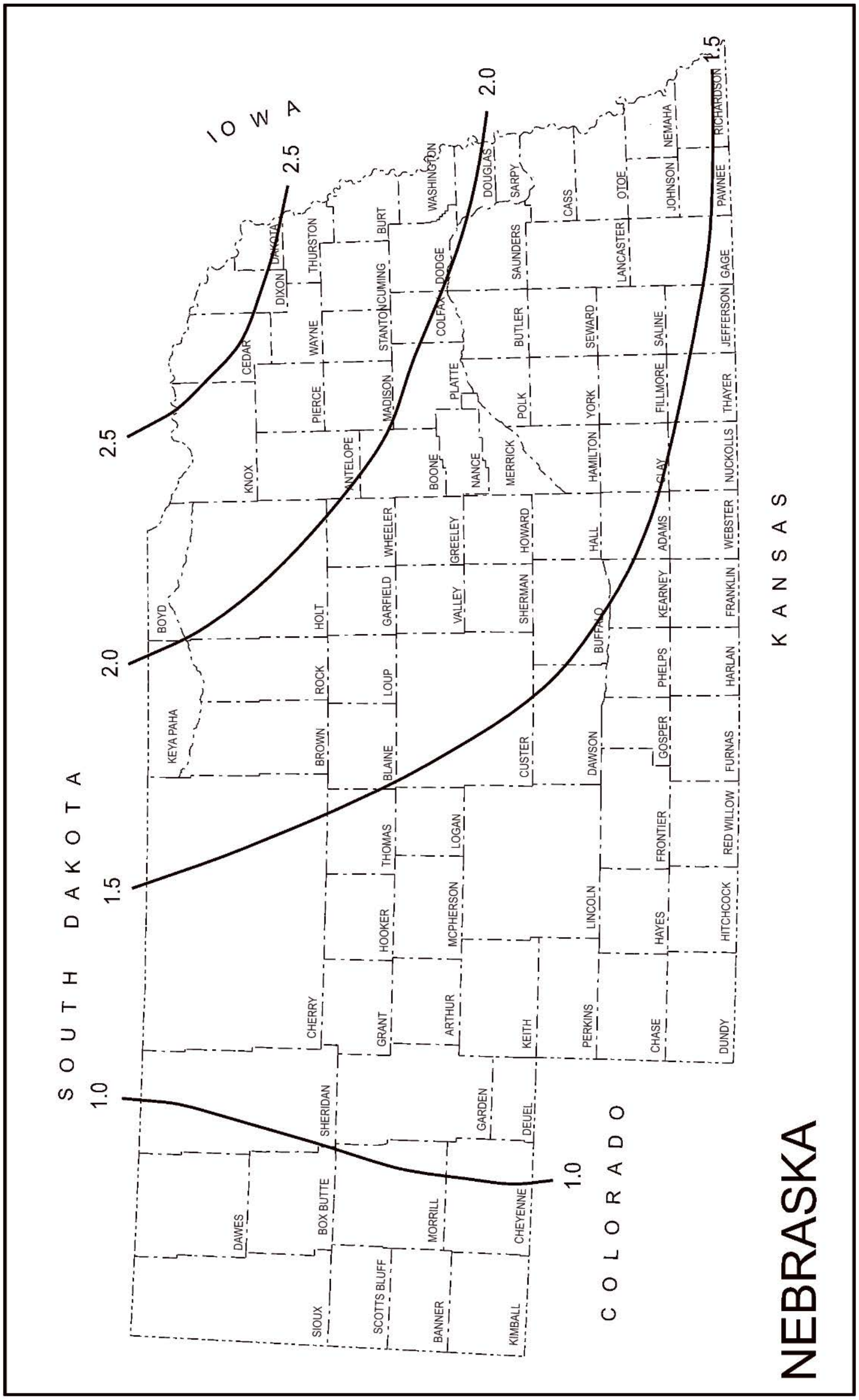


Exhibit 1.16 10% Probability-Equivalent Moisture Content of Snow as of March 15 (inches) (640 Ac. or Greater) (Ref. 1.7)

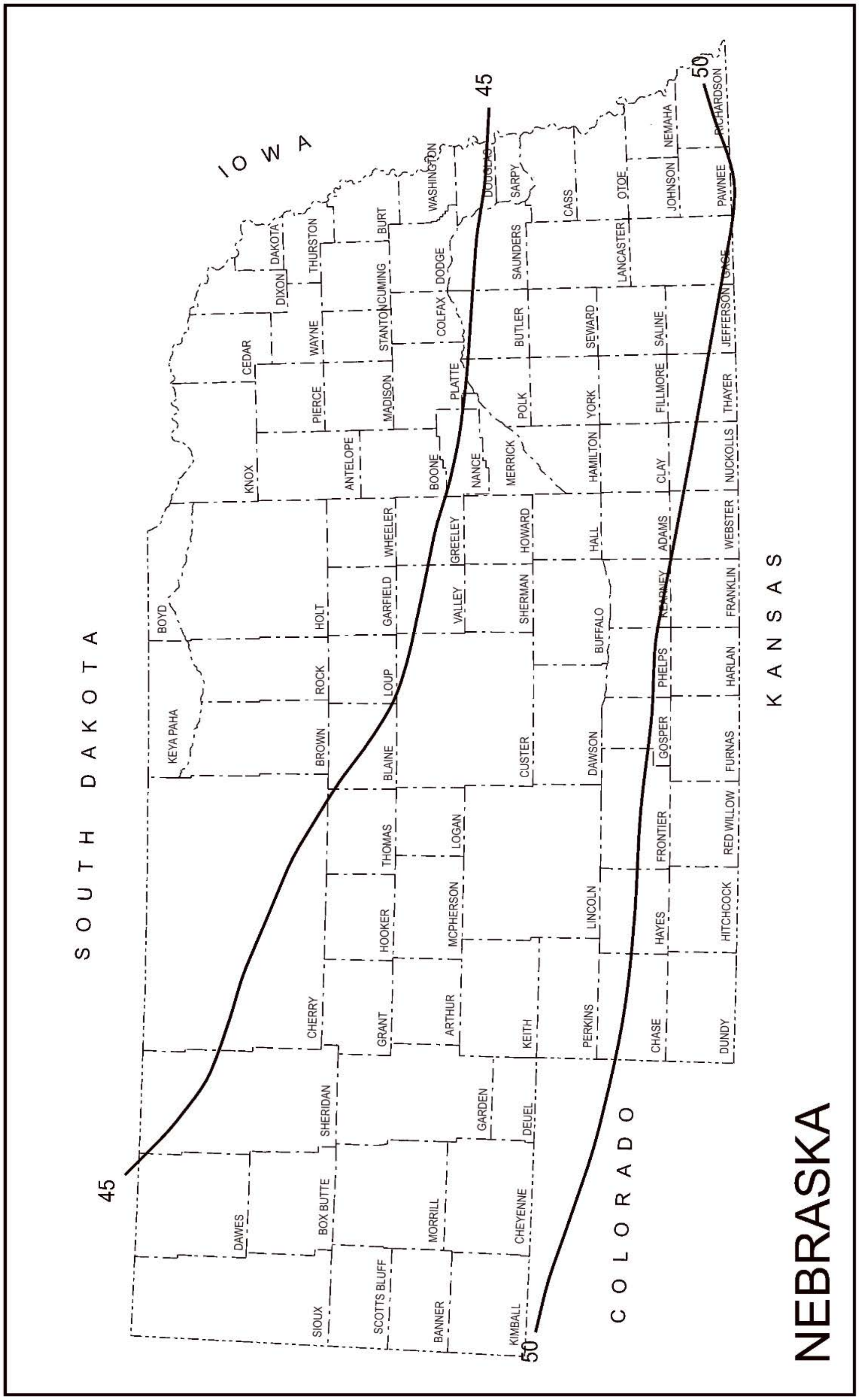


Exhibit 1.17 Normal Daily March Temperature (ºF) (640 Ac. or Greater)
(Ref. 1.7)

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 roadway designer should consult Hydraulic Design Series 4: Design of Roadside Drainage Channels (Ref. 1.8, https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds4.pdf), Water Surface Profiles, User Manual (HEC2) (Ref. 1.9, HEC-2 Water Surface Profiles), and Hydraulic Design Series 3: Design Charts for Open-Channel Flow (Ref. 1.10, https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds3.pdf) for detailed explanations of specialized procedures and methods pertaining to open channel hydraulics.

The roadway 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 consult with the **Bridge Division Hydraulics Engineer** and see the *RDM* (Ref. 1.3), Chapter Thirteen: Planning and Project Development, Section 5.B.6.

7.B Types of Open Channel Flow

Open channel flow is classified as:

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

Non-uniform, unsteady, subcritical flow is the most common type of flow in open channels. 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. 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:

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

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 representing the ratio of inertial to gravitational forces. It is defined by the following equation:

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

where: Fr = Froude Number
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)

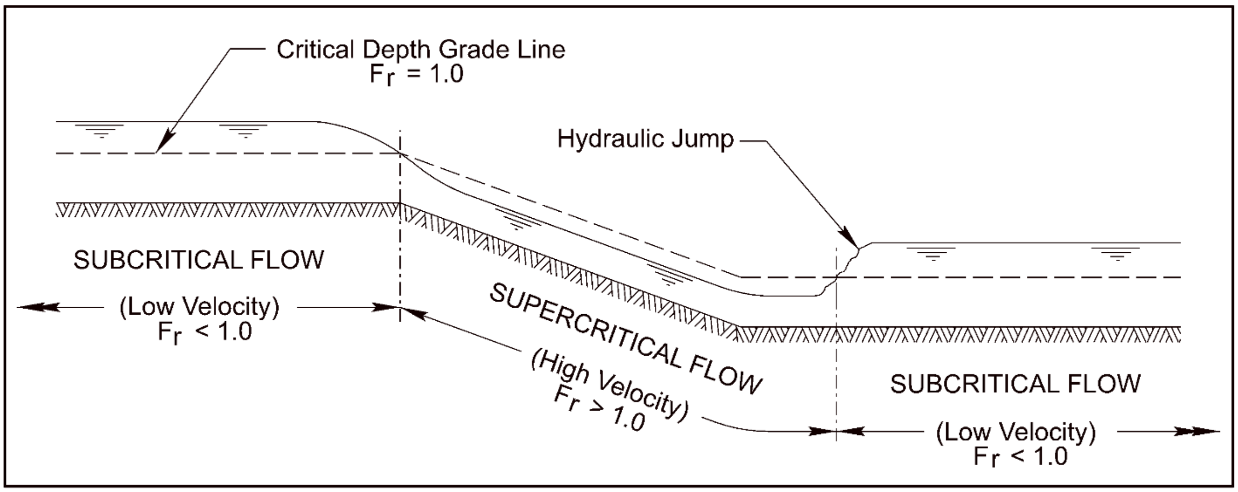


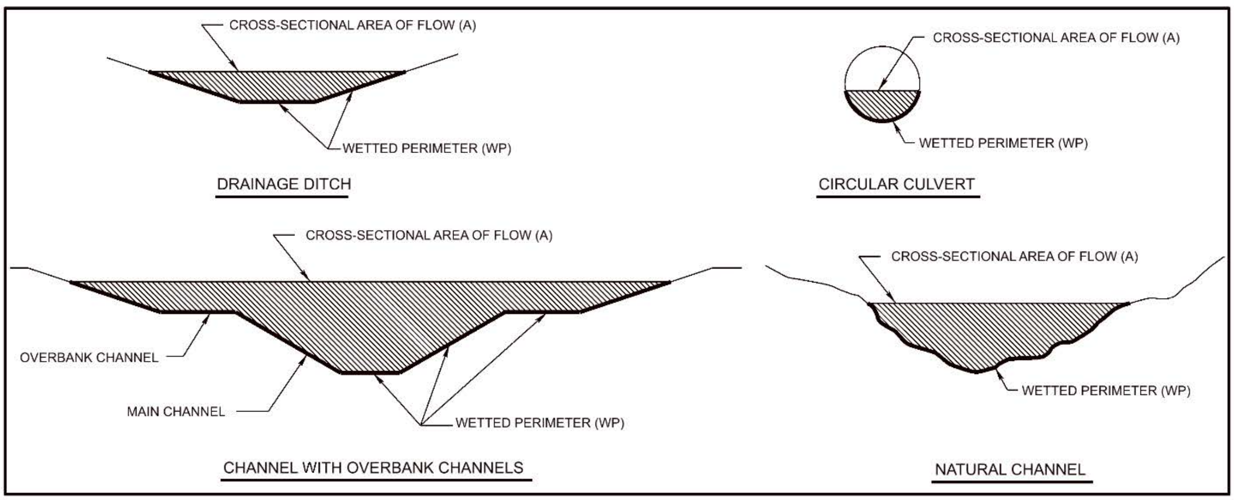
Exhibit 1.18 Water Surface Profile Illustrating Hydraulic Jump

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:

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

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", in this manual.

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 (Ref. 1.1) and Hydraulic Design Series No. 3: Design Charts for Open Channel Flow (Ref. 1.10).

