CHAPTER 5

CHANNEL DESIGN

5-1 **OPEN CHANNEL FLOW.** Roadside and median channels are open-channel systems that collect and convey storm water from the pavement surface, roadside, and median areas. These channels may outlet to a storm drain piping system via a drop inlet, to a detention or retention basin or other storage component, or to an outfall channel. Roadside and median channels are normally trapezoidal in cross section and are lined with grass or other protective lining.

The design and analysis of roadside and median channels follow the basic principles of open channel flow. Summaries of several important open channel flow concepts and relationships are presented in many hydraulic engineering texts and in the FHWA's HEC-22 manual.

5-1.1 **Flow Resistance.** The depth of flow in a channel of given geometry and longitudinal slope is primarily a function of the channel's resistance to flow or roughness. This depth is called the normal depth and is computed from Manning's equation for "V" combined with the continuity equation, $Q = VA$. The combined equation, often referred to as Manning's equation, is:

$$Q = \frac{1.486 A R^{0.67} S_o^{0.5}}{n} \tag{5-1}$$

where:

Q = discharge rate, ft^3/s

A = cross-sectional flow area, ft^2

R = hydraulic radius, $\frac{A}{P}$, ft

P = wetted perimeter, ft

S_o = energy grade line slope, ft/ft

n = Manning's roughness coefficient

Nomograph solutions to Manning's equation for triangular and trapezoidal channels are presented in Appendix B and are also available in many other texts.

5-1.1.1 The selection of an appropriate Manning's *n* value for design purposes is often based on observation and experience. Manning's *n* values are also known to vary with flow depth. Table 5-1 provides Manning's *n* values for natural channels; Table 5-2 provides a tabulation of Manning's *n* values for various channel lining materials.

Table 5-1. Manning's *n* for Natural Stream Channels (Surface Width at Flood Stage Less than 100 ft)

Stream Channel Characteristics	*n* Value
Fairly regular section:	
Some grass and weeds, little or no brush	0.030-0.035
Dense growth of weeds, depth of flow materially greater than weed height	0.035-0.05
Some weeds, light brush on banks	0.035-0.05
Some weeds, heavy brush on banks	0.05-0.07
Some weeds, dense willows on banks	0.06-0.08
For trees within the channel with branches submerged at high stage, increase all above values by	0.01-0.02
Irregular sections with pools, slight channel meander: increase these values approximately	0.01-0.02
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
Bottom of gravel, cobbles, and few boulders	0.04-0.05
Bottom of cobbles, with large boulders	0.05-0.07

Table 5-2. Manning's Roughness Coefficients for Lined Channels**

Lining Category	Lining Type	*n* Value for Given Depth Ranges		
		0 - 0.5 ft	0.5 - 2.0 ft	> 2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Element	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021

Lining Category	Lining Type	*n* Value for Given Depth Ranges		
		0 - 0.5 ft	0.5 - 2.0 ft	> 2.0 ft
Gravel Riprap	1 in. D_{50}	0.044	0.033	0.030
	2 in. D_{50}	0.066	0.041	0.034
Rock Riprap	6 in. D_{50}	0.104	0.069	0.035
	12 in. D_{50}	--	0.078	0,040
NOTE: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, *n*, vary with the flow depth. * Some "temporary" linings become permanent when buried. ** Table reproduced from FHWA HEC-15				

5-1.1.2 Manning's roughness coefficient for vegetative and other linings varies significantly depending on the amount of submergence. The classification of vegetal covers by degree of retardance is provided in Table 5-3. Table 5-4 provides a list of Manning's n relationships for five classes of vegetation defined by their degree of retardance.

Table 5-3. Classification of Vegetal Covers as to Degree of Retardance*

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall, average 2.5 ft
	Yellow bluestem	Excellent stand, tall, average 3.0 ft
	Ischaemum	
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall, average 1.0 ft
	Native grass mixture (Little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, average 2.0 ft
	Lespedeza sericea	Good stand, not woody, tall, average 1.6 ft
	Alfalfa	Good stand, uncut, average 0.91 ft
	Weeping lovegrass	Good stand, unmowed, average 1.1 ft
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut, average 1.1 ft

Retardance Class	Cover	Condition
C	Crabgrass Bermuda grass Common lespedeza Grass-legume mixture—summer (orchard grass, redtop Italian ryegrass, and common lespedeza) Centipede grass Kentucky bluegrass	Fair stand, uncut, average 0.8 to 4.0 ft Good stand, mowed, average 0.5 ft Good stand, uncut, average 0.91 ft Good stand, uncut, average 0.5 to 1.5 ft Very dense cover, average 0.5 ft Good stand, headed, average. 0.5 to 1.0 ft
D	Bermuda grass Common lespedeza Buffalo grass Grass-legume mixture—fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza) Lespedeza sericea	Good stand, cut to 0.2 ft Excellent stand, uncut, average 0.4 ft Good stand, uncut, average 0.3 to 0.5 ft Good stand, uncut, average 0.3 to 0.4 ft After cutting to 0.2-ft height, very good stand before cutting
E	Bermuda grass Bermuda grass	Good stand, cut to average 0.1 ft Burned stubble
NOTE: These covers have been tested in experimental channels. The covers were green and generally uniform. *Table reproduced from FHWA HEC-15		

Table 5-4. Manning's *n* Relationships for Vegetal Degree of Retardance

Retardance Class	Manning's n Equation*	Chapter Equation Number
A	$\frac{R^{1/6}}{\left[15.8+19.97\,log\left(R^{1.4}S_o^{0.4}\right)\right]}$	5-2
B	$\frac{R^{1/6}}{\left[23.0+19.97\,log\left(R^{1.4}S_o^{0.4}\right)\right]}$	5-3
C	$\frac{R^{1/6}}{\left[30.2+19.97\,log\left(R^{1.4}S_o^{0.4}\right)\right]}$	5-4
D	$\frac{R^{1/6}}{\left[34.6+19.97\,log\left(R^{1.4}S_o^{0.4}\right)\right]}$	5-5
E	$\frac{R^{1/6}}{\left[37.7+19.97\,log\left(R^{1.4}S_o^{0.4}\right)\right]}$	5-6
* Equations are valid for flows less than 50 ft^3/s. Nomograph solutions for these equations are in FHWA HEC-15.		

5-1.1.3 **Example 5-1**

Given: A trapezoidal channel (as shown in Figure 5-3) with these characteristics:

- S_o = 0.01
- B = 2.62 ft
- z = 3
- d = 1.64 ft

Find: The channel capacity and flow velocity for these channel linings:

(1) Riprap with a median aggregate diameter, d_{50} = 6 in.

(2) A good stand of buffalo grass, uncut, 3 to 6 in.

5-1.1.3.1 ***Solution 1*: Riprap**

Step 1. Determine the channel parameters. From Table 5-1:

n = 0.069

A = $Bd + 2(1/2)(d)(zd)$

= $Bd + zd^2$

= (2.62)(1.64) + (3)(1.64)2

= 12.4 ft^2

P = B + 2[(zd)2 + d2)]1/2

= B + 2d(z2 + 1)0.5

= (2.62)+ (2)(1.64) + (32 + 1)0.5

= 13.0 ft

$$R = \frac{A}{P}$$

$$= \frac{12.4}{13.0}$$

= 0.95 ft

Step 2. Compute the flow capacity.

$$Q_n = 1.49AR^{0.67}S_o^{0.5}$$

$$= (1.49)(12.4)(0.95)0.67(0.01)0.5$$

$$= 1.79 \text{ ft}^3/\text{s}$$

$$Q = \frac{Qn}{n}$$

$$= \frac{1.79}{0.069}$$

$$= 25.9 \text{ ft}^3/\text{s}$$

Step 3. Compute the flow velocity.

$$V = \frac{Q}{A}$$

$$= \frac{25.9}{12.4}$$

$$= 2.1 \text{ ft/s}$$

5-1.1.3.2 ***Solution 2*: Buffalo Grass**

Step 1. Determine the roughness. Use these characteristics:

- Degree of retardance from Table 5-3
- Retardance Class D
- From paragraph 5-1.1.3.1, solution 1, step 1: R = 0.95 ft
- Roughness coefficient, n, from Table 5-4

$$n = \frac{R^{0.167}}{34.6 + 19.97\,log\left[(R)^{1.4}(S_o)^{0.4}\right]}$$

$$n = \frac{(0.95)^{0.167}}{34.6 + 19.97\,log\left[(0.95)^{1.4}(0.01)^{0.4}\right]}$$

$$n = 0.055$$

Step 2. Compute the flow capacity. Use these values from step 1:

$$Q_n = 1.79 \text{ ft}^3/\text{s}$$

$$Q = \frac{Qn}{n}$$

$$= \frac{1.79}{0.55}$$

$$= 32.5 \text{ ft}^3/\text{s}$$

Step 3. Compute the flow velocity.

$$V = \frac{Q}{A}$$

$$= \frac{32.5}{12.4}$$

$$= 2.62 \text{ ft/s}$$

5-1.2 **Stable Channel Design.** HEC-15 provides a detailed presentation of stable channel design concepts related to the design of roadside and median channels. This section provides a brief summary of significant concepts.

5-1.2.1 Stable channel design concepts provide a means of evaluating and defining channel configurations that will perform within acceptable limits of stability. For most highway drainage channels, bank instability and lateral migration cannot be tolerated. Stability is achieved when the material forming the channel boundary effectively resists the erosive forces of the flow. Principles of rigid boundary hydraulics can be applied to evaluate this type of system.

