Chapter 6 - Hydraulics

6.1 Hydraulic Design

Introduction

The process of designing drainage facilities, including culverts and pipelines, consists of two distinct functions. The engineer must determine the maximum volume of flow to be transported by the drainage facility, and the type and size of drainage structure that will transport that maximum volume of flow.

Many different procedures are available to determine design flow and to size drainage structures. Numerous texts and manuals have been developed to guide the design engineer. In addition, many agencies for which drainage facilities are being designed have developed standard procedures for hydrologic analysis and drainage structure design. Because practice varies from state to state and often within states, this chapter is not intended to serve the full function of a design manual, but rather it is intended to identify procedures for determining design flow and for sizing drainage structures. A description of various flow and pipe sizing methodologies is provided and manuals or texts that include detailed design procedures are referenced.

6.2 Hydraulics of Ditches and Open Channels

Open channel flow is often considered to be uniform flow for the purpose of design calculations. This assumption requires that the depth of flow in a prismatic channel be constant along the length of the channel. In practice, uniform flow rarely occurs. This is because of slight variations in channel shape, channel slope, or channel roughness. However, the error resulting from this assumption is relatively minimal in most instances.

The flow of water in an open channel must adhere to the law of conservation of energy and the law of conservation of mass. Conservation of energy will be discussed later in this chapter. Conservation of mass requires that mass be neither created nor destroyed. An outcome of this requirement for water is the continuity equation and when applied to a hydraulic system with a constant discharge, Q, the continuity equation as shown in Eqn. 6.1:

$$Q = AV \qquad \text{(Eqn. 6-1)}$$

where:

Q = Discharge, cfs (cms);
A = Cross-sectional area of flow, ft^2 (cm^2); and,
V = Average velocity of flow, ft/s (m/s).

Chezy equation

The Chézy equation was the first equation proposed for uniform flow and was developed in the late 1700's by Antoine Chézy in France. The equation is of the form is shown in Eqn. 6.2:

$$V = C\sqrt{R}\,S \qquad \text{(Eqn. 6.2)}$$

where:

V = Velocity, ft/s (m/s);
C Chézy constant;
R = Hydraulic radius = Area/Wetted Perimeter, ft (m); and,
S = Channel slope, ft/ft (m/m).

Many researchers have attempted to develop relationships for the Chézy constant. The two which are most frequently used are the Kutter Equation and the Manning's equation.

Kutter Equation

Building on the work of Chézy, Wilhelm Kutter developed the following relationship for the Chézy constant shown in Eqn. 6.3:

$$C = \frac{41.65+\frac{0.00281}{S}+\frac{1.811}{n}}{1+\frac{n}{\sqrt{R}}\left(41.65+\frac{0.00281}{S}\right)} \qquad \text{(Eqn. 6.3)}$$

where:

C = Chézy constant;
R = Hydraulic radius = Area/Wetted Perimeter, ft (m);
S = Channel slope, ft/ft (m/m); and,
n = Channel roughness coefficient.

Manning's Equation

Robert Manning also developed a relationship for determining the Chézy constant. The Manning's equation is shown in Eqn. 6.4:

$$C = \frac{1.486}{n} R^{1/6} \qquad \text{(Eqn. 6.4)}$$

This form can be substituted into the Chézy equation to derive the most common equation for uniform open channel flow in the United States, called the Manning's Eqn. 6.5.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(Eqn. 6.5)}$$

And by direct substitution of the continuity equation, Q = AV, the Manning's equation can be established for discharge, Q, in lieu of velocity, as included in Eqn. 6.6:

$$Q = A\,\frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(Eqn. 6.6)}$$

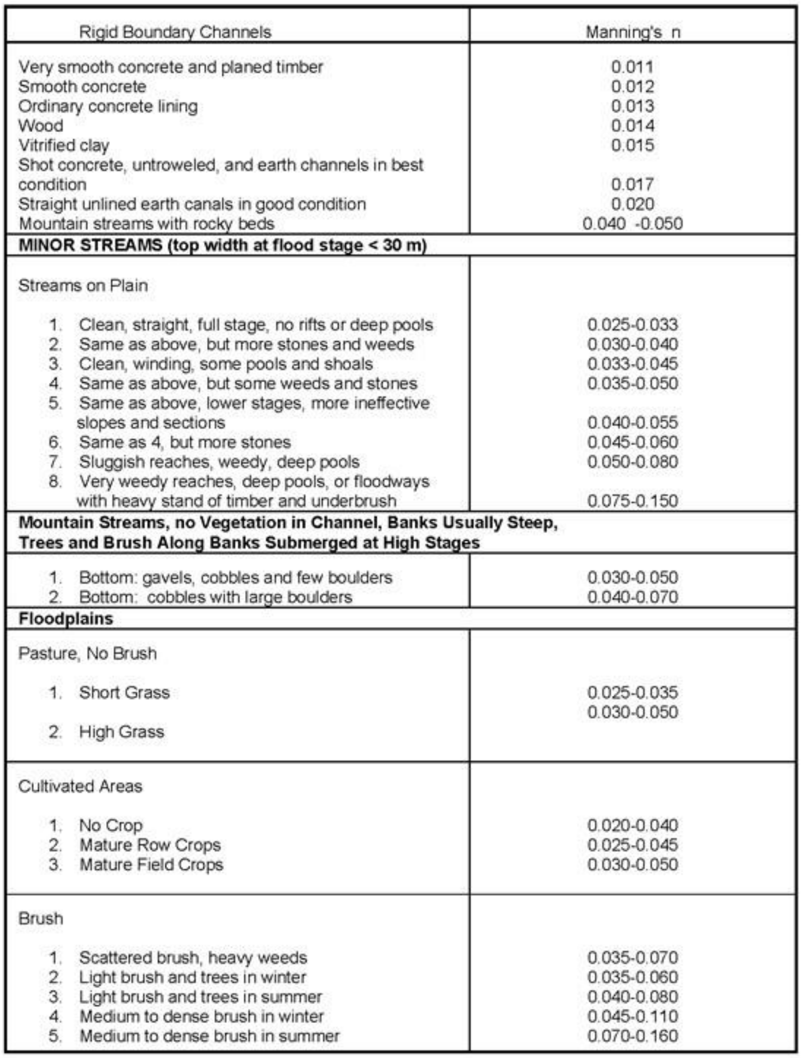
Both the Kutter and Manning's equations utilize the roughness coefficient. This is an empirically-derived value that indicates the relative roughness of a channel lining. Typical values of roughness coefficients for various channel surfaces are provided in

and typical roughness coefficients for conduits are shown in **Error! Reference source not found.**.

The values for roughness coefficient that are presented in Tables 6.1 and 6.2 are considered to be design values. Full scale hydraulic testing of pipes has exhibited values lower than those presented; however, there can be significant differences in the actual roughness coefficient of a pipe. These differences arise from varying depths of flows, from the joint gap between successive lengths of pipe, from differing pipe diameters, and from a number of additional factors. In smooth interior pipe, differences can also arise from liner undulations resulting from pipe manufacturing and circumferential shortening during loading. However, these variances have all been shown to be well-accounted for in the design values presented in Tables 6.1 and 6.2.

The Federal Highway Administration Hydraulic Design Series Number 3 "Design Charts for Open-Channel Flow" (1961) provides nomographs for many open channel shapes, including trapezoidal, rectangular, and triangular, and all are based on the Manning's equation. Each nomograph is populated with limited typical roughness coefficient values. An example of one of these nomographs is provided in Figure 6.1.

Table 6.1: Typical values of Manning's n for channels (FHWA HDS No. 4 1996)