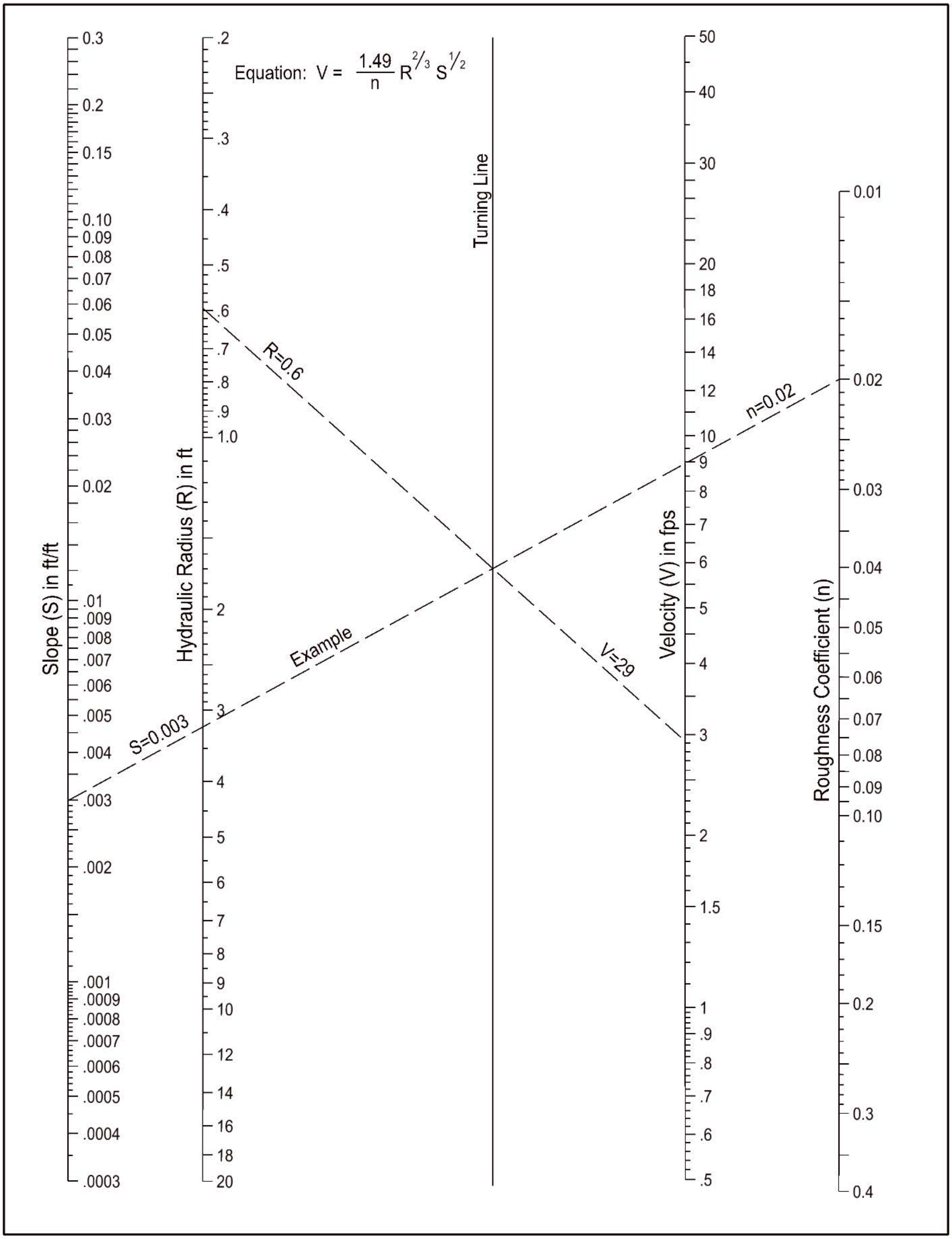


Exhibit 1.19 Nomograph for Solution of Manning's Equation, Open Channel

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** (**Bridge**) 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 (Ref. 1.11, Hydraulic Design of Highway Culverts - HDS-5 - Third Edition). 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 **DE**. 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 Chapter Seventeen: Resurfacing, Restoration and Rehabilitation (3R) Projects, Section 17.C, of the *RDM* (Ref. 1.3).

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. Additional information is required if the culvert is located in a floodplain (See Section 5.C of this chapter). Contact the **Bridge Division 408 & 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", in this manual show a culvert design checklist and a broken-back culvert checklist. Roadway 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

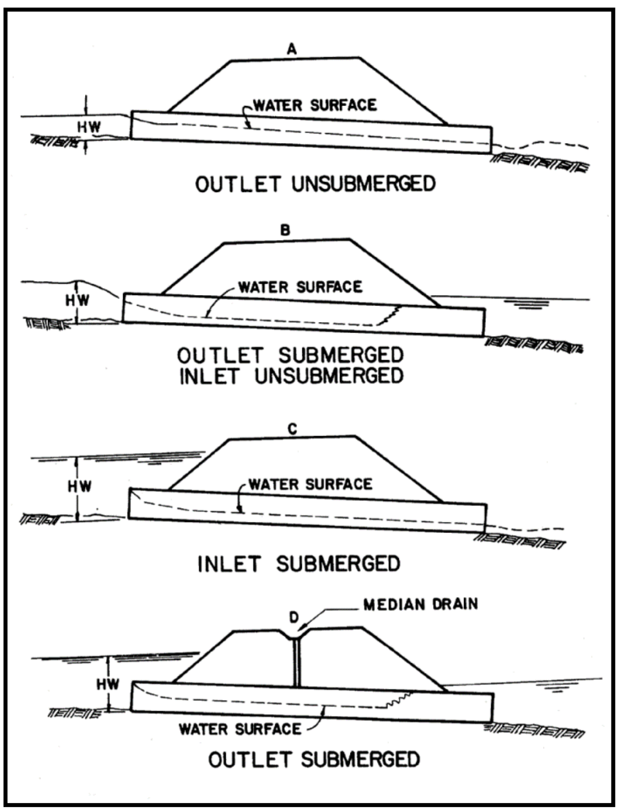
Different factors and formulas are used to compute the hydraulic capacity of a culvert for each type of control. Hydraulic analysis of a culvert design includes determining the headwater elevation at the design discharge (See Section 8.A of this chapter). This is done by comparing the inlet control headwater 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 (Ref. 1.11).

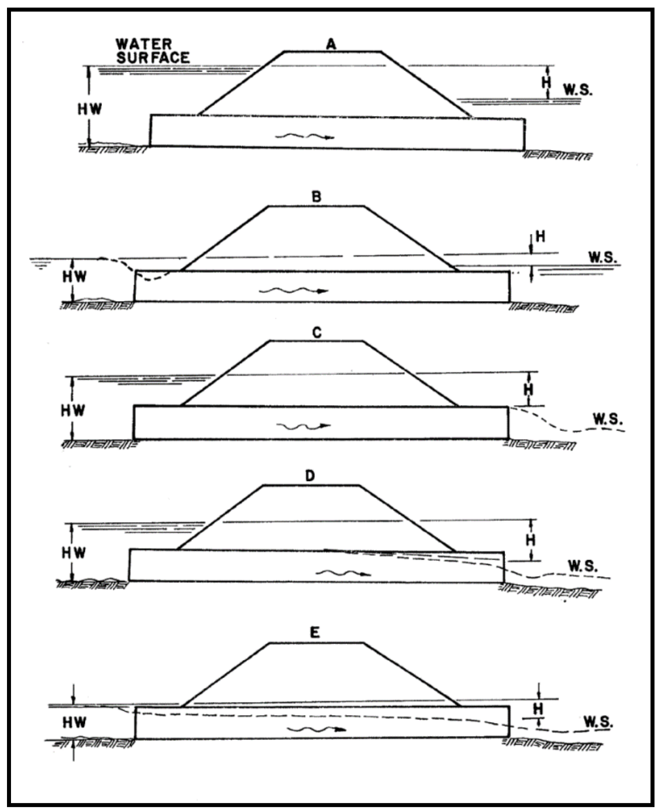
8.B.1 Inlet Control

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 of this chapter and Hydraulic Design Series 5: Hydraulic Design of Highway Culverts, Ref. 1.11).

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: Ref. 1.11)

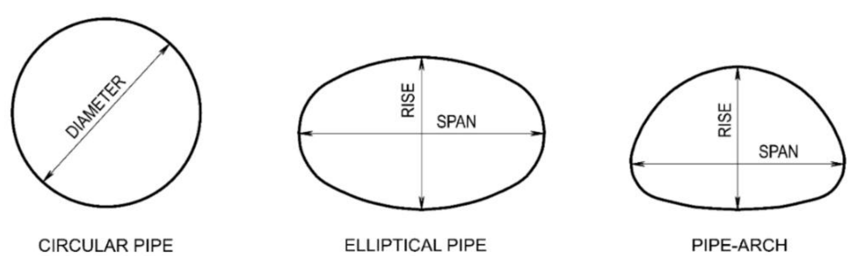
8.C Culvert Type, Material and Location

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", in this manual 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
Elliptical	Used where cover is limited. When desired size is not available, consider the use of a partially buried round pipe to meet the site conditions
Pipe arch	Used where cover is limited. Not readily available in Nebraska, consider the use of elliptical pipe or partially buried elliptical or round pipe to meet the site conditions
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	1.2	---	---	---	18	11	1.1	17	13	1.1
18	1.8	23	14	1.8	22	13½	1.6	21	15	1.6
21	2.4	---	---	---	---	---	---	24	18	2.2
24	3.1	30	19	3.3	28½	18	2.8	28	20	2.4
27	4.0	34	22	4.1	---	---	---	---	---	---
30	4.9	38	24	5.1	36¼	22½	4.4	35	24	4.5
33	5.9	42	27	6.3	---	---	---	---	---	---
36	7.1	45	29	7.4	43¾	$26^5/_8$	6.4	42	29	6.5
42	9.6	53	34	10.2	$51^1/_8$	$31^5/_{16}$	8.8	49	33	8.9
48	12.6	60	38	12.9	58½	36	11.4	57	38	11.6
54	15.9	---	---	---	65	40	14.3	64	43	14.7
60	19.6	76	48	20.5	73	45	17.7	71	47	18.1
66	23.8	83	53	24.8	---	---	---	77	52	21.9
72	28.3	91	58	29.5	88	54	25.6	83	57	26.0

* Not readably available in Nebraska, expensive to bring in from out of State.

Notes: Dimensions do not include the wall thickness.
Refer to manufacturer's literature for larger pipe sizes.
Not all pipe sizes are commonly available, contact the supplier(s) for available 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 and shall be paid for by the "Lin. Ft." (*Spec Book* (Ref. 1.16), Section 718.05)
- Circular concrete pipe should be designed in increments of 4 or 8 feet. This will minimize the need to cut pipe.
- Concrete elliptical pipe is only available in 8-foot lengths.
- If circumstances dictate, concrete pipe may be designed to the next largest whole foot.
- 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 and will be paid for by "Each" (*Spec Book* (Ref. 1.16), Section 724.05)
- Box culvert lengths shall be specified to the nearest ft. The pay items for box culverts are "Excavation for Structures", "Concrete Construction", and "Reinforcement" (*Spec Book* (Ref. 1.16), Table 717.01)
- 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 *RDM* (Ref. 1.3), Chapter Four: Intersections, Driveways and Channelization, Section 2.A.1, for the driveway culvert length policy.

	Diameter of Elbow (in.)	**Length Allowance (lin. ft.)**
For Both Concrete and CMP	Less than 36 in. round or equivalent pipe arch 36 in. x 22 in.	7
	36 in. - 54 in. round or equivalent pipe arch 43 in. x 27 in. to 65 in. x 40 in.	6
	Over 54 in. round or equivalent pipe arch 65 in. x 40 in.	5
	Diameter of Connecting Band (in.)	**Length Allowance (lin. ft.)**
CMP Only	21 in. or less round or equivalent pipe	3
	Over 21 in. round or equivalent pipe	4

Exhibit 1.23 Additional Pay Length for Elbows and Connecting Bands
(Source: Standard Specifications for Highway Construction, (*Spec Book*), Ref. 1.16, Tables 718.01 & 718.02, https://dot.nebraska.gov/media/g4qp4y0d/2017-specbook.pdf)