5-1.2.2 Both velocity and tractive force methods have been applied to the determination of channel stability. Permissible velocity procedures are empirical in nature, and have been used to design numerous channels in the United States and throughout the world. However, tractive force methods consider actual physical processes occurring at the channel boundary and represent a more realistic model of the detachment and erosion processes.

5-1.2.3 The hydrodynamic force created by water flowing in a channel causes a shear stress on the channel bottom. The bed material, in turn, resists this shear stress by developing a tractive force. Tractive force theory states that the flow-induced shear stress should not produce a force greater than the tractive resisting force of the bed material. This tractive resisting force of the bed material creates the permissible or critical shear stress of the bed material. In a uniform flow, the shear stress is equal to the effective component of the gravitational force acting on the body of water parallel to the channel bottom. The average shear stress is equal to:

$$\tau = \gamma RS \tag{5-7}$$

where:

τ = average shear stress, lb/ft^2

γ = unit weight of water, 62.4 lb/ft^3 (at 15.6 °C (60 °F))

R = hydraulic radius, ft

S = average bed slope or energy slope, ft/ft

5-1.2.4 The maximum shear stress for a straight channel occurs on the channel bed and is less than or equal to the shear stress at maximum depth. The maximum shear stress is computed as follows:

$$\tau_d = \gamma dS \tag{5-8}$$

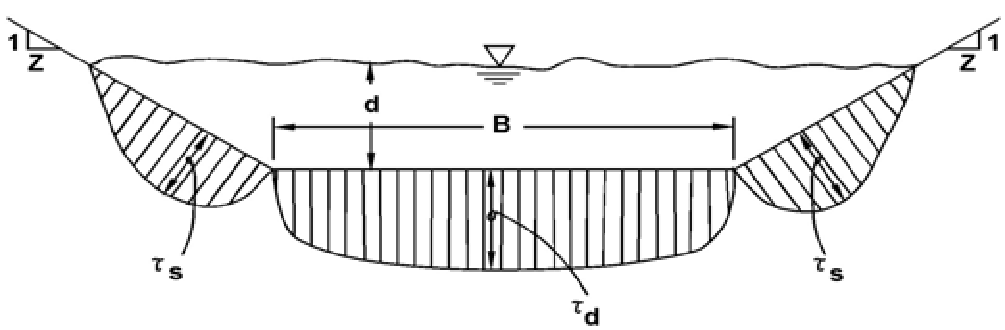
where:

τ_d = maximum shear stress, lb/ft^2

d = maximum depth of flow, ft

5-1.2.5 Shear stress in channels is not uniformly distributed along the wetted perimeter of a channel. A typical distribution of shear stress in a trapezoidal channel tends toward zero at the corners with a maximum on the bed of the channel at its centerline, and the maximum for the side slopes occurs around the lower third of the slope, as illustrated in Figure 5-1.

Figure 5-1. Distribution of Shear Stress



5-1.2.6 For trapezoidal channels lined with gravel or riprap having side slopes steeper than 3:1, side slope stability must also be considered. This analysis is performed by comparing the tractive force ratio between side slopes and channel bottom with the ratio of shear stresses exerted on the channel sides and bottom. The ratio of shear stresses on the sides and bottom of a trapezoidal channel, K_1, is given in Chart 17 of Appendix B and the tractive force ratio, K_2, is given in Chart 18. The angle of repose, θ, for different rock shapes and sizes is provided in Chart 19. The required rock size for the side slopes is found using the following equation:

$$(d_{50})_{sides} = \frac{K_1}{K_2}(d_{50})_{bottom} \tag{5-9}$$

where:

d_{50} = mean riprap size, ft

K_1 = ratio of shear stresses on the sides and bottom of a trapezoidal channel

K_2 = ratio of tractive force on the sides and bottom of a trapezoidal channel

5-1.2.6.1 Flow around bends also creates secondary currents, which impose higher shear stresses on the channel sides and bottom compared to straight reaches. Areas of high shear stress in bends are illustrated in Figure 5-2. The maximum shear stress in a bend is a function of the ratio of channel curvature to bottom width. This ratio increases as the bend becomes sharper and the maximum shear stress in the bend increases. The bend shear stress can be computed using this relationship:

$$\tau_b = K_b \tau_d \tag{5-10}$$

where:

τ_b = bend shear stress, lb/ft^2

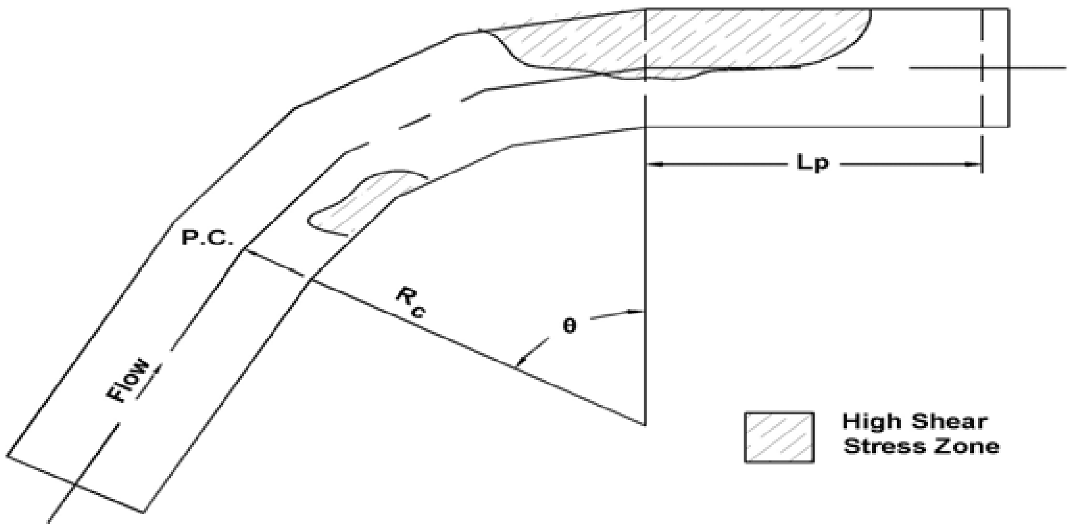
K_b = function of R_c / B (see Chart 21, HEC-22)

R_c = radius to the centerline of the channel, ft

B = bottom width of channel, ft

τ_d = maximum channel shear stress, lb/ft^2

Figure 5-2. Shear Stress Distribution in Channel Bends



5-1.2.6.2 The increased shear stress produced by the bend persists downstream of the bend a distance, *p*, as shown in Figure 5-2. This distance can be computed using this relationship:

$$L_p = \frac{0.604 R^{7/6}}{n_b} \tag{5-11}$$

where:

L_p = length of protection (length of increased shear stress due to the bend) downstream of the point of tangency, ft

n_b = Manning's roughness in the channel bend

R = hydraulic radius, ft

5-1.2.6.3 **Example 5-2**

Given: A trapezoidal channel with these characteristics:

S_o = 0.01 ft/ft

B = 3.0 ft

z = 3

Lining = A good stand of buffalo grass 3 to 6 in. high. From Example 5-1, Solution 2, n = 0.055.

The channel reach consists of a straight section and a 90-degree bend with a centerline radius of 14.8 ft. The design discharge is 28.2 ft^3/s.

Find: The maximum shear stress in the straight reach and in the bend.

Solution:

Step 1. Compute the channel parameters.

$$Q_n = (28.2)(0.055)$$

$$= 1.555 \text{ ft}^3/\text{s}$$

From (Chart 14A):

$$d/B = 0.49$$

$$d = B\ d/B$$

$$= (3.0(0.49)$$

$$= 1.47 \text{ ft}$$

Step 2. Compute the maximum shear stress in the straight reach.

$$\tau_d = \gamma dS$$

$$= (62.5)(1.47)(0.01)$$

$$= 0.92 \text{ lb/ft}^2$$

Step 3. Compute the shear stress in the bend.

$$\frac{R_c}{B} = \frac{(14.8)}{(3.0)}$$

$$= 4.93$$

From Chart 21 (HEC-22):

$$K_b = 1.55$$

Using Equation 5-10:

$$\tau_b = K_b \tau_d$$

$$= (1.55)(0.92)$$

$$= 1.43 \text{ lb/ft}^2$$

5-2 **DESIGN PARAMETERS.** Parameters required for the design of roadside and median channels include discharge frequency, channel geometry, channel slope, vegetation type, freeboard, and shear stress. This section provides criteria relative to the selection or computation of these design elements.

5-2.1 **Discharge Frequency.** Roadside and median drainage channels are typically designed to carry 5- to 10-yr design flows; however, when designing temporary channel linings, a lower return period can be used. Usually a 2-yr return period is appropriate for the design of temporary linings.

5-2.2 **Channel Geometry.** Most drainage channels are trapezoidal. Several typical shapes with equations for determining channel properties are illustrated in Figure 5-3. The channel depth, bottom width, and top width must be selected to provide the necessary flow area. Chart 22 of Appendix B provides a nomograph solution for determining channel properties for trapezoidal channels.

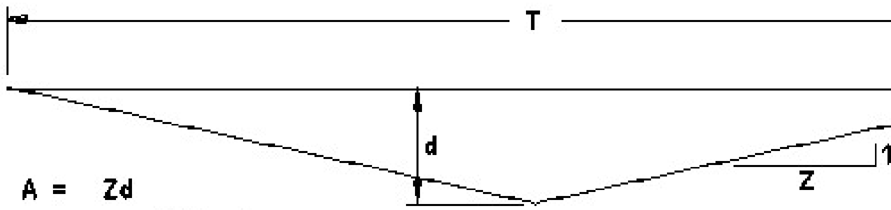
Channel side slopes for triangular or trapezoidal channels should not exceed the angle of repose of the soil and/or lining material, and should generally be 1V:3H or flatter. In areas where traffic safety may be of concern, channel side slopes should be 1V:4H or flatter.