Rigid Boundary Channels	Manning's n
Very smooth concrete and planed timber	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Wood	0.014
Vitrified clay	0.015
Shot concrete, untroweled, and earth channels in best condition	0.017
Straight unlined earth canals in good condition	0.020
Mountain streams with rocky beds	0.040 -0.050
MINOR STREAMS (top width at flood stage < 30 m)	
Streams on Plain	
1. Clean, straight, full stage, no rifts or deep pools	0.025-0.033
2. Same as above, but more stones and weeds	0.030-0.040
3. Clean, winding, some pools and shoals	0.033-0.045
4. Same as above, but some weeds and stones	0.035-0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040-0.055
6. Same as 4, but more stones	0.045-0.060
7. Sluggish reaches, weedy, deep pools	0.050-0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075-0.150
Mountain Streams, no Vegetation in Channel, Banks Usually Steep, Trees and Brush Along Banks Submerged at High Stages	
1. Bottom: gavels, cobbles and few boulders	0.030-0.050
2. Bottom: cobbles with large boulders	0.040-0.070
Floodplains	
Pasture, No Brush	
1. Short Grass	0.025-0.035 0.030-0.050
2. High Grass	
Cultivated Areas	
1. No Crop	0.020-0.040
2. Mature Row Crops	0.025-0.045
3. Mature Field Crops	0.030-0.050
Brush	
1. Scattered brush, heavy weeds	0.035-0.070
2. Light brush and trees in winter	0.035-0.060
3. Light brush and trees in summer	0.040-0.080
4. Medium to dense brush in winter	0.045-0.110
5. Medium to dense brush in summer	0.070-0.160

Table 6.2: Typical values of Manning's n for conduits (FHWA HDS No.5 1985)

Material	Manning's n
Concrete Pipe	0.010-0.015
Corrugated Metal Pipe	0.011-0.025
Spiral Rib Metal Pipe	0.012-0.013
Corrugated Metal Structural Plate	0.033-0.037
Smooth Interior Corrugated Plastic	0.009-0.015
Single-wall Corrugated Plastic	0.018-0.025
Polyvinyl Chloride (PVC)	0.009-0.011

6.3 Hydraulic Design Methods

The following sections describe design methods for culverts, storm sewers, and a brief introduction to the design of stormwater management systems.

Culverts

Culverts are open-ended structures that are used to convey stormwater runoff from one side of a roadway or railway embankment, or other obstruction, to the other side. There are three general considerations when designing a culvert. These are the culvert's location, size, and shape. The location of a culvert can be subdivided into the alignment, length, and slope. The size of the culvert is generally determined based on the volumetric rate of runoff measured during the hydrologic analysis of the site. The culvert shape is a function of the site's hydraulic requirements and the height of the cover over the culvert. Under certain hydraulic conditions, the tailwater depth (i.e., normal depth of flow in the downstream channel) and the roughness of the culvert may also impact the culvert type selected. Circular culverts are the most predominant structure type because they are both hydraulically- and structurally-efficient.

Early research on culvert hydraulics was performed by the U.S. Bureau of Public Roads and by the Federal Highway Administration and resulted in culvert flow being classified into two general flow regimes: inlet control and outlet control. Regardless of the flow regime, the resulting solution is identified as the culvert headwater. The culvert headwater is the depth of ponding at the inlet end of the culvert, measured from the culvert invert to the water's surface.