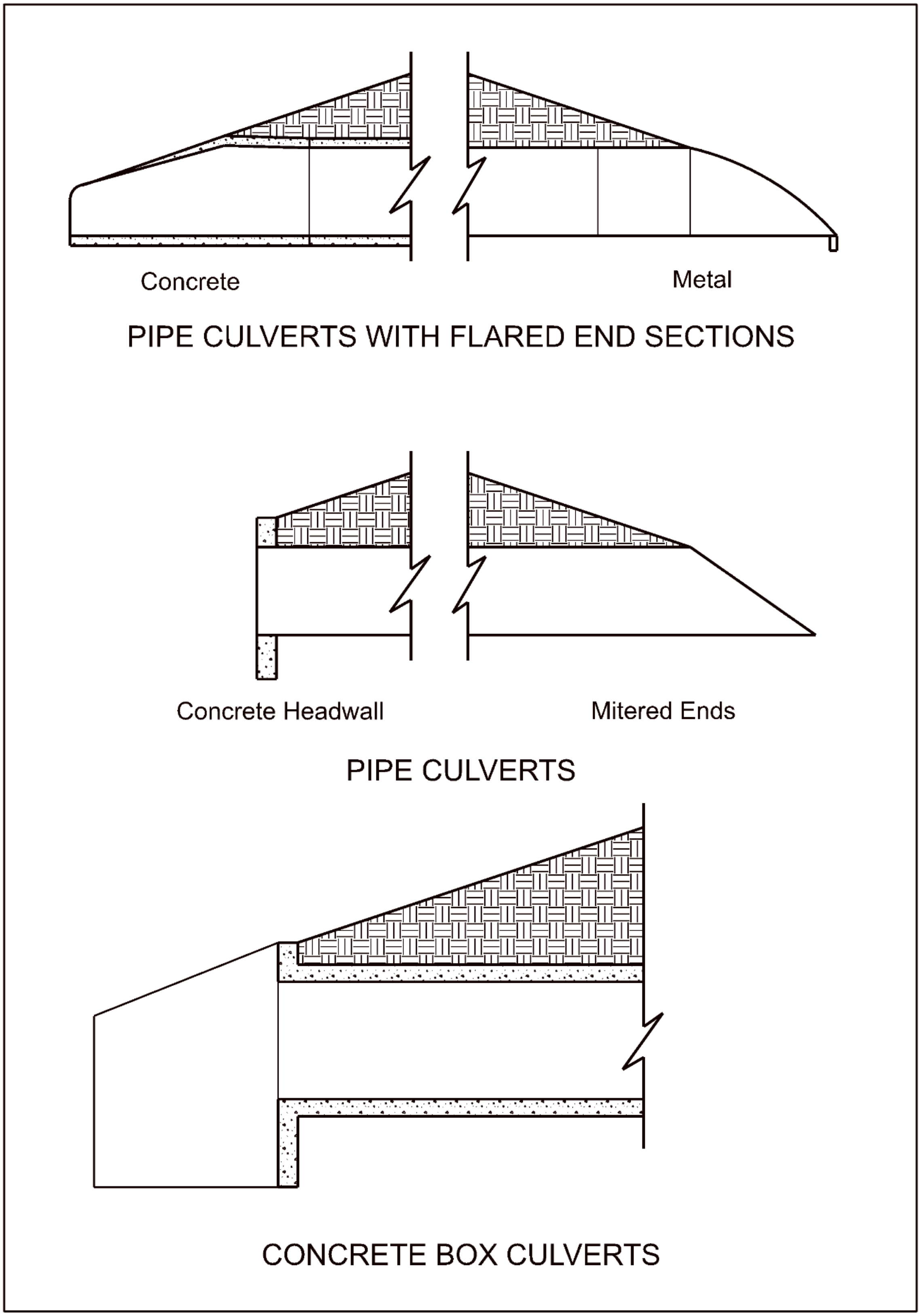


Exhibit 1.24 Side Slope Grading for Culverts

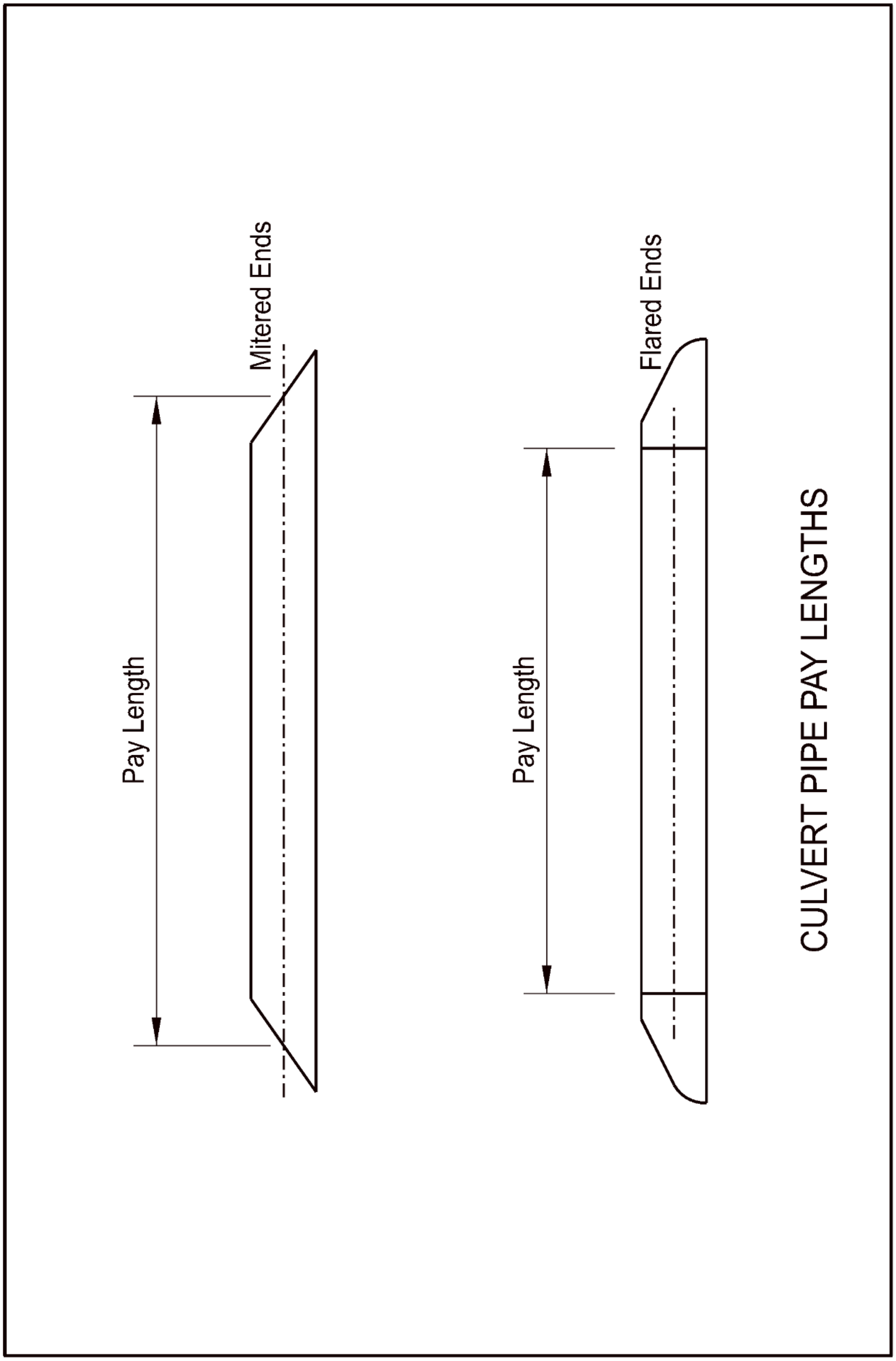


Exhibit 1.25 Pay Lengths for Culvert Pipe

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 tend to catch debris and clog the waterway and may also be 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. The use of multiple pipes should be avoided whenever possible; 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 (i.e., outside of the clear zone). Flared end sections are also 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 shall generally 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 will be designed in accordance with special plans requested from **Bridge** and may be used for:

- Multiple pipe installations
- Culverts with skews of 30° or over
- Culverts with slopes too steep for a flared end section
- 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 **Bridge**.

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. The potential for damage to adjacent property is greater in urban areas because of the number and value of properties that can be affected. Additional areas of concern in the design of culverts are:

- 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 (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 roadway 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 roadway designer an opportunity to use a high headwater or ponding to attenuate flood peaks. If high headwater is necessary, the roadway designer should investigate the need to acquire additional right-of-way. If deep ponding is considered, the possibility of catastrophic failure should be investigated as 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 **Bridge Division 408 & Roadway Design Hydraulic Engineer**.

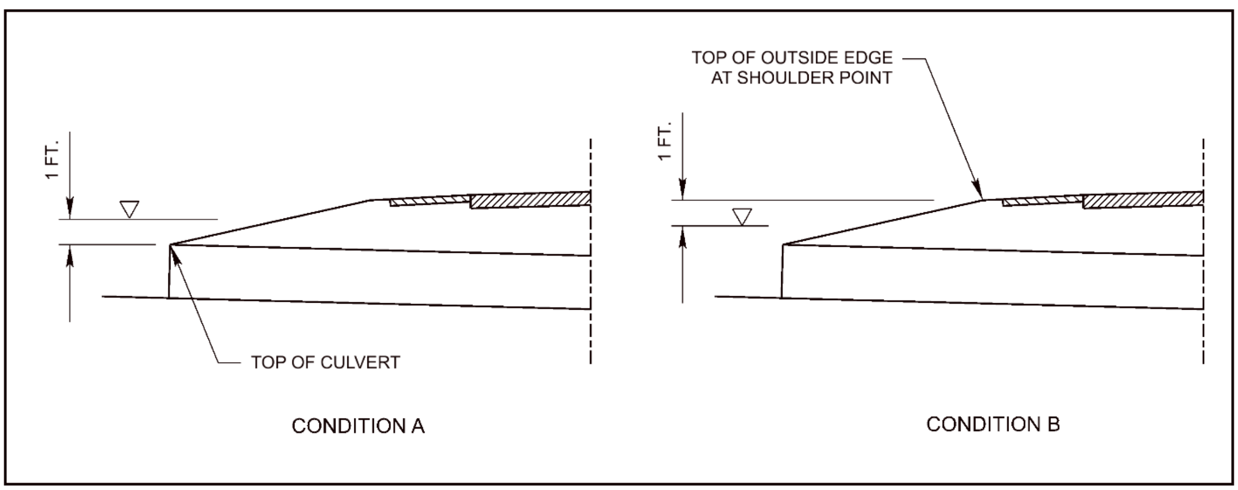


Exhibit 1.26 Maximum Allowable Headwater

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 (Ref. 1.11) or from computer programs. For the input consisting of hydrological and topographical data, these programs assist the roadway 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. See Chapter Two: Erosion and Sediment Control, Sections 6 and 7 of this manual, 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 Longitudinal	 15 in. 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”, in this manual. 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 of this chapter.

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

Entrance head loss: $H_e = k_e(V^2 \div 2g)$

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
[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", in this manual.	
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

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; 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 of this chapter) 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, 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 Design Series 5: Hydraulic Design of Highway Culverts (Ref. 1.11). 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

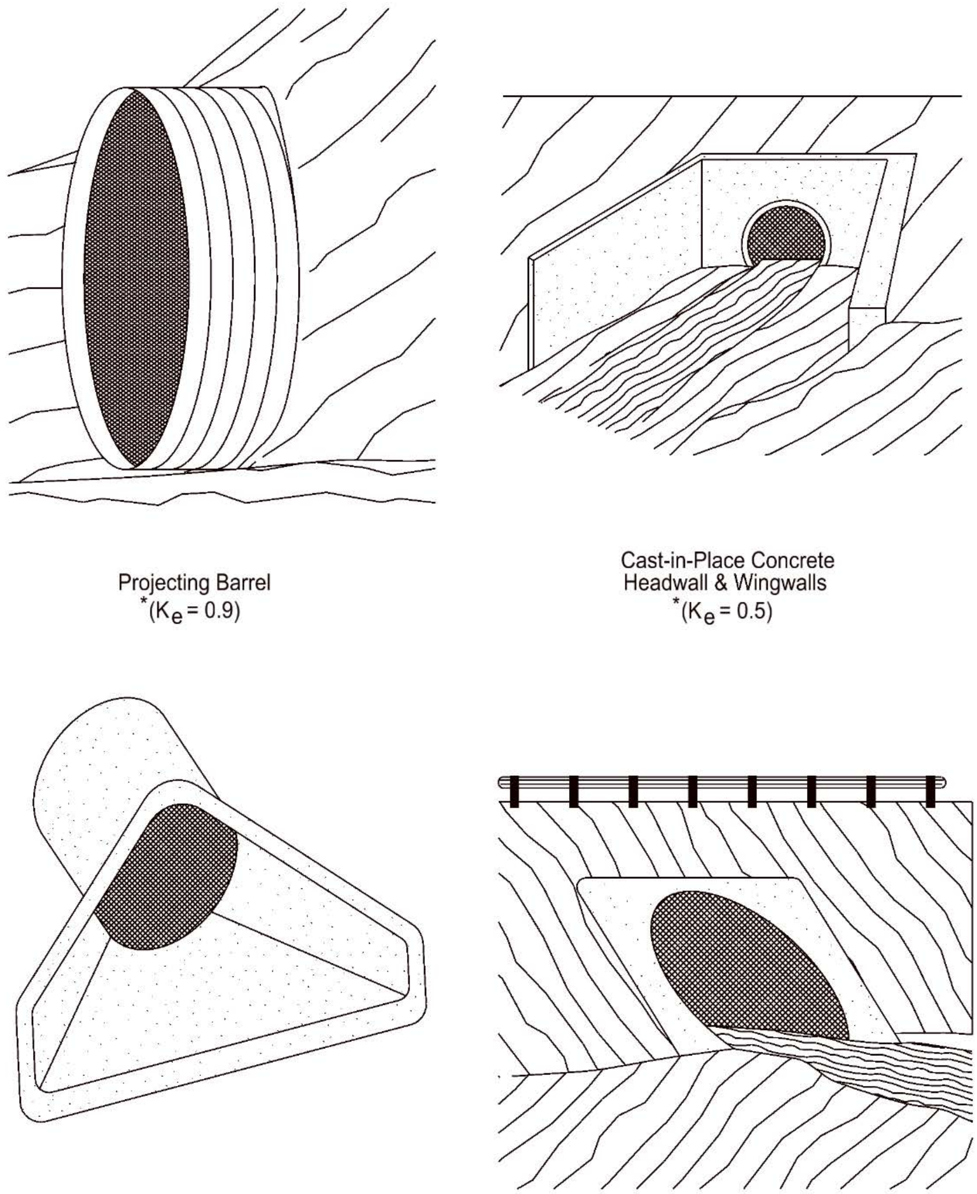
A standard code sheet shall be used when ordering a special design from **Bridge** for the improved inlet.

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:

- 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: Ref. 1.11)

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 roadway designer should consult Hydraulic Design Series 5: Hydraulic Design of Highway Culverts (Ref. 1.11) for detailed discussion and design procedures for improved side tapered and slope tapered inlets.

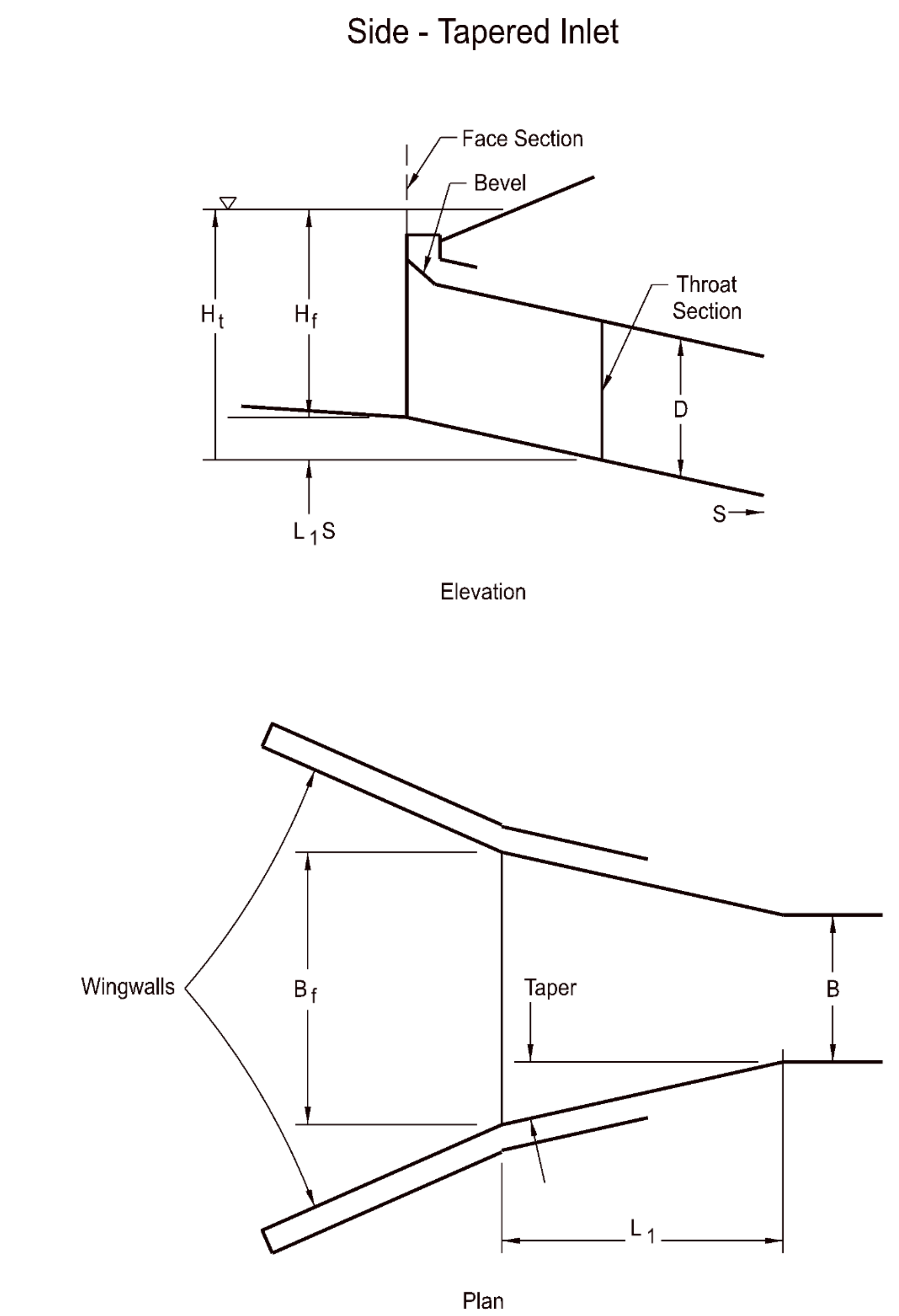


Exhibit 1.30 Side-Tapered Inlets (Source: Ref. 1.11)