Design of roadside and median channels should be integrated with the geometric and pavement design to ensure proper consideration of safety and pavement drainage needs.

5-2.3 **Channel Slope.** Channel bottom slopes are generally dictated by the road profile or other constraints. However, if channel stability conditions warrant, it may be feasible to adjust the channel gradient slightly to achieve a more stable condition. Channel gradients greater than 2 percent may require the use of flexible linings to maintain stability. Most flexible lining materials are suitable for protecting channel gradients of up to 10 percent, with the exception of some grasses. Linings such as riprap and wire-enclosed riprap are more suitable for protecting very steep channels with gradients in excess of 10 percent. Rigid linings, such as concrete paving, are highly susceptible to failure from structural instability due to such occurrences as overtopping, freeze thaw cycles, swelling, and excessive soil pore water pressure.

5-2.4 **Freeboard.** The freeboard of a channel is the vertical distance from the water surface to the top of the channel. The importance of this factor depends on the consequence of overflow of the channel bank. At a minimum the freeboard should be sufficient to prevent waves, superelevation changes, or fluctuations in water surface from overflowing the sides. In a permanent roadside or median channel, about 0.5 ft of freeboard is generally considered adequate. For temporary channels no freeboard is necessary. However, a steep gradient channel should have a freeboard height equal to the flow depth to compensate for the large variations in flow caused by waves, splashing, and surging.

Figure 5-3. Channel Geometries

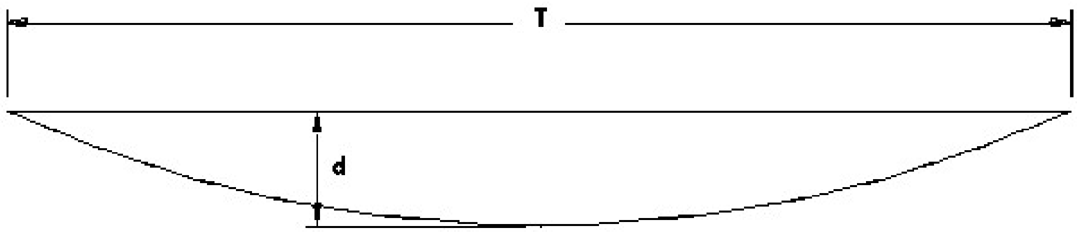


$A = Zd$

$P = 2d\sqrt{Z^2 + 1}$

$T = 2dZ$

V - Shape

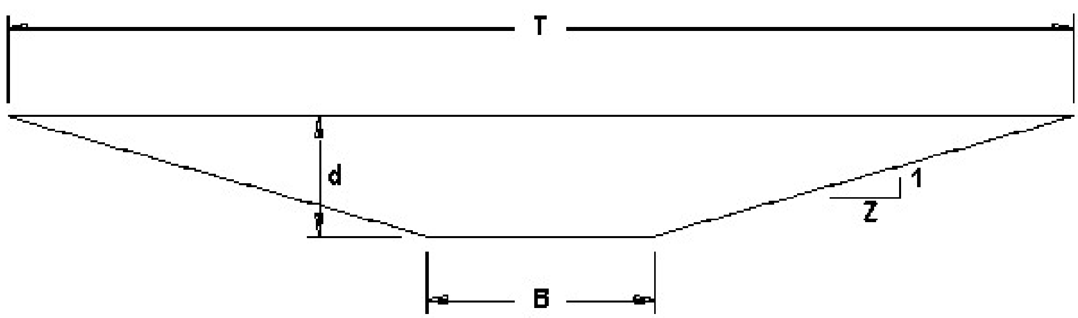


$A = \frac{2}{3}Td$

$P = \frac{1}{2}\sqrt{16d^2 + T^2} + \left(\frac{T^2}{8d}\right)\ln_e\left(\frac{4d + \sqrt{16d^2 + T^2}}{T}\right)$

$T = 1.5\,\frac{A}{D}$

Parabolic



$A = Bd + Zd^2$

$P = B + 2d\sqrt{Z^2 + 1}$

$T = b + 2dZ$

Trapeziodal

5-2.5 **Shear Stress.** The permissible or critical shear stress in a channel defines the force required to initiate movement of the channel bed or lining material. Table 5-5 shows permissible shear stress values for manufactured, vegetative, and riprap channel lining. The permissible shear stress for non-cohesive soils is a function of mean diameter of the channel material as shown in Chart 23 of Appendix B. For larger stone sizes not shown in Chart 23 and rock riprap, the permissible shear stress is given by the following equation:

$$\tau_p = 4.0 D_{50} \tag{5-12}$$

where:

τ_p = permissible shear stress, lb/ft^2

d_{50} = mean riprap size, ft

For cohesive materials, the plasticity index provides a good guide for determining the permissible shear stress as illustrated in Chart 24 of Appendix B.

Table 5-5. Permissible Shear Stresses for Lining Materials**

Lining Category	Lining Type	Permissible Unit Shear Stress, lb/ft^2
Temporary*	Woven Paper Net	0.15
	Jute Net	0.45
	Fiberglass Roving:	
	Single	0.60
	Double	0.85
	Straw with Net	1.45
	Curled Wood Mat	1.55
	Synthetic Mat	2.00
Vegetative	Class A	3.70
	Class B	2.10
	Class C	1.00
	Class D	0.60
	Class E	0.35
Gravel Riprap	1 in.	0.33
	2 in.	0.67
Rock Riprap	6 in.	2.00
	12 in.	4.00

Lining Category	Lining Type	Permissible Unit Shear Stress, lb/ft^2
Bare Soil	Non-cohesive	
	Cohesive	
*Some “temporary” linings become permanent when buried. **Table reproduced from HEC-15		

5-2.5.1 **Example 5-3**

Given: The channel section and flow conditions in Example 5-2, paragraph 5-1.2.6.3.

Find: Determine if a good stand of buffalo grass (Class D degree of retardance) will provide an adequate lining for this channel.

Solution:

Step 1. Determine the permissible shear stress.

From Table 5-4:

$$\tau_p = 0.60 \text{ lb/ft}^2$$

Step 2. Compare τ_p with the maximum shear stress in the straight section, τ_d, and with the shear stress in the bend, τ_b.

$$\tau_d = 0.92 \text{ lb/ft}^2$$

$$\tau_b = 1.43 \text{ lb/ft}^2$$

$$\tau_p = 0.60 < \tau_d = 0.92$$

$$\tau_p = 0.60 < \tau_b = 1.43$$

5-2.5.2 **Example 5-4**

Given: The channel section and flow conditions in Example 5-2 (paragraph 5-1.2.6.3) and Example 5-3 (paragraph 5-2.5.1).

Find: Determine the length of increased shear stress downstream of the point of tangency of the 90-degree bend.

Solution:

Step 1. Determine the flow depth and hydraulic radius.

Assume that the flow depth and hydraulic radius in the bend will be approximately the same as those in the straight reach.

From Example 5-2:

$$d = 1.47 \text{ ft}$$

$$\text{with } d/B = 1.47/3.0$$

$$= 0.49$$

From Chart 22:

$$R/d = 0.61$$

$$R = d\,R/d$$

$$= (1.47)(0.61)$$

$$= 0.90 \text{ ft}$$

Step 2. Determine the channel roughness in the bend.

From Example 5-2:

$$n = 0.055$$

Step 3. Determine length of increased shear stress.

Using Equation 5-11:

$$L_p = \frac{0.604 R^{7/6}}{n_b}$$

$$= \frac{0.604(0.90)^{7/6}}{(0.055)}$$

$$= 9.7 \text{ ft}$$

Since the permissible shear stress, τ_p, was less than the actual shear stress in the bend, τ_b, an adequate lining material would have to be installed throughout the bend plus the length, L_p, downstream of the point of tangency of the curve.

CHAPTER 6

STORM DRAIN DESIGN

6-1 **PURPOSE AND SCOPE.** A storm drain is that portion of the drainage system that receives surface water through inlets and conveys the water through conduits to an outfall. It is composed of different lengths and sizes of pipe or conduit connected by appurtenant structures. A section of conduit connecting one inlet or appurtenant structure to another is termed a "segment" or "run." The storm drain conduit is most often a circular pipe, but it can also be a box or other enclosed conduit shape. Appurtenant structures include inlet structures (excluding the actual inlet opening), access holes, junction chambers, and other miscellaneous structures. Generalized design considerations for these structures are presented in Chapter 7. The computation of energy losses through these structures is described in detail in HEC-22, Chapter 7.

6-2 **DESIGN PROCEDURES FOR THE DRAINAGE SYSTEM.** Design storm runoff must be efficiently removed to avoid interruption of operations during or following storms and to prevent temporary or permanent damage to pavement subgrades. Removal is accomplished by a drainage system unique to each site. Drainage systems will vary in design and extent depending upon local soil conditions and topography; size of the physical facility; vegetation cover or its absence; the anticipated presence or absence of ponding; and most importantly, upon local storm intensity and frequency patterns. The drainage system should function with a minimum of maintenance difficulties and expense and should be adaptable to future expansion. Open channels or natural water courses are permitted only at the periphery of an airfield or heliport facility and must be well removed from the landing strips and traffic areas. Provisions for subsurface pavement drainage, the requirements for which are provided in UFC 3-250-01FA or UFC 3-260-01, may necessitate careful consideration. Subdrains are used to drain the base material, lower the water table, or drain perched water tables. Fluctuations of the water table must be considered in the initial design of the facility. A detailed step-by-step design procedure starts in section 6-3.