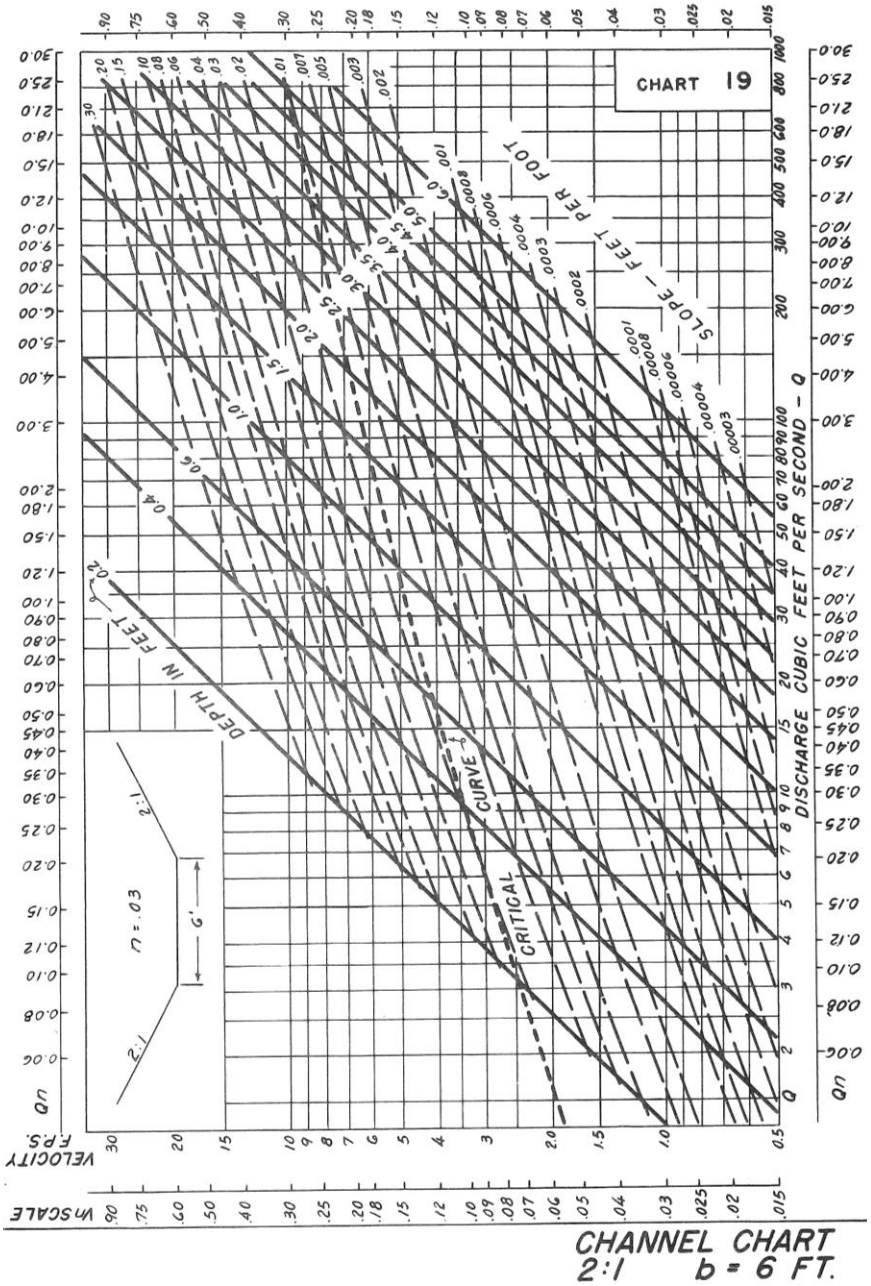


Figure 6.1: Nomograph for the solution of Manning's equation (FHWA HDS No. 3 1961)

Allowable Headwater Criteria

The criteria used to determine the allowable headwater depth come from either physical controls or from arbitrary controls. Physical controls are those which are enacted to protect buildings or other physical structures, upstream productive land, or the roadway or railway. It is also desirable to avoid the unnecessary diversion of runoff from one watershed to another. As such, highpoints in ditch lines which act as a watershed break may also be utilized as a limiting headwater physical control.

Arbitrary headwater controls are those which are enacted to protect the roadway embankment from erosion while allowing the efficient operation of the culvert. The control may be based on either the design or the check year discharge. Arbitrary controls may be a percentage of the culvert diameter or rise, such as 1.2×diameter (1.2D) which would allow a headwater of 120 percent of the culvert diameter or rise. It may also represent an allowable depth of ponding above the culvert barrel, such as diameter plus depth (D+2). Greater depths of ponding are typically permitted for the check year discharge while shallower depths are typically allowed for larger diameter culverts.

While it may be possible to determine whether a culvert will operate under inlet or outlet control, such calculations are tedious and may be inaccurate. Standard practice allows designers to calculate both inlet and outlet control headwater elevations and to use the larger of the two values to be the design headwater.

Inlet Control

Inlet control is when the inlet of a culvert dictates the hydraulic performance of the structure. Under inlet control, the performance of the culvert is based solely on the size, shape, and configuration of the culvert inlet, as well as the amount of discharge reaching the culvert. Roughness of the pipe barrel is not a factor for inlet-controlled scenarios. Culverts with relatively short runs and steep slopes are typically inlet-controlled.