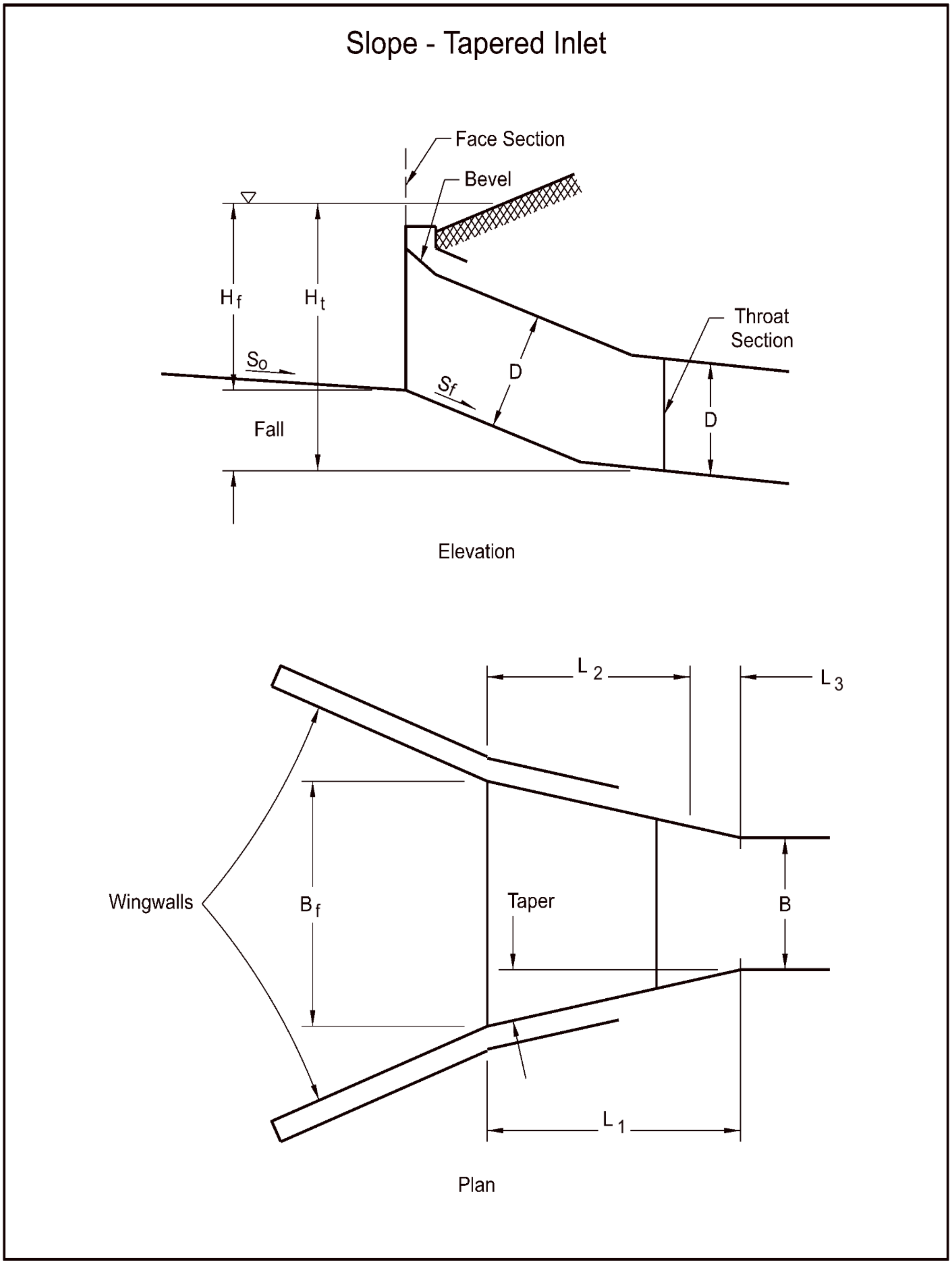


Exhibit 1.31 Slope-Tapered Inlets (Source: Ref. 1.11)

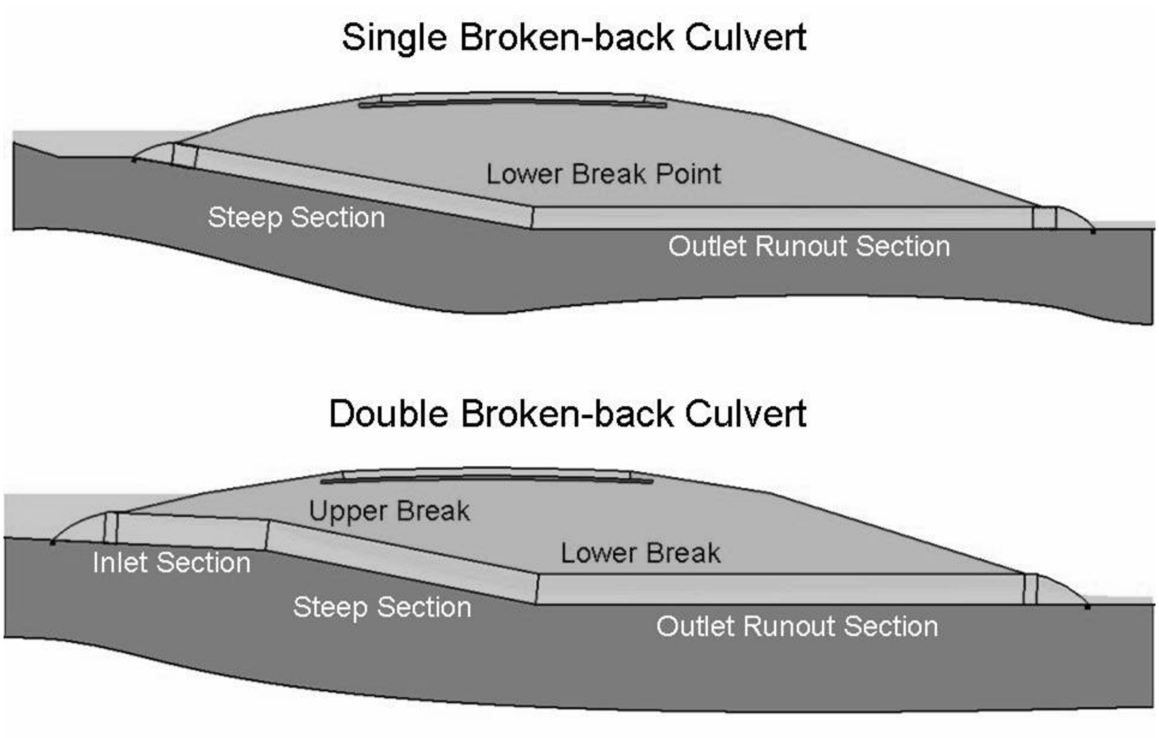
8.L Special Hydraulic Considerations

In addition to the hydraulic considerations discussed in the preceding sections, the roadway 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 of this chapter)
- Irregular profile and alignment (See Section 8.L.2 of this chapter)
- Anchorage (See Section 8.L.4 of this chapter)

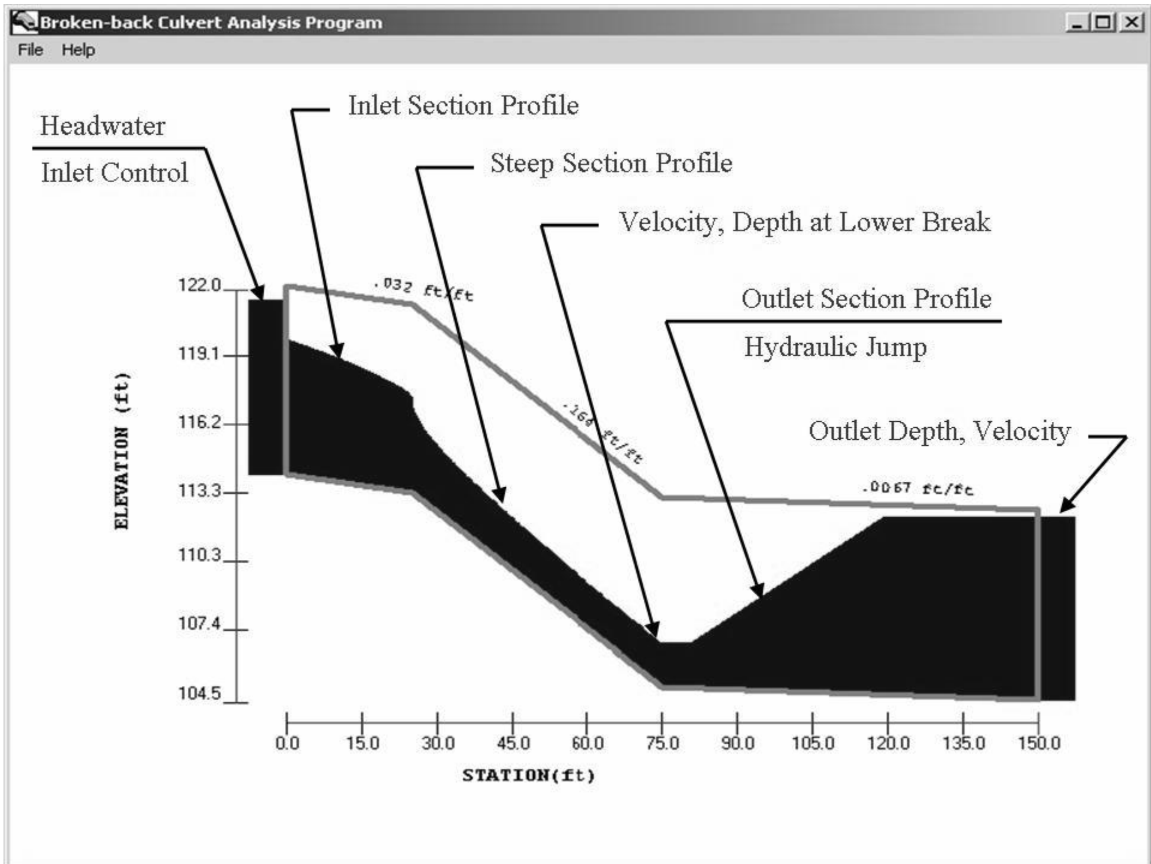
8.L.1 Broken-Back Culverts

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 **NDOT** (https://dot.nebraska.gov/business-center/design-consultant/custom-apps/). 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 built to control head cut erosion when there is great differential between the inlet and outlet elevations. In broken-back culverts the 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. Whenever possible, pipe material with a Manning's n-value of 0.022 or greater should be used for the outlet runout section of broken-back culverts.

8.L.2 Irregular Profile and Alignment

EXHIBIT 1.32 shows slope adjustments for skewed angles (See the *RDM* (Ref.1.3) Chapter Ten: Miscellaneous Design Issues, Section 2.B). 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 1:6, the resultant slope is 1:6.62.