6-2.1 **Grading.** Proper grading is the most important single factor contributing to the success of the drainage system. Development of grading and drainage plans must be fully coordinated. Specific grading criteria for airfields can be found in UFC 3-260-01 for DOD and AC 150/5300-13 for FAA.

6-2.2 **Classification of Storm Drains.** Storm drains may be classified in two groups, primary and auxiliary.

6-2.2.1 **Primary Drains.** Primary drains consist of main drains and laterals that have sufficient capacity to accommodate the project design storm, either with or without supplementary storage in ponding basins above the drain inlets. To lessen construction requirements for drainage facilities, maximum use of ponding consistent with operational and grading requirements will be considered. The location and elevation of the drain inlets are determined in the development of the grading plans.

6-2.2.2 **Auxiliary Drains.** Auxiliary drains normally consist of any type or size drains provided to facilitate the removal of storm runoff but lacking sufficient capacity to remove the project design storm without excessive flooding or overflow. Auxiliary storm drains may be used in certain airfields to provide positive drainage of long flat swales located adjacent to runways or in unpaved adjacent areas. During less frequent storms of high intensity, excess runoff should flow overland to the primary drain system or other suitable outlet with a minimum of erosion. An auxiliary drain may also be installed to convey runoff from pavement gutters wherever a gutter capacity of less than design discharge is provided.

6-2.3 **Hydraulics of Storm Drainage Systems.** Hydraulic design of storm drainage systems requires an understanding of basic hydrologic and hydraulic concepts and principles. Hydrologic concepts were discussed earlier in this UFC. Important hydraulic principles include flow classification, conservation of mass, conservation of momentum, and conservation of energy. These elements are discussed in hydraulic texts. The following sections assume a basic understanding of these topics.

6-2.3.1 **Flow Type Assumptions.** The design procedures presented here assume that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform; however, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is conservative to design using the steady uniform flow assumption.

6-2.3.2 **Open Channel vs. Pressure Flow.** Two design philosophies exist for sizing storm drains under the steady uniform flow assumption. The first is referred to as open channel or gravity flow design. To maintain open channel flow, the segment must be sized so that the water surface within the conduit remains open to atmospheric pressure. For open channel flow, flow energy is derived from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). If the water surface throughout the conduit is to be maintained at atmospheric pressure, the flow depth must be less than the height of the conduit.

6-2.3.2.1 Pressure flow design requires that the flow in the conduit be at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within the conduit. In pressure flow, flow energy is again derived from the flow velocity, depth, and elevation. The significant difference here is that the pressure head will be above the top of the conduit, and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the hydraulic grade line. A detailed explanation of the hydraulic grade line is presented later in this chapter.

6-2.3.2.2 The question of whether open channel or pressure flow should control design has been debated. For a given flow rate, design based on open channel flow requires larger conduit sizes than those sized based on pressure flow. While it may be more expensive to construct storm drainage systems designed based on open channel flow, this design procedure provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. This factor of safety is often desirable since the methods of runoff estimation are not exact, and once placed, storm drains are difficult and expensive to replace.

6-2.3.2.3 There may be situations in which pressure flow design is desirable, however. For example, on some projects, there may be adequate headroom between the conduit and inlet/access hole elevations to tolerate pressure flow. In such a case, a significant cost savings may be realized over the cost of a system designed to maintain open channel flow. Also, in some cases it may be necessary to use an existing system that must be placed under pressure flow to accommodate the proposed design flow rates. In instances such as these, making a cursory hydraulic and economic analysis of a storm drain using both design methods before making a final selection may be advantageous.

6-2.3.2.4 Under most ordinary conditions, it is recommended that storm drains be sized based on a gravity flow criteria at flow full or near full. Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow. However, the designer should maintain an awareness that pressure flow design may be justified in certain instances. When pressure flow is allowed, special emphasis should be placed on the proper design of the joints so that they are able to withstand the pressure flow.

6-2.3.3 **Hydraulic Capacity**. The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced that define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's equation.

6-2.3.3.1 Manning’s equation was introduced in Chapter 3 for computing gutter capacity and the capacity for roadside and median channels. For circular storm drains flowing full, Manning's equation becomes:

$$V = \frac{0.59}{n} D^{0.67} S_o^{0.5} \quad Q = \frac{0.46}{n} D^{2.67} S_o^{0.5} \tag{6-1}$$

where:

V = mean velocity, ft/s

Q = rate of flow, ft^3/s

n = Manning's coefficient (Table 6-1)

D = storm drain diameter, ft

S_o = slope of the hydraulic grade line, ft/ft

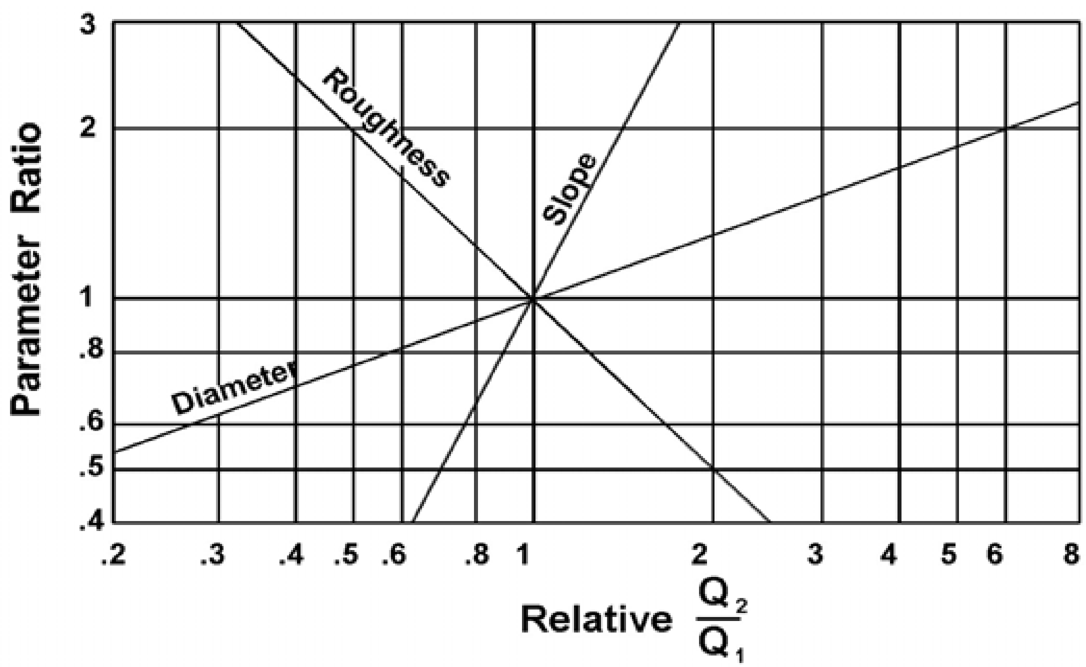
6-2.3.3.2 A nomograph solution of Manning's equation for full flow in circular conduits is presented in Chart 25 of Appendix B. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 6-1. Remember that the values in the table are for new pipe tested in a laboratory. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

Table 6-1. Manning's Coefficients for Storm Drain Conduits

Type of Pipe	Roughness or Corrugation	Manning's *n**
Concrete Pipe	Smooth	0.010-0.011
Concrete Boxes	Smooth	0.012-0.015
Spiral Rib Metal Pipe	Smooth	0.012-0.013
Corrugated Metal Pipe, Pipe-Arch, and Box (Annular or Helical Corrugations — see HDS-5, Manning's *n* varies with barrel size)	2.66 by 0.5 in. Annular	0.022-0.027
	2.66 by 0.5 in. Helical	0.011-0.023
	6 by 1 in. Helical	0.022-0.025
	5 by 1 in.	0.025-0.026
	3 by 1 in.	0.027-0.028
	6 by 2 in. Structural Plate	0.033-0.035
	9 by 2.5 in. Structural Plate	0.033-0.037
Corrugated Polyethylene	Smooth	0.009-0.015
Corrugated Polyethylene	Corrugated	0.018-0.025
Polyvinyl chloride (PVC)	Smooth	0.009-0.011
NOTE: The Manning's *n* values in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for storm drains may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.		

6-2.3.3.3 Figure 6-1 illustrates storm drain capacity sensitivity to the parameters in Manning's equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased by a factor of 6.0; if the slope is doubled, the capacity is increased by a factor of 1.4; however, if the roughness is doubled, the pipe capacity will be reduced by 50 percent.

Figure 6-1. Storm Drain Capacity Sensitivity



6-2.3.3.4 The hydraulic elements graph in Chart 26 of Appendix B is provided to assist in the solution of the Manning's equation for part-full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and illustrates the following important points:

- Peak flow occurs at 93 percent of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
- The velocity in a pipe flowing half-full is the same as the velocity for full flow.
- Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
- As the depth of flow drops below half-full, the flow velocity drops off rapidly.

6-2.3.3.5 The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape. Table 6-2 provides a tabular listing of the increase in capacity that can be achieved using alternate conduit shapes that have the same height as the original circular shape, but have a different cross-sectional area. Although these alternate shapes are generally more expensive then circular shapes, their use can be justified in some instances based on their increased capacity.