The inlet configuration of the culvert is the type of inlet edge of the barrel of the culvert. Common entrance configurations include thin edge, beveled edge, or grooved edge. The configuration is directly related to the entrance loss coefficient, which is used to determine the head loss at the entrance to a culvert. Grooved or beveled entrances provide for a gradual transition from the channel to the culvert barrel and are more hydraulically efficient than a thin edge entrance. The culvert barrel may be projecting from the roadway embankment or contained within a headwall. Projecting entrances, or those without a headwall, are hydraulically inefficient but are relatively inexpensive. They can be prone to scour damage and can be a traffic hazard. Therefore, the projecting entrance configuration is typically restricted to smaller culvert diameters. Common entrance configurations are shown in **Error! Reference source not found.** and entrance loss oefficients are included in Table 6.3.

(A) Thin Edge Projecting – The culvert barrel projects out of the embankment.

(B) Mitered entrance – The culvert barrel is cut so it is flush with the embankment slope.

(C) Square edge in headwall – The end of the culvert barrel is flush with the headwall.

(D) Groove edge projecting – A concrete pipe culvert section extends beyond the fill or headwall.

Figure 6.2: Common culvert entrance configurations (FHWA HDS No. 5 1985)

Table 6.3: Entrance loss coefficients, adapted from (FHWA HDS No.5 1985)

Type of Entrance	Entrance Loss Coefficient, K_e
Sharp-edge projecting from fill	0.90
Square-edge in headwall	0.50
Mitered to conform to fill slope	0.70
Beveled edge, 33.7° or 45° bevel	0.20
Flared-end section	0.20

Outlet Control

When a culvert is operating under outlet control, all of the inlet control factors apply. In addition, the culvert length, slope, roughness, outlet configuration, and tailwater elevation also affect the discharge capacity of a culvert operating under outlet control. Outlet control typically occurs when a culvert is on a gentle slope, has a rougher barrel, or has a long length.

Determine Headwater Elevation and Exit Velocity

The Federal Highway Administration Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts" (1985), provides detailed information on the design and selection of culverts. The publication also includes a large number of Inlet Control and Outlet Control design charts for the most prevalent culvert shapes, materials, and inlet configurations. It is a free publication that should be consulted prior to beginning the design of a culvert.

With a given design discharge and allowable headwater elevation, the selection of appropriately-sized culverts can be determined using the various charts in the FWHA publication. Figure 6.3 shows a typical Inlet Control nomograph.

Outlet control calculations follow a procedure similar to that for inlet control, but are more complex due to additional design variables which are required. Figure 6.4 presents a typical Outlet Control nomograph.

Culvert exit velocities are generally greater than that of the receiving channel. Outlet velocity can be calculated using Manning's equation for culverts operating under inlet control. For culverts operating under outlet control, the use of the continuity equation is recommended. The use of riprap, stilling basins, or other energy dissipation methods is required if the culvert exit velocity exceeds the velocity resistance of the receiving channel, or when appropriate for stormwater management planning. The design of dissipation devices can be found in the FHWA Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (2006).