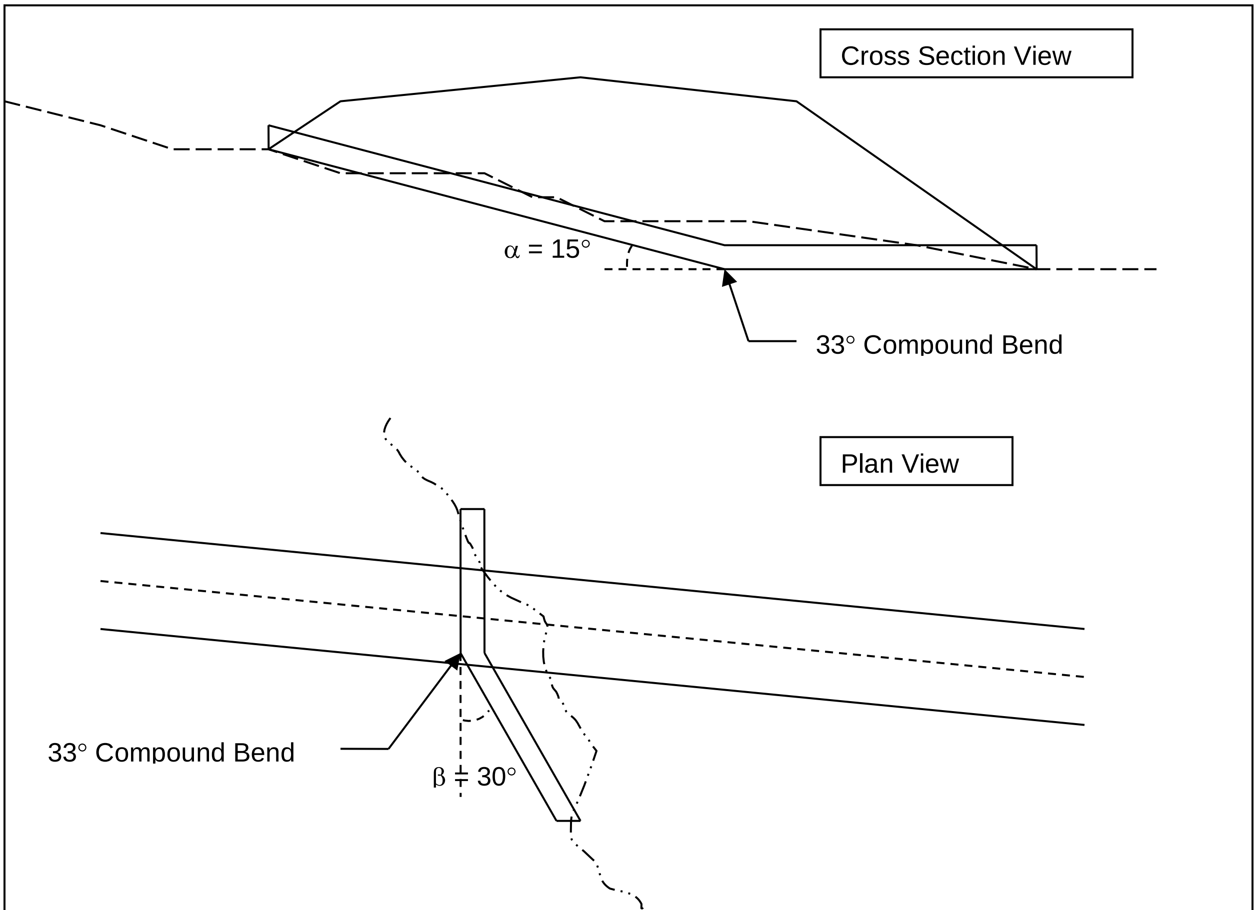
Skew Angle	**Foreslope**				
	1:1.5	1:2	1:3	1:4	1:6
5°	1.51	2.01	3.01	4.02	6.02
10°	1.52	2.03	3.05	4.06	6.10
15°	1.55	2.07	3.11	4.14	6.22
20°	1.60	2.13	3.19	4.26	6.38
25°	1.66	2.21	3.31	4.41	6.62
30°	1.73	2.31	3.46	4.62	6.92
35°	1.83	2.44	3.66	4.88	7.32
40°	1.96	2.61	3.92	5.22	7.84
45°	2.12	2.83	4.24	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°- Cos^{-1}(- cos(α) x cos(β)) Eq. 1.6



Calculation:

Cos(15°) = 0.966 ; Cos(30°) = 0.866

Cos^{-1}(-0.966 x 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 inlet and outlet 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 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. 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.

The inclusion of anchorage as a pay item should be discussed with the **Assistant Design Engineer** (**ADE**).

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 roadway 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 either 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 roadway designer should consult Hydraulic Design Series 5: Hydraulic Design of Highway Culverts (Ref. 1.11) for design of debris control structures.

8.N Corrosion and Abrasion

The durability of culvert material 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.

Abrasion is the erosion of culvert materials by sediment carried by streams.

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. Plastic pipe is superior to corrugated metal and concrete pipe in corrosion resistance. Refer to the **M&R** soil and situation report for soils information (See the *RDM* (Ref. 1.3) Chapter Seven: Earthwork, Section 8.A.3).

8.O Multiple-Use Culverts

Culverts 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 culvert will have a span of 5 ft. and a 7 ft. rise. The roadway 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. 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 roadway 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. For additional information see the *RDM* (Ref. 1.3), Chapter Ten: Miscellaneous Design Issues, Section 7.

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 roadway 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. For additional information see the *RDM* (Ref. 1.3), Chapter Sixteen: Pedestrian and Bicycle Facilities, Section 5.

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", in this manual). 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 (*Standard Plans*) (Ref. 1.12). (https://dot.nebraska.gov/business-center/design-consultant/stand-spec-manual/) 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. **Bridge** shall be contacted for a recommendation before extending a concrete 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 from the survey the roadway 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) 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 a 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 Design Detail Plans (See the *RDM* (Ref. 1.3), Chapter Eleven: <u>Highway Plans Assembly</u>, Section 1.C) and verify that 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", in this manual, request that **Bridge** perform a structural study and verify the structural adequacy of the existing culvert and bedding before extending the culvert.

Unless a problem is noted at the 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. will need to be cambered. The height of the camber required is based on the settlement and length of the structure, the roadway designer should work with the **Soils Engineer** in **M&R** to determine the appropriate values. Control joints will be used in concrete box culverts with fills over 10 feet when settlement is anticipated (See the *Standard Plans* (Ref. 1.12), Standard Plan 404).

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 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 *RDM* (Ref. 1.3), 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 flared end section(s) or a 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.

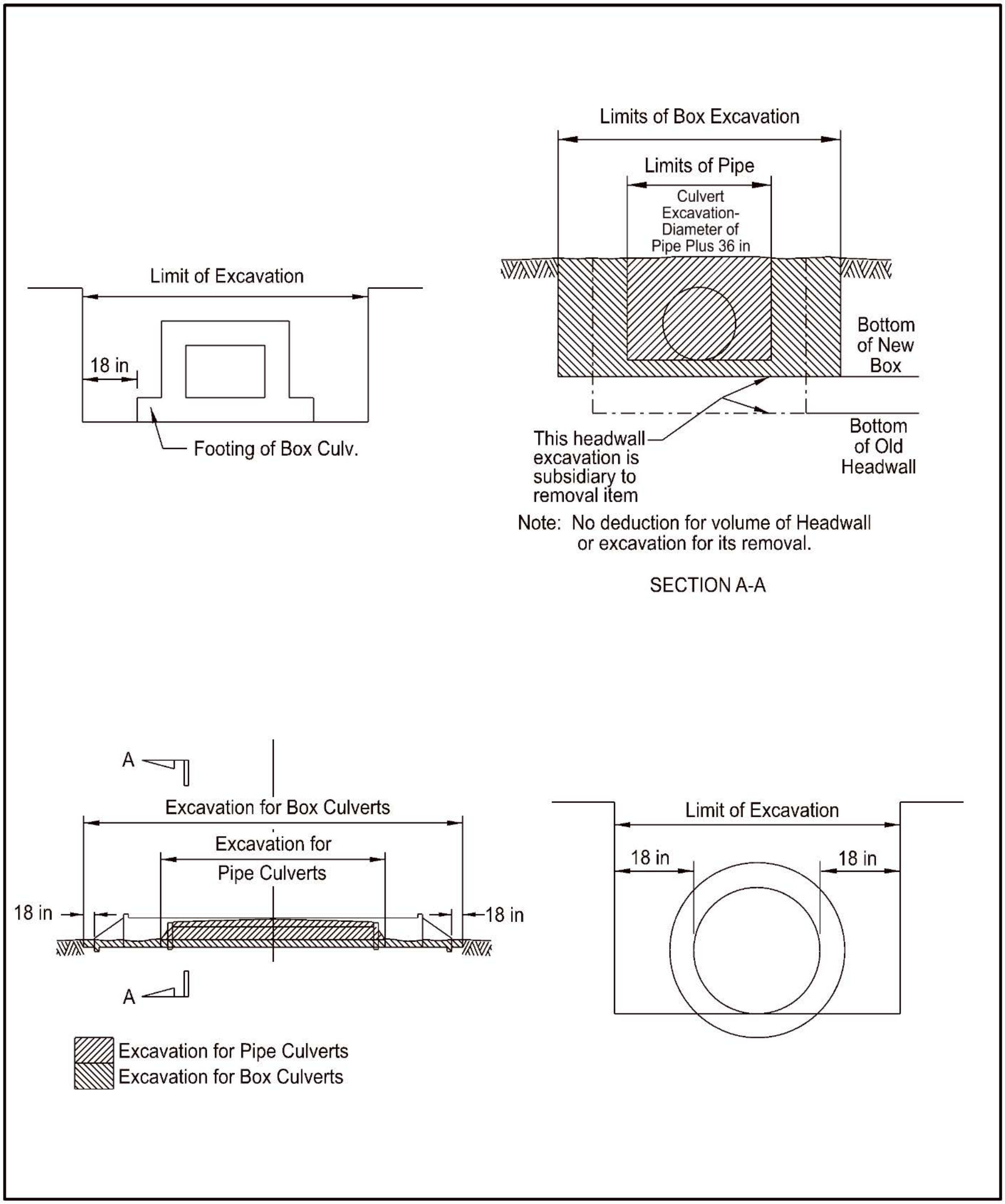


Exhibit 1.34 Separation of Excavation for Pipe Culverts and Headwalls, and Excavation for Box Culverts

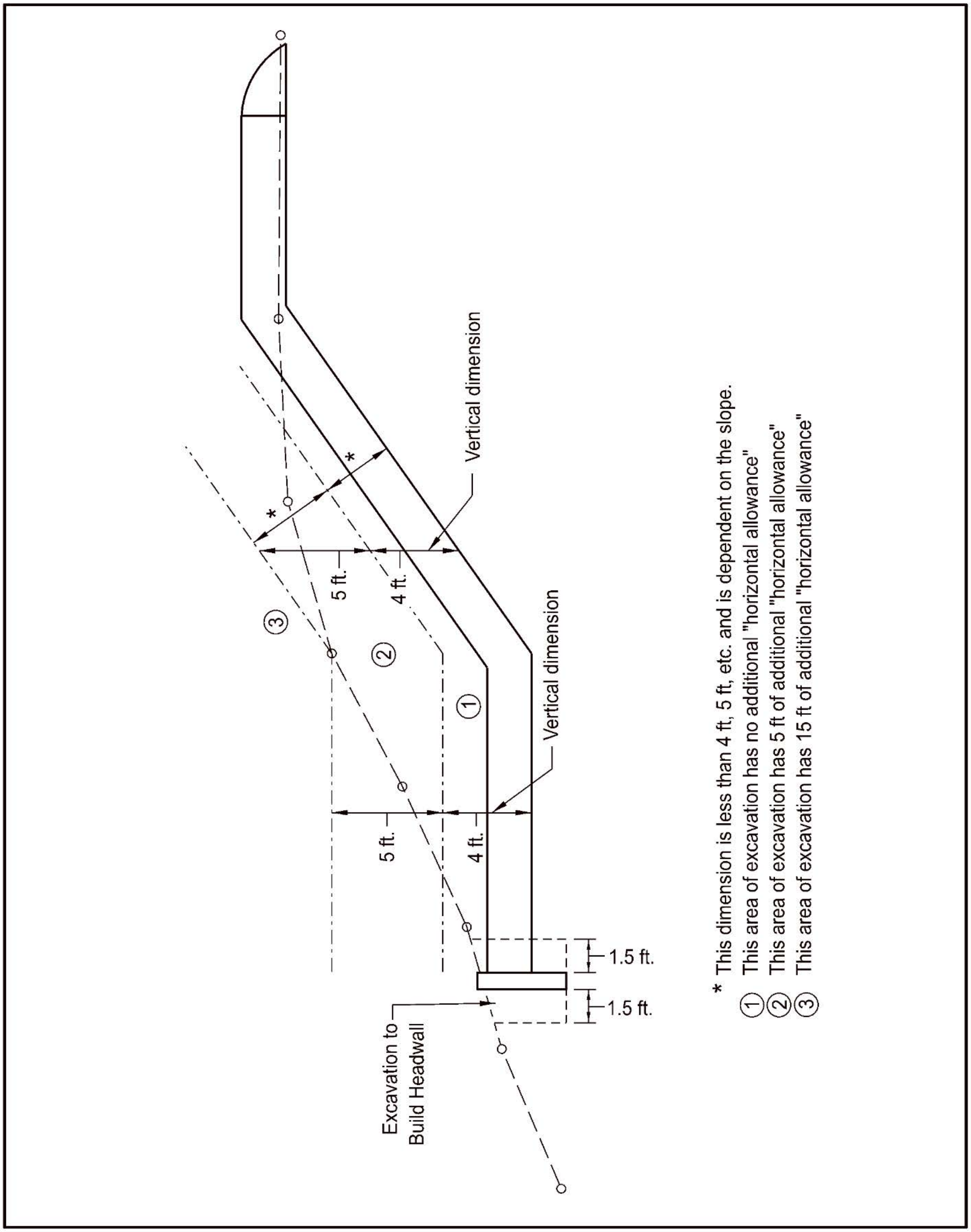


Exhibit 1.35 Excavation for Broken-Back Culverts

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

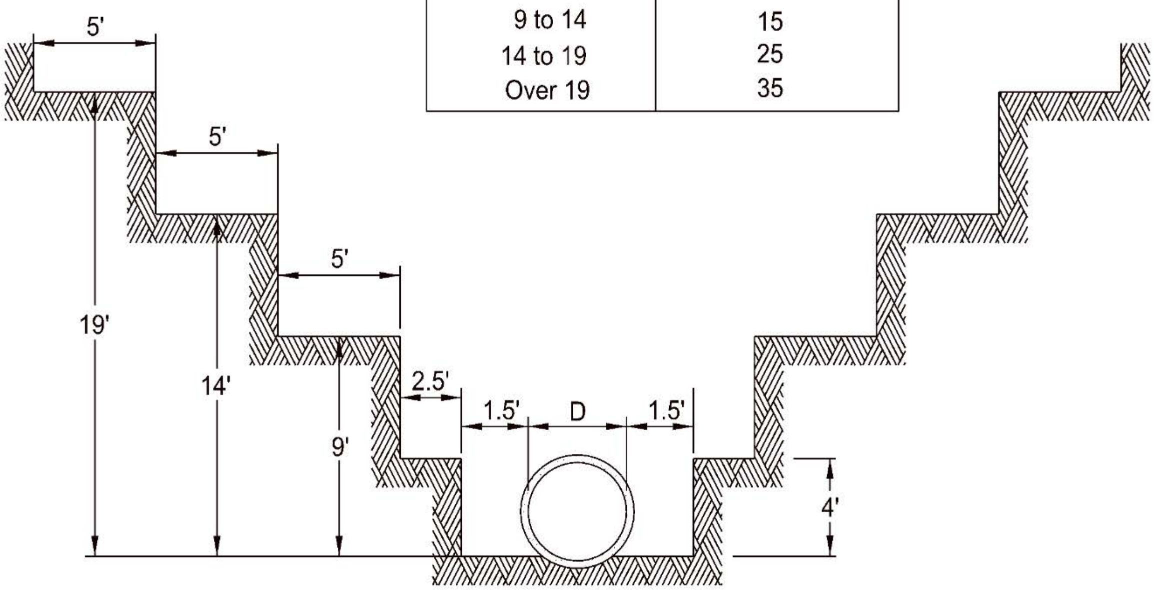


Exhibit 1.36 Culvert Excavation

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; 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 (Ref. 1.13, https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec12.pdf). The following sections discuss surface drainage of pavements and curb and gutter flow. Storm sewer inlets are discussed in Section 10.B of this chapter.