Table 6-2. Increase in Capacity of Alternate Conduit Shapes Based on a Circular Pipe with the Same Height

Shape	Area (Percent Increase)	Conveyance (Percent Increase)
Circular		
Oval	63	87
Arch	57	78
Box (*B* = *D*)	27	27

In addition to the nomograph in Chart 25 of Appendix B, numerous charts have been developed for conduits with specific shapes, roughness, and sizes.

6-2.3.3.5 **Example 6-1**

Given: $Q = 17.6\ \text{ft}^3/\text{s}$

$S_o = 0.015$ ft/ft

Find: The pipe diameter needed to convey the indicated design flow. Consider use of both concrete and helical corrugated metal pipes.

Solution:

Step 1. Concrete Pipe. Using Equation 6-1 or Chart 25 with $n = 0.013$ for concrete:

$$D = \left[\frac{(Qn)}{\left(0.46S_o^{0.5}\right)}\right]^{0.375}$$

$$D = \left[\frac{(17.6)(0.013)}{\left\{(0.46)(0.015)^{0.5}\right\}}\right]^{0.375}$$

$$D = 1.69 \text{ ft} = 20.3 \text{ in}$$

Use $D = 21$ in diameter standard pipe size.

Step 2. Helical Corrugated Metal Pipe. Using Equation 6-1 or Chart 25:

Assume $n = 0.017$

$$D = \left[\frac{(Qn)}{(0.46 S_o^{0.5})}\right]^{0.375}$$

$$D = \left[\frac{(17.6)(0.017)}{\{(0.46)(0.015)^{0.5}\}}\right]^{0.375}$$

D = 1.87 ft = 20.3 in

Use D = 24 in. diameter standard size. (***NOTE:*** The *n* value for 24 in. = 0.017. The pipe size and *n* value must coincide as shown in Table 6-1.)

6-2.3.3.6 **Example 6-2**

Given: The concrete and helical corrugated metal pipes in Example 6-1.

Find: The full flow pipe capacity and velocity.

Solution: Use Equation 6-1 or Chart 25.

Step 1. Concrete pipe:

$$Q = \left(\frac{0.46}{n} D^{2.67} S_o^{0.5}\right)$$

$$Q = \frac{(0.46)}{(0.013)} (1.75)^{2.67} (0.015)^{0.5}$$

Q = 19.3 ft^3/s

Step 2. Helical corrugated metal pipe:

$$Q = \left(\frac{0.46}{n} D^{2.67} S_o^{0.5}\right)$$

$$Q = \frac{(0.46)}{(0.017)} (2.0)^{2.67} (0.015)^{0.5}$$

Q = 21.1 ft^3/s

$$V = \left(\frac{0.59}{n} D^{0.67} S_o^{0.5}\right)$$

$$V = \frac{(0.59)}{(0.017)}(2.05)^{0.67}(0.015)^{0.5}$$

$$V = 6.8 \text{ ft/s}$$

6-2.3.4 **Energy Grade Line/Hydraulic Grade Line**. The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head, and pressure head. The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. To develop the EGL, it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses. The friction losses can be calculated using Manning's equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, *K*. Various tables and calculations exist for developing the value of *K* depending on the structure being evaluated for loss. Knowing the location of the EGL is critical to understanding and estimating the location of the hydraulic grade line (HGL).

6-2.3.4.1 The HGL is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. The HGL is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions.

6-2.3.4.2 The HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to pipe flow as well as open channel flow. Figure 6-2 illustrates the EGLs and HGLs for open channel and pressure flow in pipes.

6-2.3.4.3 When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface. When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, this condition is between open channel flow and pressure flow. At this condition, the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

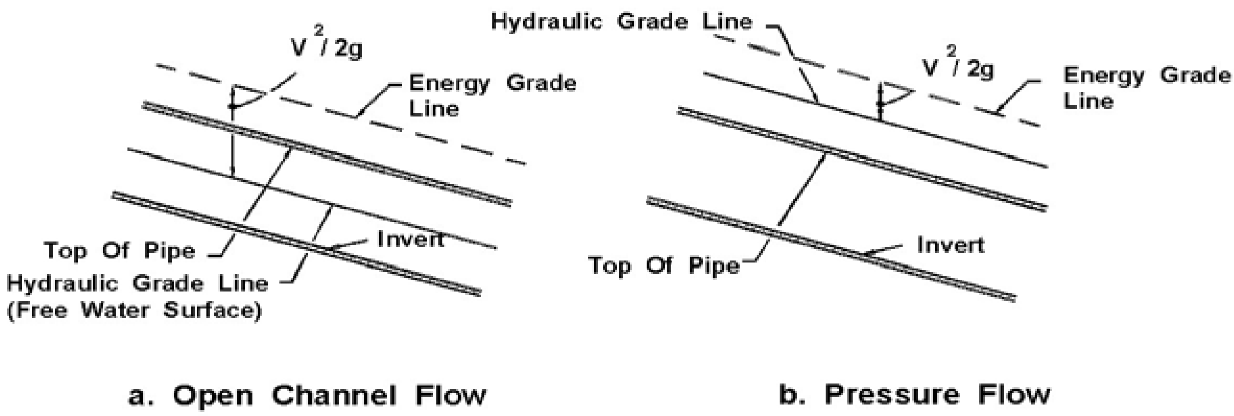


Figure 6-2. Hydraulic and Energy Grade Lines in Pipe Flow

6-2.3.4.4 Inlet surcharging and possible access hole lid displacement can occur if the HGL rises above the ground surface. A design based on open channel conditions must be planned carefully as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Often storm drainage systems can alternate between pressure and open channel flow conditions from one section to another.

6-2.3.4.5 A detailed procedure for evaluating the EGL and the HGL for storm drainage systems is presented later in this chapter.

6-2.3.5 **Storm Drain Outfalls.** All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel that is either existing or proposed for the purpose of conveying the storm water away from the highway. The procedure for calculating the EGL through a storm drainage system begins at the outfall; therefore, consideration of outfall conditions is an important part of storm drain design.

6-2.3.5.1 Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

6-2.3.5.2 The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If not, the water may need to be pumped or otherwise lifted to the elevation of the outfall.

6-2.3.5.3 The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the HGL for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet; however, the starting point for the HGL determination should be either the design tailwater elevation or the average of the critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

6-2.3.5.4 An exception to this rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever is highest.

6-2.3.5.5 If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed.

Table 6-3 provides a comparison of discharge frequencies for coincidental occurrence for a 10- and 100-yr design storm. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 500 acres and the storm drainage system has a drainage area of 5 acres, the ratio of receiving area to storm drainage area is 500 to 5, which equals 100 to 1. From Table 6-3 and considering a 10-yr design storm occurring over both areas, the flow rate in the main stream will be equal to that of a 5-yr storm when the drainage system flow rate reaches its 10-yr peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-yr peak flow rate, the flow rate from the storm drainage system will have fallen to the 5-yr peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

6-2.3.5.6 There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by using a pump station.

6-2.3.5.7 Energy dissipation may be required to protect the storm drain outlet. Protection is usually required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected. See HEC-14 for guidance with designing an appropriate dissipator.

Table 6-3. Frequencies for Coincidental Occurrence

Area Ratio	Frequencies for Coincidental Occurrence			
	10-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10,000 to 1	1	10	2	100
	10	1	100	2
1,000 to 1	2	10	10	100
	10	2	100	10
100 to 1	5	10	25	100
	10	5	100	25
10 to 1	10	10	50	100
	10	10	100	50
1 to 1	10	10	100	100
	10	10	100	100

6-2.3.5.8 The orientation of the outfall is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure cannot be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

6-2.3.6 **Energy Losses.** Prior to computing the HGL, estimate all energy losses in pipe runs and junctions. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The calculation of these losses is extremely important when designing the storm drain. If the storm drain design does not account for energy losses, the performance of the storm drain system is uncertain. HEC-22 has a comprehensive description of all of the energy losses and includes an example problem that demonstrates the application of some of these relationships. Refer to Chapter 7 of HEC-22.

6-2.4 **Design Guidelines and Considerations.** Design criteria and considerations describe the limiting factors that qualify an acceptable design. Several of these factors, including design and check storm frequency, time of concentration and discharge determination, allowable high water at inlets and access holes, minimum flow velocities, minimum pipe grades, and alignment, are explained in paragraphs 6-2.4.1 through 6-2.4.2.5.

6-2.4.1 **Design Storm Frequency.** The storm drain conduit is one of the most expensive and permanent elements within storm drainage systems. Storm drains normally remain in use longer than any other system elements. Once a storm drain is installed, increasing the capacity or repairing the line is very expensive. Consequently, the design flood frequency for projected hydrologic conditions should be selected to meet the need of the proposed facility both now and well into the future.

6-2.4.1.1 The design storm frequencies for DOD airfields and heliports, areas other than airfields, and FAA facilities are given in Chapter 2 of this UFC; however, exercise caution in selecting an appropriate storm frequency. Consider traffic volume, type and use of roadway, speed limit, flood damage potential, and the needs of the local community.

6-2.4.1.2 The highway community recommends designing storm drains that drain sag points where runoff can be removed only through the storm drainage system for a minimum 50-year frequency storm. The inlet at the sag point as well as the storm drain pipe leading from the sag point must be sized to accommodate this additional runoff. This can be done by computing the bypass occurring at each inlet during a 50-year rainfall and accumulating it at the sag point. Another method would be to design the upstream system for a 50-year design to minimize the bypass to the sag point. Evaluate each case on its own merits and assess the risk and impacts of flooding a sag point.