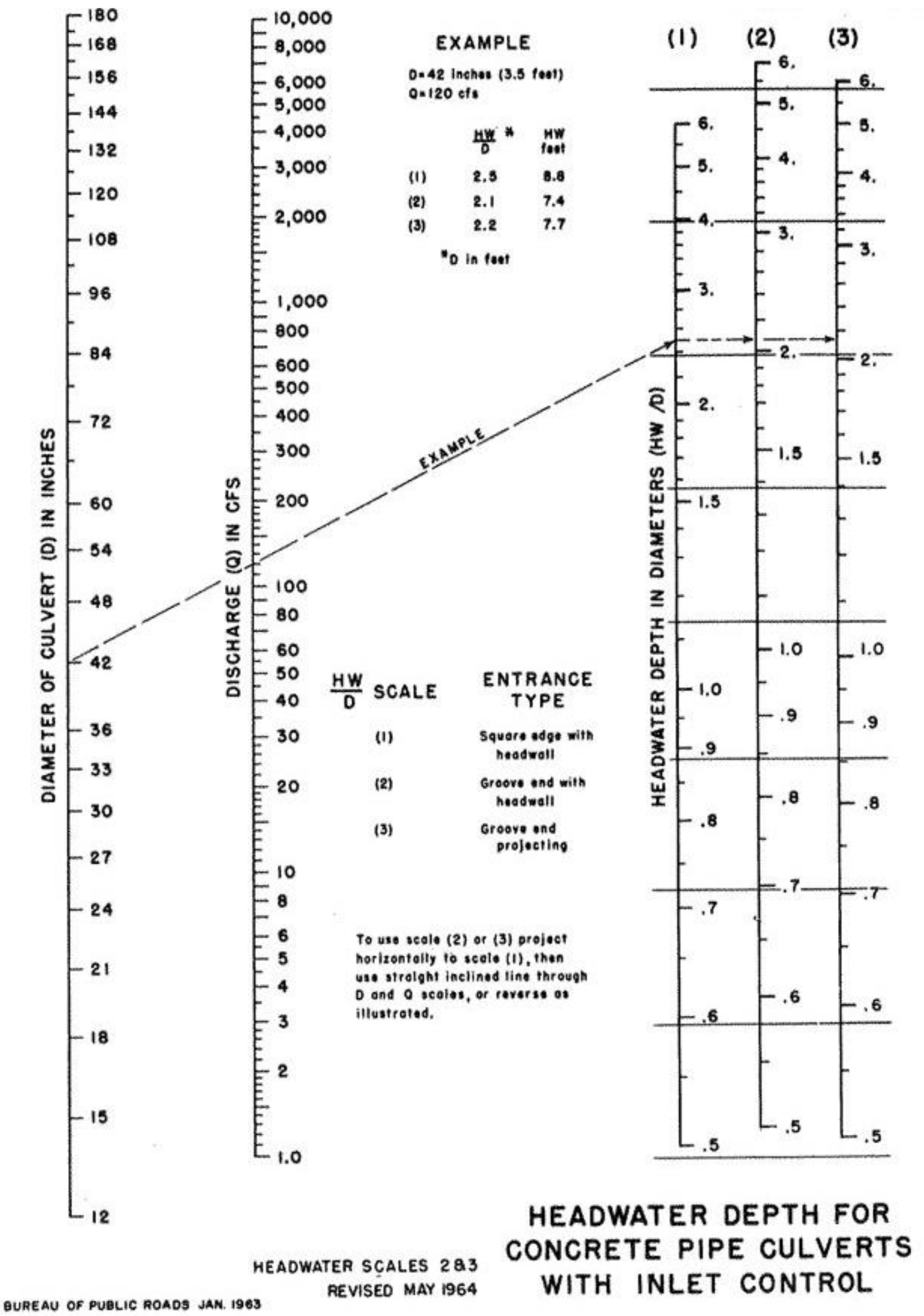


Figure 6.3: Inlet control nomograph (FHWA HDS No. 5 1985)