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 *RDM* (Ref. 1.3), Chapter Three: Roadway Alignment, Section 2.C.1).

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 *RDM* (Ref. 1.3), Chapter Three: Roadway Alignment, Section 3.B.2). 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 of this chapter). 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. <u>Curb and Gutter Flow</u>: 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 Large Scale (J) Sheets (See the *RDM* (Ref. 1.3), Chapter Eleven: <u>Highway Plans Assembly</u>, Section 4.J.1). 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. <u>Bridge Decks</u>: The roadway designer should coordinate with **Bridge** regarding drainage from bridge decks. The roadway designer should ask **Bridge** 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 generally should be sloped to drain away from the pavement, except with narrow raised 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 given 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 of this chapter). 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 of this chapter).

Hydraulic design procedures described in this section are based on information found in Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements (Ref. 1.13) and Hydraulic Engineering Circular No. 22: Urban Drainage Design (Ref. 1.14, Urban Drainage Design (HEC-22) 4th edition). See Appendix D: "Storm Sewer Policy", in this manual 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.

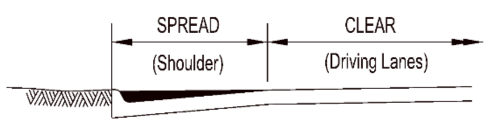
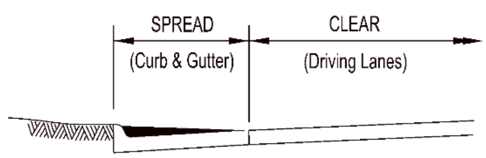
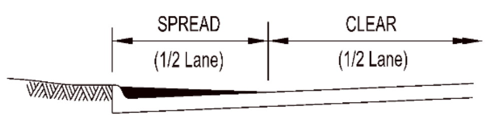
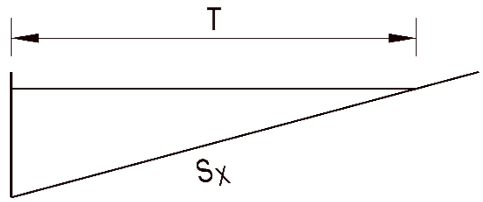
LOCATION	LIMITS OF SPREAD	ALLOWABLE SPREAD	DESIGN Q
HIGH-SPEED ROADWAYS (50 MPH AND OVER) WITH/SHOULDER		LOWEST EDGE OF DRIVING LANES	Q_{50}
HIGH-SPEED ROADWAYS (50 MPH AND OVER) WITHOUT SHOULDER		LOWEST EDGE OF DRIVING LANES (CREATE CURB & GUTTER OUTSIDE OF DRIVING LANE)	Q_{50}
LOW-SPEED ROADWAY (45 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 (Ref.1.13) 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:



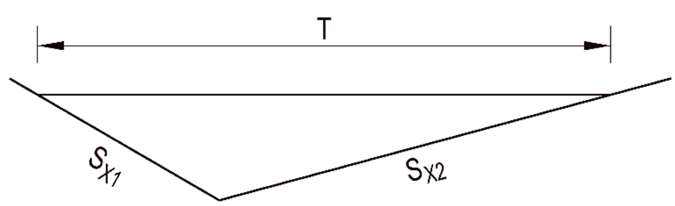
$Q = 0.56(z/n)S^{1/2}d^{8/3}$ (Use when depth (d) is known) Eq. 1.7a

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:



$S_X = (S_{X1}S_{X2}) \div (S_{X1} + S_{X2})$ 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", in this manual) 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", in this manual. Section 14.D of this chapter 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 J sheets (See the *RDM* (Ref. 1.3), Chapter Eleven: Highway Plans Assembly, Section 4.J). 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:

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 on page 89 of Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements (Ref. 1.13).

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.

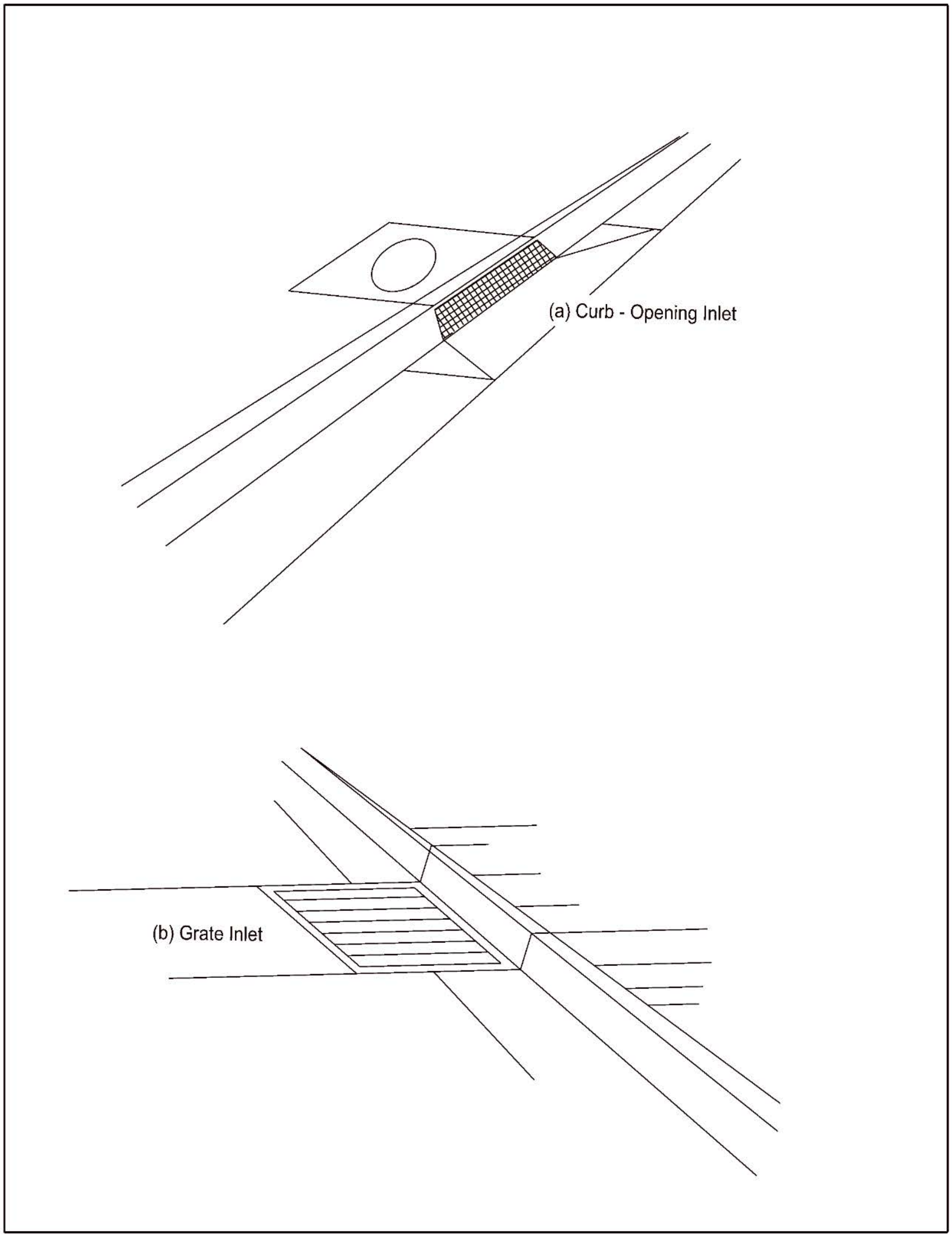
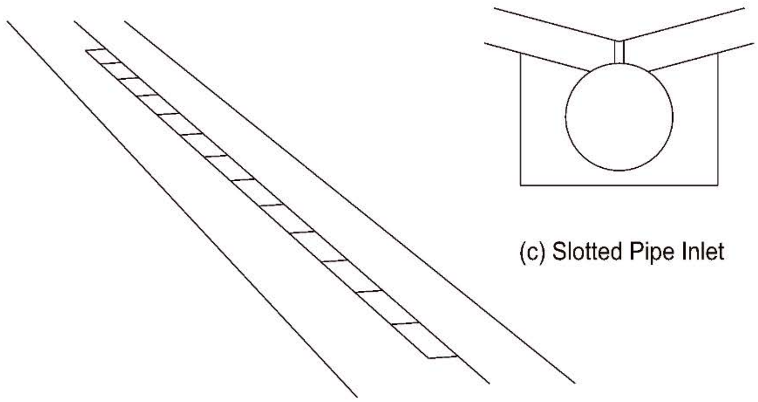
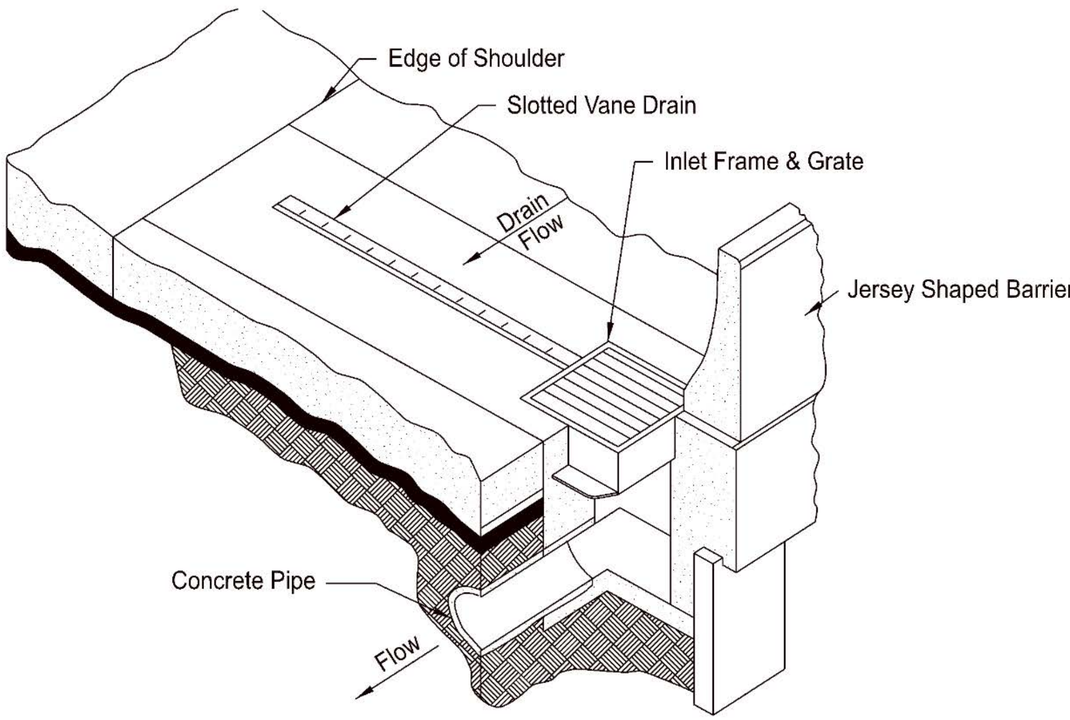


Exhibit 1.38 Types of Inlets



(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
- The upgrade of bridges
- The 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 at the upgrade of intersections, median breaks, crosswalks, and where pavement surfaces are warped serve 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 **Bridge** 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, inlet 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", in this manual contains performance curves for the curb inlets detailed on Standard Plan No. 443 in the *Standard Plans* (Ref. 1.12). The roadway designer should avoid placing curb inlets on intersection radii, they are difficult to construct and maintain. Roadway 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", in this manual are for curb inlets on continuous grades. The capacity of the inlet depends on the length of opening and depth of flow at the opening. This depth in turn depends on 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 Plans* (Ref. 1.12)
- 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 $Q_a \div L_a$, 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):

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

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

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

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

5. Flow intercepted, Q, is the ratio, $Q \div Q_a$, 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 for the capacity of curb inlets in a low point or sump (See Appendix G: "Nomographs and Charts for Gutter Flow & Inlet Design", in this manual) 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

The following values are shown in EXHIBIT G.4:

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 on 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 (Ref. 1.14).

10.B.3 Grate Inlets

At low velocities, all of the water flowing in the gutter section occupied by the grate, called frontal flow, is usually 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, the 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 (Ref. 1.13) and 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 a straight cross slope is expressed by the following equation:

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

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", in this manual 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:

$$Q_s \div Q = [1 - (Q_w \div Q)] = 1 - E_o \qquad \text{Eq. 1.11}$$

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:

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

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:

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

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:

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

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 of this chapter.

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 of this chapter.

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 = acceleration due to gravity, 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 EXHIBIT G.9 in Appendix G: "Nomographs and Charts for Gutter Flow & Inlet Design", in this manual.

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 of this chapter 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:

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

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:

$$E = 1 – [1 – (L \div L_T)]^{1.8} \qquad \text{Eq. 1.19}$$

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", in this manual.

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., depending 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", in this manual 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 of this chapter.

Orifice Condition

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

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

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

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

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

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 of this chapter 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.

For hydraulic design procedures for determining the capacity of curved vane grates, refer to Section 10.B.3 of this chapter.

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

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

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”, in this manual.

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 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 Plans* (Ref. 1.12) 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", in this manual, 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, Storm Sewer Hydraulic Grade Line, of this chapter 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 warranted by extenuating circumstances. The roadway 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 *RDM* (Ref. 1.3), Chapter Ten: Miscellaneous Design Issues, Section 12.D.4.

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:

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

where: V = Velocity of flow, ft./sec.
n = Manning's roughness coefficient (Appendix B: "Manning's Coefficient, n")
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:

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

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:

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

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

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", in this manual.

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 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:

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

where: S_f = Friction slope, ft./ft.
Q = Rate of flow, cfs
n = Manning's roughness coefficient (Appendix B: “Manning’s Coefficient, n”)
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:

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

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:

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

and

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

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.