6-2.4.1.3 Following the initial design of a storm drainage system, it is prudent to evaluate the system using a higher check storm. Check storms are also explained in Chapter 2. Often for roadway design, a 100-year frequency storm is recommended for the check storm. The check storm is used to evaluate the performance of the storm drainage system and determine if the major drainage system is adequate to handle the flooding from a storm of this magnitude. Again, review local criteria.

6-2.4.2 **Time of Concentration and Discharge.** The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less than this total. The Rational Method is the most common means of determining design discharges for storm drain design. The time of concentration is very influential in determining the design discharge using the Rational Method. The time of concentration is the period required for water to travel from the most hydraulically distant point of the watershed to the point of interest. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration for inlet spacing is the time required for water to flow from the hydraulically most distant point of the unique drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. If the total time of concentration to the upstream inlet is less than 5 minutes, a minimum time of concentration of 5 minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

6-2.4.2.1 The time of concentration for pipe sizing is the time required for water to travel from the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest.

6-2.4.2.2 The flow path with the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small, relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the high runoff coefficient (*C* value) and high intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

6-2.4.2.3 The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighted *C* value and the intensity associated with the longest time of concentration.

- Second, calculate the runoff using only the smaller, less pervious area. The typical procedure would be followed using the *C* value for the small less pervious area and the intensity associated with the shorter time of concentration.

Compare the results of these two calculations and use the largest value of discharge for design.

6-2.4.2.4 The second exception exists when a smaller, less pervious area is tributary to the larger primary watershed. When this occurs, two sets of calculations should also be made.

- First, calculate the runoff from the total drainage area with its weighted *C* value and the intensity associated with the longest time of concentration.

- Second, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time as the peak from the smaller, less pervious tributary area. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, use the intensity associated with the time of concentration from the smaller, less pervious area. The portion of the larger primary area to be considered is determined by this equation:

$$A_c = A\frac{t_{c1}}{t_{c2}} \tag{6-2}$$

A_c is the most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area. A is the area of the larger primary area, t_{c1} is the time of concentration of the smaller, less pervious tributary area, and t_{c2} is the time of concentration associated with the larger primary area as is used in the first calculation. The C value to be used in this computation should be the weighted C value that results from combining C values of the smaller, less pervious tributary area and the area A_c. The area to be used in the Rational Method is the area of the less pervious area plus A_c. This second calculation should be considered only when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

6-2.4.2.5 Finally, compare the results of these calculations and use the largest value of discharge for design.

6-2.4.3 **Maximum Highwater.** Maximum highwater is the maximum allowable elevation of the water surface (HGL) at any given point along a storm drain. These points include inlets, access holes, or any place where there is access from the storm drain to the ground surface. The maximum highwater at any point should not interfere with the intended functioning of an inlet opening or reach an access hole cover. Maximum allowable highwater levels should be established along the storm drainage system prior to initiating hydraulic evaluations.

6-2.4.4 **Minimum Velocity and Grades.** It is desirable to maintain a self-cleaning velocity in the storm drain to prevent deposition of sediments and subsequent loss of capacity. For this reason, storm drains should be designed to maintain full-flow pipe velocities of 3 ft/s or greater. A review of the hydraulic elements in Chart 26 (Appendix B) indicates that this criteria results in a minimum flow velocity of 2 ft/s at a flow depth equal to 25 percent of the pipe diameter. Minimum slopes required for a velocity of 3 ft/s can be computed using the form of Manning's formula in Equation 6-3. Alternately, use values in Table 6-4.

$$S = 2.67\left(\frac{nV}{D^{0.67}}\right)^2 \tag{6-3}$$

where:

D = in feet when using Equation 6-3

Table 6-4. Minimum Pipe Slopes to Ensure 3.0 ft/s Velocity in Storm Drains Flowing Full

Pipe Size, in.	Full Pipe Flow, ft^3/s	Minimum Slopes, ft/ft		
		n = 0.012	*n* = 0.013	*n* = 0.024
8	1.1	0.0064	0.0075	0.0256
10	1.6	0.0048	0.0056	0.0190
12	2.4	0.0037	0.0044	0.0149
15	3.7	0.0028	0.0032	0.0111
18	5.3	0.0022	0.0026	0.0087
21	7.2	0.0018	0.0021	0.0071
24	9.4	0.0015	0.0017	0.0059
27	11.9	0.0013	0.0015	0.0051
30	14.7	0.0011	0.0013	0.0044
33	17.8	0.0010	0.0011	0.0039
36	21.2	0.0009	0.0010	0.0034
42	28.9	0.0007	0.0008	0.0028
48	37.7	0.0006	0.0007	0.0023
54	47.7	0.0005	0.0006	0.0020
60	58.9	0.0004	0.0005	0.0017
66	71.3	0.0004	0.0005	0.0015
72	84.8	0.0003	0.0004	0.0014

6-3 **PRELIMINARY DESIGN PROCEDURE.** The preliminary design of storm drains can be accomplished by using the following steps and the storm drain computation sheet in Figure 6-3. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the EGL and HGL computations have been completed.

6-3.1 **Step 1.** Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

a. Location of Storm Drains

(1) Preliminary Layout**.** Prepare a preliminary map (scale 1 in. = 200 ft or larger) showing the outlines of roadways, runways, taxiways, and parking aprons. Contours should represent approximately the finished grade for

the airfield, heliport, or roadway facility. Details of grading, including ponding basins around primary drain inlets, need not be shown more accurately than with 1-ft contour intervals.

(2) Profiles. Plot profiles of all roadways, or runways, taxiways, and aprons so that elevations controlling the grading of intermediate areas may be determined readily at any point.

b. Direction of Flow. Avoid drainage patterns consisting of closely spaced interior inlets in pavements with intervening ridges for airfields. Such grading may cause taxiing problems, including bumping or scraping of wing tanks. Crowned sections are the standard cross sections for roadways, runways, taxiways, and safety areas. Crowned sections generally slope each way from the center line of the runway on a transverse grade to the pavement. Although crowned grading patterns result in the most economical drainage, adjacent pavements, topographic considerations, or other matters may necessitate other pavement grading.

c. Location of Access Holes and Other Structures

(1) Drain Outlets. Consider the limiting grade elevations and feasible channels for the collection and disposition of the storm runoff. Select the most suitable locations for outlets of drains serving various portions of the field. Then select a tentative layout for primary storm drains. The most economical and most efficient design is generally obtained by maintaining the steepest hydraulic gradient attainable in the main drain and maintaining approximately equal lateral length on each side of the main drain.

(2) Cross-sectional Profiles of Intermediate Areas. Assume the location of cross-sectional profiles of intermediate areas. Plot data showing controlling elevations and indicate the tentatively selected locations for inlets by means of vertical lines. See Chapter 3 for guidance on the preliminary location of inlets. To facilitate a comparison of the elevations of intermediate areas with those of paved areas, projections of roadways, runways, taxiways, or aprons for limited distances should be shown on the profiles. Generally, one cross-sectional profile should follow each line of the underground storm drain system. Other profiles should pass through each of the inlets at approximately right angles to paved roadways, runways, taxiways, or aprons.

(3) Correlation of the Controlling Elevations and Limiting Grades. Begin at points corresponding to the controlling elevations, such as the edges of runways, and sketch the ground profile from the given points to the respective drain inlets. Make the grades conform to the limiting slopes. Review the tentative grading and inlet elevations and make such

adjustments in the locations of drain inlets and in grading details as necessary to obtain the most satisfactory general plan.

d. Number or Label Assigned to Each Structure

e. Location of All Existing Utilities (e.g., water, sewer, gas, underground cables)

(1) Trial Drainage Layouts**.** Several trial drainage layouts will be necessary before the most economical system can be selected. The first consideration will be the tentative layout serving all of the depressed areas in which overland flow will accumulate. The inlet structures will be located, during the initial step, at the lowest points within the field areas. The pipelines will be shown next. Each of the inlet structures will be connected to the field pipelines, which in turn will be connected to the major outfalls.

(2) Rechecking of Finished Contours. Before proceeding further, recheck the finished contours to determine whether the surface flow is away from the paved areas, that the flow is not directed across them, that no field structures fall within the paved areas (except in aprons), that possible ponding areas are not adjacent to pavement edges, and that surface water will not have to travel excessively long distances to flow into the inlets. If there is a long, gradually sloping swale between a runway and its parallel taxiway (in which the longitudinal grade, for instance, is all in one direction), additional inlets should be placed at regular intervals down this swale. Should this be required, ridges may be provided to protect the area around the inlet, prevent bypassing, and facilitate the entry of the water into the structure. If the ridge area is within the runway safety area, the grades and grade changes will need to conform to the limitations established for runway safety areas in other pertinent publications.

(3) Maximum Spread and Ponding. Estimate the maximum elevation of storage permissible in the various ponding areas and check the elevations against the profiles. Ponding requirements for airfields and heliports are provided in Chapter 2. Scale the distances from the respective drain inlets to the point where the elevation of maximum permissible ponding intersects the ground line, transfer the scaled distances to the map prepared in (1) above, and sketch a line through the plotted points to represent the boundary of the maximum ponding area during the design storm. Criteria for allowable width of spread for roadways is provided in Chapter 3.

(4) Ditches. A system of extensive peripheral ditches may become an integral part of the drainage system. Ditch size and function are variable. Some ditches carry the outfall away from the pipe system and drainage areas into the natural drainage channels or into existing water courses. Others receive outfall flow from the airport site or adjacent terrain. Open

ditches are subject to erosion if their gradients are steep and if the volume of flow is large. When necessary, the ditches may be turfed, sodded, stabilized, or lined to control erosion. A complete explanation of median drainage can be found in Chapter 3. Stable channel design is detailed in Chapter 5.

(5) Study of the Contiguous Areas. After the storm drain system has been tentatively laid out and before the actual computations have been started, the areas contiguous to the graded portion of the airport that may contribute surface flow upon it should again be studied. A system of open channels, intercepting ditches, or storm drains should be designed where necessary to intercept this storm flow and conduct it away from the facility to convenient outfalls. A study of the soil profiles will assist in locating porous strata that may be conducting subsurface water into the airport. If this condition exists, the subsurface water should be intercepted and diverted.

Figure 6-3. Preliminary Storm Drain Computation Sheet

COMPUTED BY ______ DATE ______
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ROUTE ______
SECTION ______
COUNTY ______

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	( )	INC ( )	TOTAL ( )		INC. ( )	TOTAL ( )	INLET (min)	SYSTEM (min)	(/hr)	(^{3} /s)	( )	(^{3}/s)	FULL (/s)	DESIGN (/s)	(min)	U/S ( )	D/S ( )	( )	(/)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

6-3.2 **Step 2***.* Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system. Use the completed grading plan as a guide and sketch the boundaries of specific drainage areas tributary to their respective drain inlets. Compute the area of paved and unpaved areas tributary to the respective inlets.

- Drainage areas
- Runoff coefficients
- Travel time

6-3.3 **Step 3**. Using the information generated in Steps 1 and 2, complete the following information on the design form for each run of pipe starting with the upstream-most storm drain run:

- FROM and TO stations, Columns 1 and 2.
- LENGTH of run, Column 3.
- INC. drainage area, Column 4. The incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.
- RUNOFF COEFF. "C," Column 6. The runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases, a composite runoff coefficient will need to be computed.

6-3.4 **Step 4**. Using the information from Step 3, compute this information:

- TOTAL area, Column 5. Add the incremental area in Column 4 to the previous section's total area and place this value in Column 5.
- INC. "AREA" X "C," Column 7. Multiply the drainage area in Column 4 by the runoff coefficient in Column 6. Put the product, *CA*, in Column 7.
- TOTAL "AREA" X "C," Column 8. Add the value in Column 7 to the value in Column 8 for the previous storm drain run, and put this value in Column 8.
- RAIN "I," Column 11. Using the larger of the two times of concentration in Columns 9 and 10, and an IDF curve, determine the rainfall intensity, *I*, and place this value in Column 11.
- RUNOFF "Q," Column 12. Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.

- SLOPE, Column 21. Place the pipe slope value in Column 21. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.

- PIPE DIA., Column 13. Size the pipe using relationships and charts presented in Chapter 4 to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to a full gravity flow. Since most calculated sizes will not be available, a nominal size will be used. The designer will decide whether to go to the next larger size and have part-full flow or whether to go to the next smaller size and have pressure flow.

- Q (CAPACITY) FULL, Column 14. Compute the full flow capacity of the selected pipe using Equation 6-1, and put this information in Column 14.

- VELOCITY, Columns 15 (FULL) and 16 (DESIGN). Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from $V = Q/A$, Equation 6-1, or Chart 25 (Appendix B). If the pipe is not flowing full, the velocity can be determined from Chart 26.

- SEC (SECTION) TIME, Column 17. Calculate the travel time in the pipe section by dividing the pipe length (Column 3) by the design flow velocity (Column 16). Place this value in Column 17.

- CROWN DROP, Column 20. Calculate an approximate crown drop at the structure to off-set potential structure energy losses using Equation 7-9 of HEC-22.

- INVERT ELEV., Columns 18 and 19. Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this section of pipe, including any pipe size changes that occurred along the section.

6-3.5 **Step 5**. Repeat steps 3 and 4 for all pipe runs to the storm drain outlet. Use equations and nomographs to accomplish the design effort.

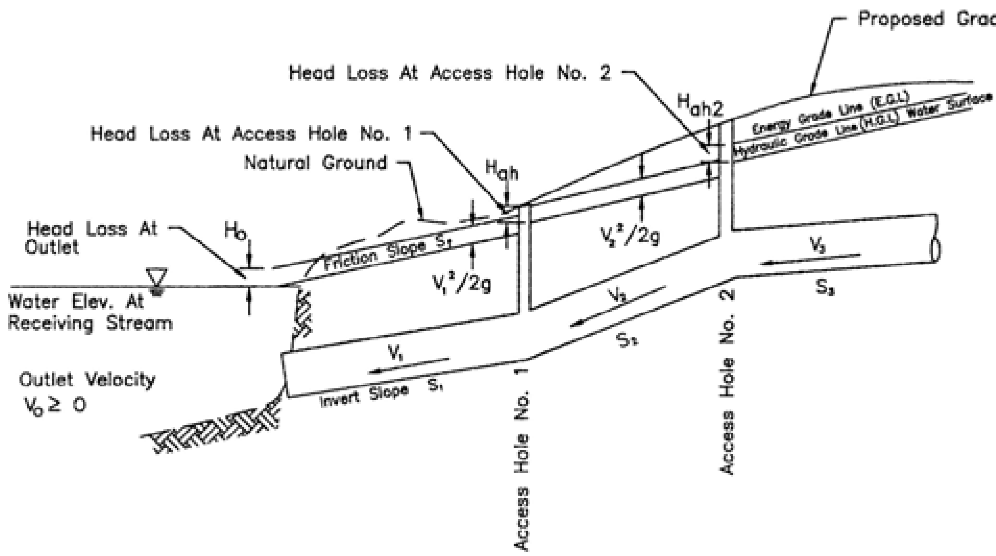
6-3.6 **Step 6**. Check the design by calculating the EGL and HGL as described in section 6-4.

An example of storm drain sizing and layout is provided in Chapter 7 of HEC-22.

6-4 **ENERGY GRADE LINE EVALUATION PROCEDURE.** This section presents a step-by-step procedure for manual calculation of the EGL and the HGL using the energy loss method. For most storm drainage systems, computer methods such as HYDRA are the most efficient means of evaluating the EGL and HGL; however, it is important that the designer understand the analysis process to better interpret the output from computer-generated storm drain designs.

6-4.1 Figure 6-4 provides a sketch illustrating the use of the two grade lines in developing a storm drainage system. The step-by-step procedure in paragraph 6-4.3 can be used to manually compute the EGL and HGL. The computation tables in Figure 6-5 and Figure 6-6 can be used to document this procedure.

Figure 6-4. Energy and Hydraulic Grade Line Illustration



6-4.2 Before beginning the computational steps in the procedure, it is important to understand the organization of data on the form. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines, the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity).

Table A (Figure 6-5) is used to calculate the HGL and EGL elevations, while table B (Figure 6-6) is used to calculate the pipe losses and structure losses. Values obtained in Table B are transferred to Table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

6-4.3 EGL computations begin at the outfall and are worked upstream, taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime. Again, HEC-22 includes a complete example that works through these steps.

NOTE: In the EGL computational procedure, values obtained in Table B are transferred to Table A for use during the design procedure. **In the step-by-step description, a column number will be followed by a letter A or B to indicate the appropriate table to be used.**

6-4.3.1 **Step 1**. The first line of Table A includes information on the system beyond the end of the conduit system. Define this as the stream, pool, existing system, etc., in Column 1A. Determine the EGL and HGL for the downstream receiving system. If this is a natural body of water, the HGL will be at the water surface. The EGL will also be at the water surface if no velocity is assumed or will be a velocity head above the HGL if there is a velocity in the water body. If the new system is being connected to an existing storm drain system, the EGL and the HGL will be that of the receiving system. Enter the HGL in Column 14A and the EGL in Column 10A of the first line on the computation sheet.

6-4.3.2 **Step 2**. Identify the structure number at the outfall (this may be just the end of the conduit, but it needs a structure number), the top of conduit (TOC) elevation at the outfall end, and the surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 15A, and 16A, respectively. Also, add the structure number in Column 1B.

6-4.3.3 **Step 3**. Determine the EGL just upstream of the structure identified in Step 2. Two different cases exist when the conduit is flowing full:

- Case 1: If the tailwater at the conduit outlet is greater than $(d_c + D)/2$, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions.

- Case 2: If the tailwater at the conduit outlet is less than $(d_c + D)/2$, the EGL will be the HGL plus the velocity head for the conduit flow conditions. The equivalent HGL, EHGL, will be the invert plus $(d_c + D)/2$.

The velocity head needed in either Case 1 or 2 will be calculated in the next steps, so it may be helpful to complete Step 4 and work Step 5 to the point where velocity head (Column 7A) is determined and then come back and finish this step. Enter the EGL in Column 13A.

NOTE: The values for d_c for circular pipes can be determined from Chart 27. Charts for other conduits or other geometric shapes can be found in HDS-5. Note that the value of d_c cannot be greater than the height of the conduit.