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

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:

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

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", in this manual 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). 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", in this manual).

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

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

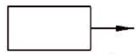
V_1, V_2, V_3, V_4 are the horizontal velocities of foregoing discharges, respectively, ft./sec.

Also assume:

- $H_b = K(V_1^2) \div 2g$ 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 of this chapter for an example problem.

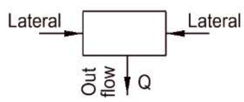


$$H_{tm} = \frac{v^2}{2g}$$

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

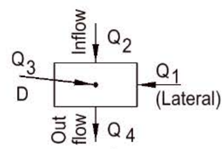
$$H_e = \frac{0.5\ v^2}{2g}$$

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



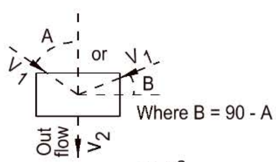
$$H_{j1} = \frac{v^2 \text{(Outflow)}}{2g}$$

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



$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ\ \ V_1\ \ I^2}{2_gQ_4}$$

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

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: Ref. 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 **DWEE NE** (See Rules and Regulations for the Design, Operation and Maintenance of Wastewater Works, Ref. 1.15, NAC Title 123 - Rules & Regulations for Design, Operation & Maintenance of Wastewater Works).

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 *RDM* (Ref. 1.3), Chapter Ten: Miscellaneous Design Issues, Section 12.D.

12. PIPE MATERIAL POLICY

Under this policy, roadway 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", in this manual.

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. **M&R** specifies the type of pavement subdrain to be used (See the *RDM* (Ref. 1.3), Chapter Eight: Surfacing, Section 3).

13.B Inverted Siphons

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 conditions. However, single barrel siphons have been constructed with diameters ranging in size from about 6 in. to 90 in. Siphons generally 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”, in this manual). 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.

If flared end sections are used on multiple pipes, additional clearance between pipes may be necessary to allow the flared ends to fit. See EXHIBIT 1.41 for jacking culvert pipe.

For additional information see the *Spec Book* (Ref. 1.16), Section731.

Length Increments of Jacked Pipe

When designing, keep jacking lengths to increments of 4 or 8 ft. For a length of 118 ft., the jacking contractor would either jack 14 sections of pipe and not get paid for 6 ft. or jack 15 sections of pipe and not get paid for 2 ft. Also, considering the size of the pipe being jacked, cutting off an excess amount of pipe can be difficult for the contractor. The steel casing generally in use comes in 20-ft. increments; 10-ft sections can be used if a 20-foot segment is cut in half. Typically, we only need to jack under the prism of the roadway itself.

Jacking Pipe Operations

A 30 in. steel casing is used when a 24 in. concrete pipe is jacked. Due to cost, the 30 in. steel casing is left in place instead of being pushed out. The space between the 24 in. concrete pipe and 30 in. steel casing is grouted and filled. Due to the bell on a 24 in. concrete pipe, it can be difficult to grout this area. The bell does not exist for pipes sized 30 in. and greater. If you have a circumstance where you need to jack a 24 in. pipe, it is recommended to size up and jack a 30 in. pipe.

Jack & Bore Horizontal Clearance

When boring and jacking a new pipe next to an existing pipe and the new pipe is 42 in. or less in diameter with a length of less than 100 ft., 3 ft. of clearance as measured from the outside of the new pipe to the outside of the existing pipe should be sufficient. A minimum of 5 ft. clearance should be provided if the new pipe is greater than 42 in. in diameter and/or the length of jacking is 100 ft. or more.

Jacking Round Equivalent Pipes

Jacking round equivalent pipe is not recommended. When jacking a round equivalent pipe, a steel casing is used to clear most of the material. When the round equivalent pipe is pushed through, the outside edges of the pipe can crack. Once installed, the contractor must fill in the void left around the pipe and settlement may be seen in the pavement and embankment above. In these cases, it is best to size a round pipe up and bury the bottom or let the silt fill the bottom naturally over time.

Summary of Jacking Operation Notes

1. Typically jack from the outlet side towards the inlet side.
2. Typically both a jacking (push pit) and a receiving pit on the other side will be needed.
3. Typically the push pit is 40 ft. x12 ft., measured from the jacking limits. The pit can be sized down to 36 ft. x 12 ft., but this forces the use of 10 ft. casing pipe lengths.
4. Typically the receiving pit is 24 ft. x 12 ft., measured from the jacking limits, 30 ft. x 12 ft. is preferred.
5. Typically any excavation would be approximately 1:1, but shoring may still be necessary at times.
6. Typically the maximum pipe slope allowed is 30 degrees (measured from horizontal).
7. **NDOT Right-of-Way Design** has a policy of adding a 50 ft. x 50 ft. jacking pit easement requirement, which may require acquiring temporary easements at the least.

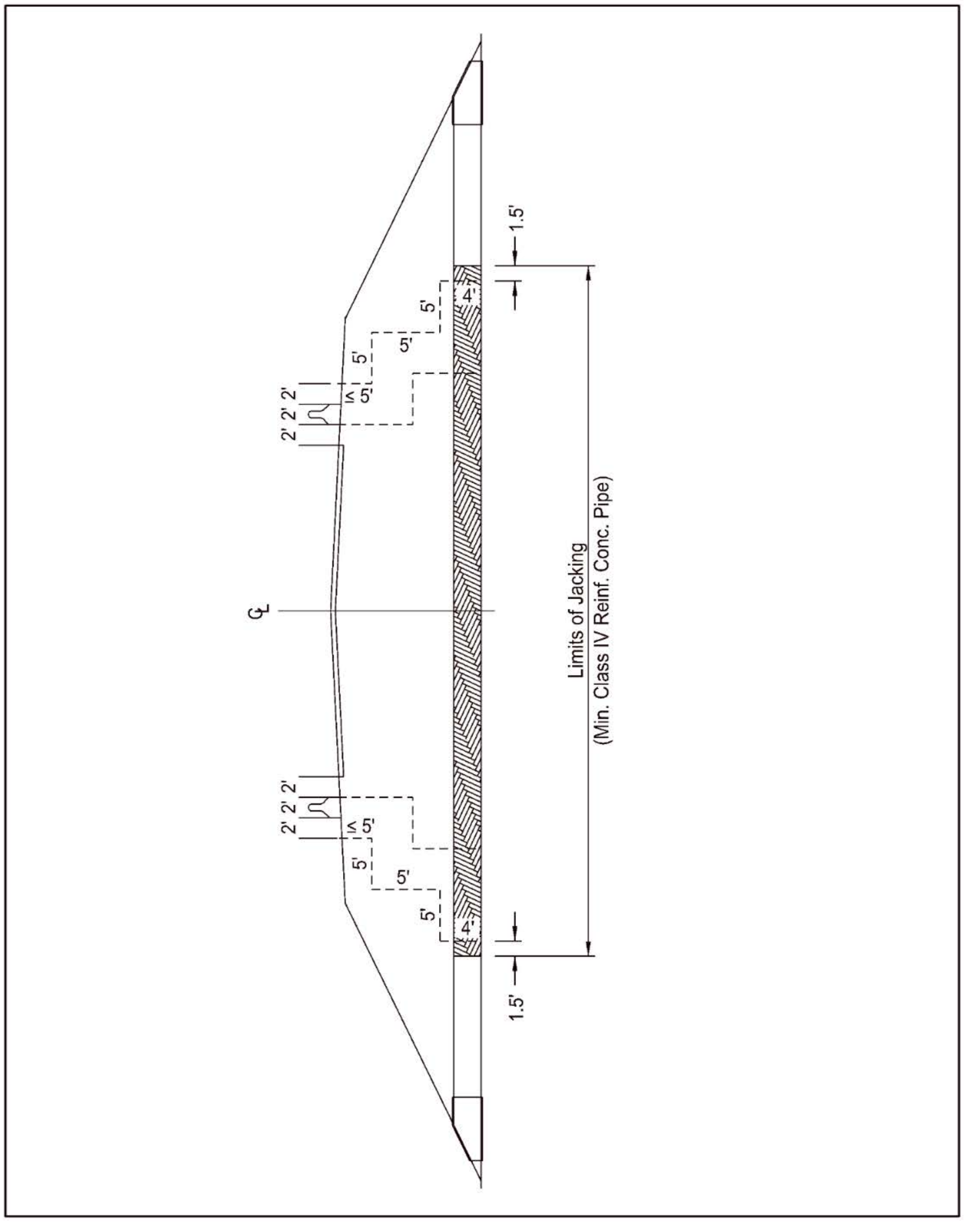


Exhibit 1.41 Jacking Culvert Pipe

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. Storage facilities are also used when improvements to the current stormwater system are not possible or are not 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 verifying that the stormwater runoff basins perform as required, there are other considerations that should be evaluated. One consideration is multiple use; 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. For additional information on sedimentation basins, see Chapter Two: Erosion and Sediment Control, Section 7.I.2, of this manual.

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 farmland; 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, the runoff coefficient (C) is 0.44.

Step 2: Determine the time of concentration (T_C) from EXHIBIT 1.11 using length of channel (max. 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, 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 ac ÷ 640 (there are 640 ac/sq. mi.) = 0.570 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 ac ÷ 640 (there are 640 ac/sq. mi.) = 0.391 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 ac ÷ 640 (there are 640 ac/sq. mi.) = 0.781 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”, in this manual).

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”, in this manual compute headwater depth.

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

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

$$EL_{hi} = EL_i + HW$$
$$= 82 \text{ ft.} + 6.84 \text{ ft.}$$
$$= 88.84 \text{ ft.}$$

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 (d_c + D) ÷ 2

(d_c + D) ÷ 2 = (4.27 + 6) ÷ 2 = 5.13 ft. > TW = 5 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: ______

C.N. ______ SHEET ______ OF ______

DESIGNER/DATE: ______ /

REVIEWER/DATE: ______ /

HYDROLOGICAL DATA

☐ METHOD: ______

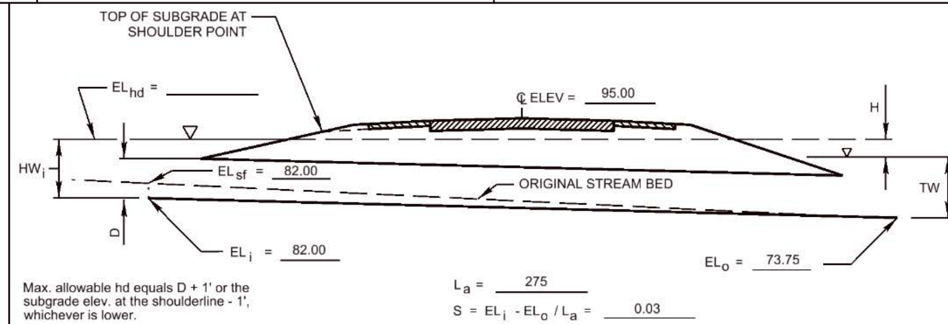
☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______

☐ CHANNEL SHAPE: ______

☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
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. 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 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) (Appendix B: "Manning's Coefficient, n", in this manual), n = 0.016.

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

$$Q = (0.56 \div n)\, S_x^{5/3}\, S^{1/2}\, T^{8/3}$$

$$5.75 = (0.56 \div n) \times 0.0208^{5/3} \times 0.03^{1/2} \times T^{8/3}$$

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

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", in this manual. 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:

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

Step 4: Compute L/L_a ratio

$$L/L_a = 12 \text{ ft.}/23 \text{ ft.} = 0.52$$

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

$Q/Q_a = 0.60$

Step 6: Determine flow intercepted and bypassed.

Q (intercepted) = 0.60 x 5.75 cfs
= 3.45 cfs

Q (bypassed) = 5.75 cfs - 3.45 cfs
= 2.30 cfs

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", in this manual. 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", in this manual, 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", in this manual

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", in this manual

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

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", in this manual

D = 2 in.= 0.17 ft.

Q_i = 1.75 cfs x (1.50 in./1.75 in.)
= 1.50 cfs

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.

$$Q_i = 0.94Ld^{0.5}$$
$$= 0.94 \text{ x } 10 \text{ ft. x } (0.417 \text{ ft.})^{0.5}$$
$$= 6.07 \text{ cfs}$$

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.

$$L_T = 0.6\, Q^{0.42}\, S^{0.3}\, [1 \div (nS_x)]^{0.6}$$
$$= 0.6 \text{ x } 1.39^{0.42} \text{ x } 0.02^{0.3} \text{ x } [1 \div (0.0150 \text{ x } .0417)]^{0.6}$$
$$= 17.8 \text{ ft.}$$

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

$$E = 1 - [1 - (L \div L_T)]^{1.8}$$
$$= 1 - [1 - (10 \div 17.8)]^{1.8}$$
$$= 0.77$$

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

$$Q_i = EQ$$
$$= 0.77 \times 1.39$$
$$= 1.07 \text{ cfs}$$

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", in this manual

$$K = 36.5$$

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

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

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

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”, in this manual.

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.

$$
\begin{aligned}
C \times A &= 0.30 \times 1.5 \text{ acres} \\
&= 0.45
\end{aligned}
$$

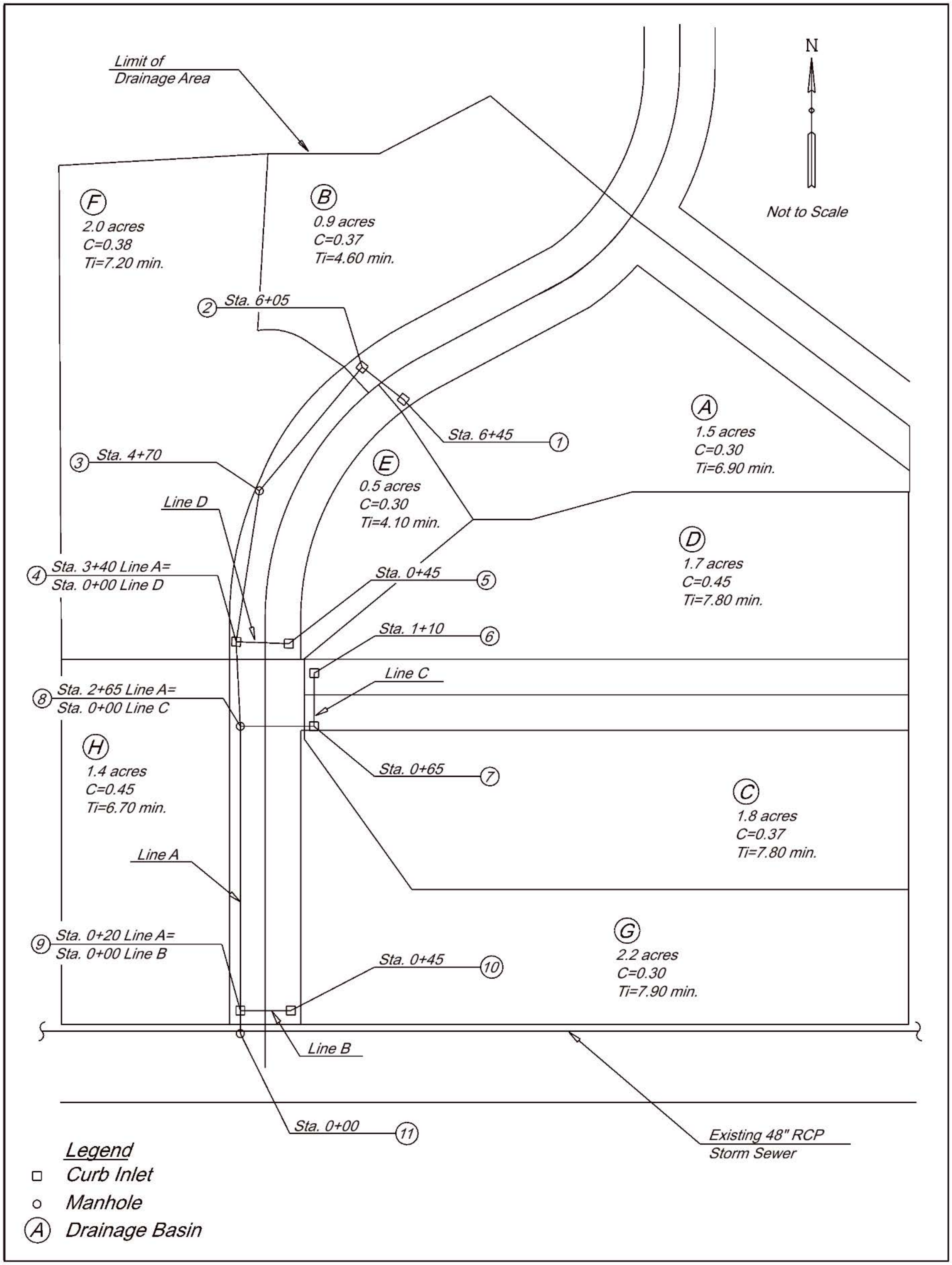


Exhibit 1.43 Drainage Area Map

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

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

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

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 = 40 ft. ÷ 6.38 fps = 6.3 sec., which at 60 sec./min. = 0.10 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:

$$C_w = [C_{(1)}A_{(1)} + C_{(2)}A_{(2)}] \div (A_{(1)} + A_{(2)})$$

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

$$C_w = [(0.3 \text{ x } 0.5) + (0.38 \text{ x } 2.0)] \div (2.0 + 0.5) = 0.36$$

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.

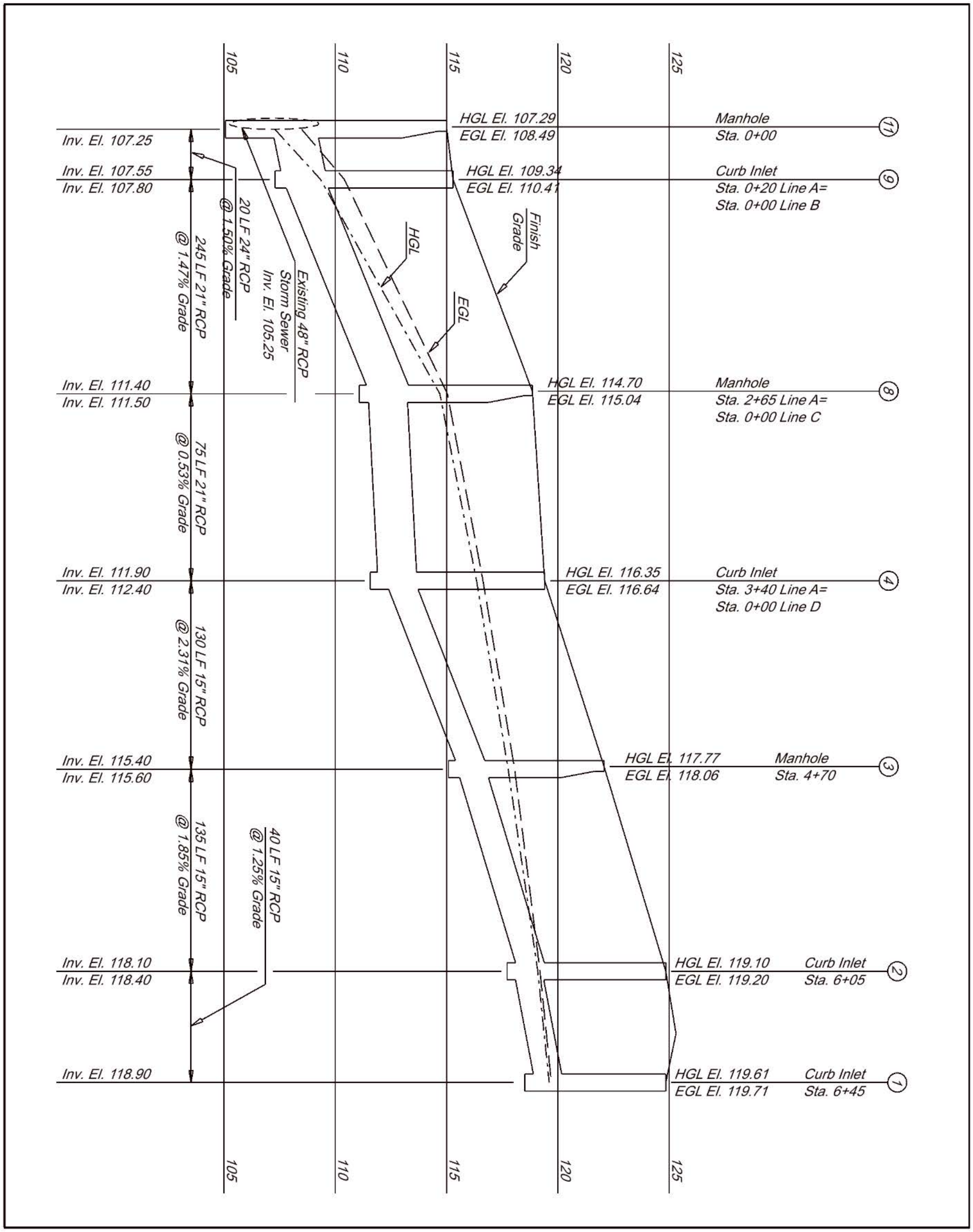


Exhibit 1.44a Storm Sewer Profile

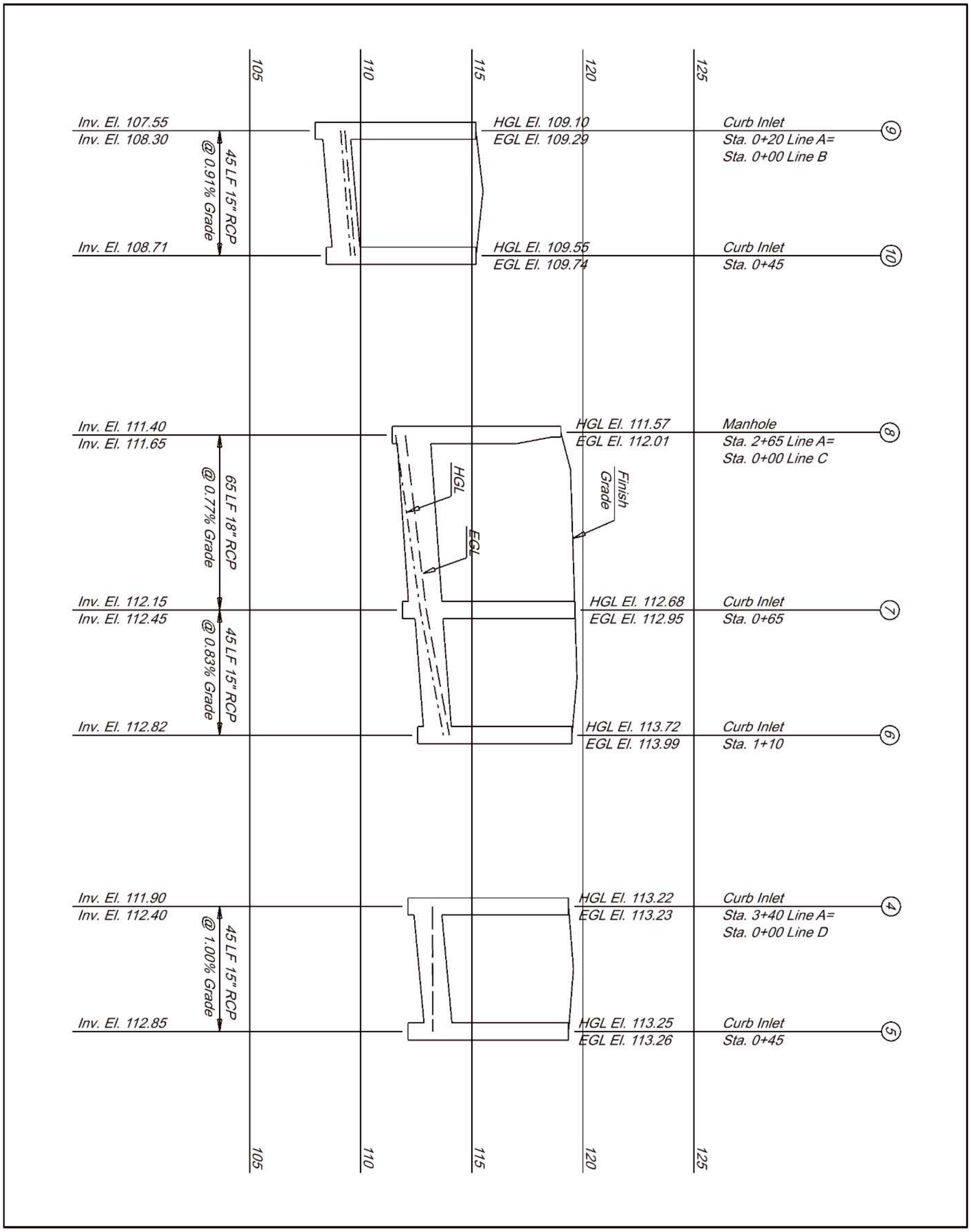


Exhibit 1.44b Storm Sewer Profile

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.

$$q \div Q_{full} = 3.11 \text{ cfs} \div 7.82 \text{ cfs} = 0.40$$

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.

$$d \div D = 0.44$$

$$d_n = 0.44 \text{ x } (15 \text{ ft.} \div 12 \text{ in.}) = 0.55 \text{ ft.}$$

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

$$d_c = 0.71 \text{ ft.}$$

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.

$$d_n = 0.55 \text{ ft.} < dc = 0.71 \text{ ft.}$$

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:

$$V_q = q \div A_{full}$$

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

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

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

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:

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

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

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

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

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:

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

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

$$S_f = [Q_n \div (1.486AR^{2/3})]^2$$

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

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

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

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:

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

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

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

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

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

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

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

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

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

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

15. REFERENCES

1.1 American Association of State Highway and Transportation Officials, Model Drainage Manual, Washington D.C., 2014

1.2 American Association of State Highway and Transportation Officials, Highway Drainage Guidelines, 2007.

1.3 Nebraska Department of Transportation, Roadway Design Manual, Current Edition (https://dot.nebraska.gov/business-center/design-consultant/rd-manuals/)

1.4 State of Nebraska Administrative Code Title 455, Chapter 1, Nebraska Natural Resources Commission Rules and Regulations Concerning Minimum Standards for Floodplain Management Programs, 07/01/2008. (https://dnr.nebraska.gov/sites/default/files/doc/desk-reference/legal-authority/Title_455_0708.pdf)

1.5 Kirpich, P.Z., Civil Engineering, Vol. 10, No. 6, June 1940.

1.6 Cordes, K.E. and R.H. Hotchkiss, Design Discharge of Culverts, NDOT Research Project No. RES-1 (0099) P466, December 1993.

1.7 U.S. Department of Commerce, U.S. Weather Bureau, Rainfall Frequency Atlas of the U.S. for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years, Technical Paper No. 40, May 1961. (TechnicalPaper_No40.pdf)

1.8 U.S. Department of Transportation, Federal Highway Administration, Design of Roadside Drainage Channels, Hydraulic Design Series No. 4, 1965. (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds4.pdf) - Archived

1.9 U.S. Army Corps of Engineers, Water Surface Profiles, User Manual, Hydrologic Engineering Circular (HEC2), 1990. (HEC-2 Water Surface Profiles)

1.10 U.S. Department of Transportation, Federal Highway Administration, Design Charts for Open-Channel Flow, Hydraulic Design Series No. 3, 1961. (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds3.pdf) – Archived, may no longer reflect current or accepted regulation, policy, guidance or practice.

1.11 U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, April 2012. (Hydraulic Design of Highway Culverts - HDS-5 - Third Edition)

1.12 Nebraska Department of Transportation, Standard/Special Plans Book, Current Edition. (https://dot.nebraska.gov/business-center/design-consultant/stand-spec-manual/)

1.13 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) - Archived

1.14 U.S. Department of Transportation, Federal Highway Administration, Urban Drainage Design, Hydraulic Engineering Circular (HEC) 22, Fourth Edition, FHWA-HIF-24-006, February 2024. (Urban Drainage Design (HEC-22) 4th edition)

1.15 State of Nebraska Administrative Code Title 123 – Rules and Regulations for the Design, Operation and Maintenance of Wastewater Works, Chapter 5 – Design Standards (NAC Title 123 - Rules & Regulations for Design, Operation & Maintenance of Wastewater Works)

1.16 Nebraska Department of Transportation, Standard Specifications for Highway Construction, (*Spec Book*) 2017. (https://dot.nebraska.gov/media/g4qp4y0d/2017-specbook.pdf)