Figure 6-5. Energy Grade Line Computation Sheet - Table A

COMPUTED BY ________ DATE ________
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PAGE ________ OF ________
INITIAL TAILWATER ELEV. ____________

ROUTE ________________
SECTION ________________
COUNTY ________________

Str. ID (1)	D ( ) (2)	Q (^{3}/s) (3)	L ( ) (4)	V (/s) (5)	d ( ) (6a)	d_c ( ) (6b)	$V^2/2g$ ( ) (7)	S_f (/) (8)	Total Pipe Loss (table B) ( ) (9)	EGL_o ( ) (10)	K (table B) (11)	$K(V^2/2g)$ ( ) (12)	EGL_i ( ) (13)	HGL ( ) (14)	U/S TOC ( ) (15)	Surf. Elev. ( ) (16)

Figure 6-6. Energy Grade Line Computation Sheet - Table B

COMPUTED BY ______ DATE ______
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ROUTE ______
SECTION ______
COUNTY ______

	Pipe Losses ( )						Structure Losses ( )							
Str. ID (1)	H_f (2)	h_b (3)	H_o (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)

6-4.3.4 **Step 4**. Identify the structure for the junction immediately upstream of the outflow conduit (for the first conduit) or immediately upstream of the last structure (if working with subsequent lines) and enter this value in Column 1A and Column 1B of the next line on the computation sheets. Enter the conduit diameter (*D*) in Column 2A, the design discharge (*Q*) in Column 3A, and the conduit length (*L*) in Column 4A.

6-4.3.5 **Step 5**. If the barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Put "full" in Column 6a and not applicable (n/a) in Column 6b of Table A. Continue with Step 6. If the barrel flows only partially full, continue with Step 5A.

NOTE: If the pipe is flowing full because of high tailwater or because the pipe has reached its capacity for the existing conditions, the velocity will be computed based on continuity using the design flow and the full cross-sectional area. Do not use the full flow velocity determined in Column 15 of the Preliminary Storm Drain Computation Sheet (Figure 6-3) for part-full flow conditions. For part-full conditions defined in Step 5, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL and HGL calculations.

6-4.3.5.1 **Step 5A**. Part-full flow: Using the hydraulic elements graph in Chart 26 with the ratio of part-full to full flow (values from the Preliminary Storm Drain Computation Sheet, Figure 6-3), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5, respectively, of Table A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

6-4.3.5.2 **Step 5B.** Compute the critical depth for the conduit using Chart 27. (If the conduit is not circular, see HDS-5 for additional charts.) Enter this value in Column 6b of Table A.

6-4.3.5.3 **Step 5C**. Compare the flow depth in Column 6a (Table A) with the critical depth in Column 6b (Table A) to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical; continue with Step 6. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical; continue with Step 5D. In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for super critical flow in the upstream section of pipe, ensure that the EGL is higher in the pipe than in the structure.

6-4.3.5.4 **Step 5D**. Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.

6-4.3.5.5 **Step 5E**. Enter the structure ID for the next upstream structure on the next line in Column 1A and Column 1B. Enter the pipe diameter (*D*), discharge (*Q*), and conduit length (*L*) in Columns 2A, 3A, and 4A, respectively, of the same line.

NOTE: After a downstream pipe has been determined to flow in supercritical flow, it is necessary to check each succeeding upstream pipe for the type of flow. This is done by calculating normal depth and critical depth for each pipe. If normal depth is less than the diameter of the pipe, the flow will be open channel flow and the critical depth calculation can be used to determine whether the flow is sub or supercritical. If the flow line elevation through an access hole drops enough that the invert of the upstream pipe is not inundated by the flow in the downstream pipe, the designer goes back to Column 1A and begins a new design as if the downstream section did not exist.

6-4.3.5.6 **Step 5F**. Compute the normal depth for the conduit using Chart 26 and the critical depth using Chart 27. (If the conduit is not circular, see HDS-5 for additional charts.) Enter these values in Columns 6a and 6b of Table A.

6-4.3.5.7 **Step 5G**. If the pipe barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Go to Step 3, Case 2 to determine the EGL at the outlet end of the pipe. Put this value in Column 10A and go to Step 6. For part-full flow, continue with Step 5H.

6-4.3.5.8 **Step 5H**. Part-full Flow: Compute the velocity of flow in the conduit and enter this value in Column 5A. Compute the velocity head ($V^2/2g$) and place the value in Column 7A.

6-4.3.5.9 **Step 5I**. Compare the flow depth in Column 6a with the critical depth in Column 6b to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical; continue with Step 5J. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical; continue with Step 5K.

6-4.3.5.10 **Step 5J**. Subcritical Flow Upstream: Compute the EGL at the outlet of the structure (EGL_o) at the outlet of the previous structure as the outlet invert plus the sum of the outlet pipe flow depth and the velocity head. Place this value in Column 10A of the appropriate structure and go to Step 9.

6-4.3.5.11 **Step 5K**. Supercritical Flow Upstream: Access hole losses do not apply when the flow in two successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 15B of the intermediate structure (previous line). The HGL at the structure is equal to the pipe invert elevation plus the flow depth. Check the invert elevations and the flow depths both upstream and downstream of the structure to determine where the highest HGL exists. The highest value should be placed in Column 14A of the previous structure line. Perform Steps 20 and 21 and then repeat Steps 5E through 5K until the flow regime returns to subcritical. If the next upstream structure is end-of-line, skip to Step 10B and then perform Steps 20, 21, and 24.

6-4.3.6 **Step 6**. Compute the friction slope (S_f) for the pipe: $S_f = H_f / L = [Q\,n / (0.46\,D^{2.67})]^2$

Enter this value in Column 8A of the current line. This equation assumes full flow in the outlet pipe. If full flow does not exist, set the friction slope equal to the pipe slope.

6-4.3.7 **Step 7**. Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_o), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) using Equations 7-5 through 7-8 of HEC-22 and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Columns 7B and 9A.

6-4.3.8 **Step 8**. Compute the EGL value at the outlet of the structure (EGL_o) as the EGL for an inflow pipe (EGL_i) elevation from the previous structure (Column 13A) plus the total pipe losses (Column 9A). Enter the EGL_o value in Column 10A.

6-4.3.9 **Step 9**. Estimate the depth of water in the access hole (estimated as the depth from the outlet pipe invert to the HGL in the pipe at the outlet). It is computed as EGL_o (Column 10A) minus the pipe velocity head in Column 7A minus the pipe invert elevation (from the Preliminary Storm Drain Computation Sheet, Figure 6-3). Enter this value in Column 8B. If supercritical flow exists in this structure, leave this value blank and skip to Step 5E.

6-4.3.10 **Step 10**. If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equation 7-10 and Equation 7-11 of HEC-22. Start by computing the initial structure head loss coefficient (K_o) based on the relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5:

6-4.3.10.1 **Step 10A**. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in Equation 7-10 to K_e as reported in Table 7-5b of HEC-22. Enter this value in Columns 15B and 11A and continue with Step 17. Add a note on Table A indicating that this is a drop structure.

6-4.3.10.2 **Step 10B**. If the outflow pipe functions under inlet control, compute the depth in the access hole (HGL) using Chart 28 or 29 (Appendix B). If the storm conduit shape is other than circular, select the appropriate inlet control nomograph from HDS-5. Add these values to the access hole invert to determine the HGL. Since the velocity in the access hole is negligible, the EGL and HGL are the same. Enter the HGL in Column 14A and the EGL in Column 13A. Add a note on Table A indicating that this is a drop structure. Go to Step 20.

6-4.3.11 **Step 11**. Using Equation 7-13 of HEC-22, compute the correction factor for pipe diameter (C_D) and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

6-4.3.12 **Step 12**. Using Equation 7-14 of HEC-22, compute the correction factor for flow depth (C_d) and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

6-4.3.13 **Step 13**. Using Equation 7-15 of HEC-22, compute the correction factor for relative flow (C_Q) and enter this value in Column 12B. This factor equals 1.0 if there are less than 3 pipes at the structure.

6-4.3.14 **Step 14**. Using Equation 7-16 of HEC-22, compute the correction factor for plunging flow (C_P) and enter this value in Column 13B. This factor equals 1.0 if there is no plunging flow. This correction factor is only applied when $h > d_{aho}$.

6-4.3.15 **Step 15**. Enter in Column 14B the correction factor for benching (C_B) as determined from Table 7-6 of HEC-22. Linear interpolation between the two columns of values will most likely be necessary.

6-4.3.16 **Step 16**. Using Equation 7-11 of HEC-22, compute the value of *K* and enter this value in Columns 15B and 11A.

6-4.3.17 **Step 17**. Compute the total access hole loss (H_{ah}) by multiplying the *K* value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

6-4.3.18 **Step 18**. Compute EGL_i at the structure by adding the structure losses in Column 12A to the EGL_o value in Column 10A. Enter this value in Column 13A.

6-4.3.19 **Step 19**. Compute the HGL at the structure by subtracting the velocity head in Column 7A from the EGL_i value in Column 13A. Enter this value in Column 14A.

6-4.3.20 **Step 20**. Determine the top of conduit (TOC) value for the inflow pipe (using information from the Preliminary Storm Drain Computation Sheet, Figure 6-3) and enter this value in Column 15A.

6-4.3.21 **Step 21**. Enter the ground surface, top of grate elevation, or other high water limits at the structure in Column 16A. If the HGL value in Column 14A exceeds the limiting elevation, design modifications will be required.

6-4.3.22 **Step 22**. Enter the structure ID for the next upstream structure in Columns 1A and 1B of the next line. When starting a new branch line, skip to Step 24.

6-4.3.23 **Step 23**. Continue to determine the EGL through the system by repeating Steps 4 through 23. (Begin with Step 2 if working with a drop structure. This begins the design process again as if there were no system downstream from the drop structure.)

6-4.3.24 **Step 24**. When starting a new branch line, enter the structure ID for the branch structure in Columns 1A and 1B of a new line. Transfer the values from Columns 2A through 10A and 2B to 7B associated with this structure on the main branch run to the corresponding columns for the branch line. If flow in the main storm drain at the branch point is subcritical, continue with Step 9; if it is supercritical, continue with Step 5E.