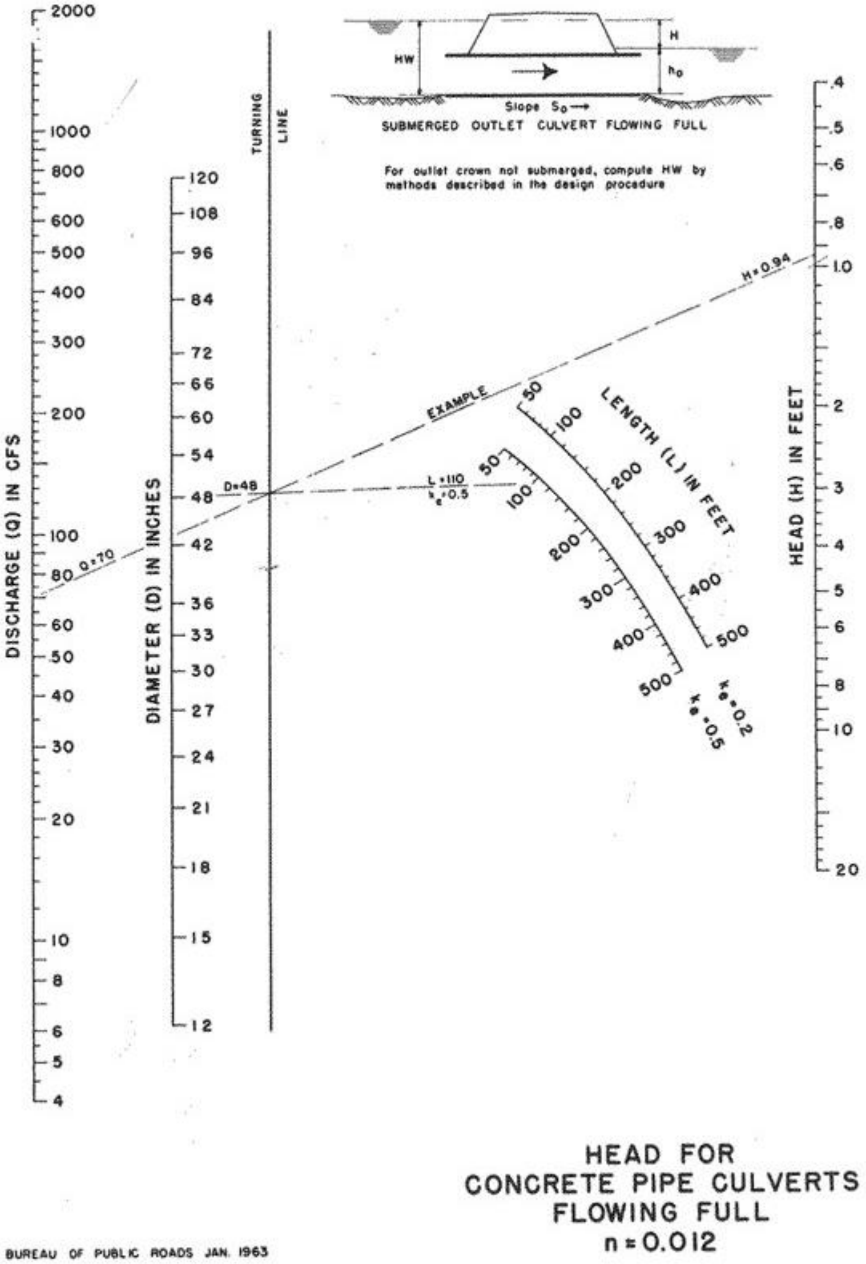


Figure 6.4: Outlet control nomograph (FHWA HDS No. 5 1985)

The FHWA offers the software package HY-8 which is a free PC-based software program used in the design of culverts and culvert energy dissipation. It is based on the FHWA publications HDS No. 5 (1985) and HEC No. 14 (2006).

Minimum Pipe Size

Most State DOTs, counties, and many cities establish minimum pipe sizes for use within their jurisdictions. Minimum sizes have been observed to range from 12 to 36 in. (300 to 900 mm), depending on the importance of the structure, the cost for replacement, and local requirements. Jurisdictions may also require the arbitrary over-sizing of culverts to allow for future maintenance when the cost of replacement is high, such as under deep fills or under high-volume roadways.

Gravity Flow Storm Sewer

In the laws of conservation, the two fundamental principles typically employed for the design of gravity flow storm sewer systems are the continuity equation the energy equation.

Specific Energy

The specific energy, E, is the total head in the conduit or channel and is defined in Eqn. 6.7 as:

$$E = d + \frac{v^2}{2g} \qquad \text{(Eqn. 6.7)}$$

where:

E = specific energy, ft (m);
d = depth of flow, ft (m);
v = velocity, ft/s (m/s); and,
g = gravitational acceleration, 32.2 ft/s^2 (9.8 m/s^2).

By plotting the specific energy versus the depth of flow, a trend similar to that shown in Figure 6.5 can be developed. For each value of specific energy there are two depths, known as conjugate depths. The only exception is at the point of inflection, which is the point of minimum specific energy. The depth of flow at this point is termed the critical depth, d_c. Depths of flow greater than d_c are known to be subcritical, and depths of flow less than d_c are identified to be supercritical.

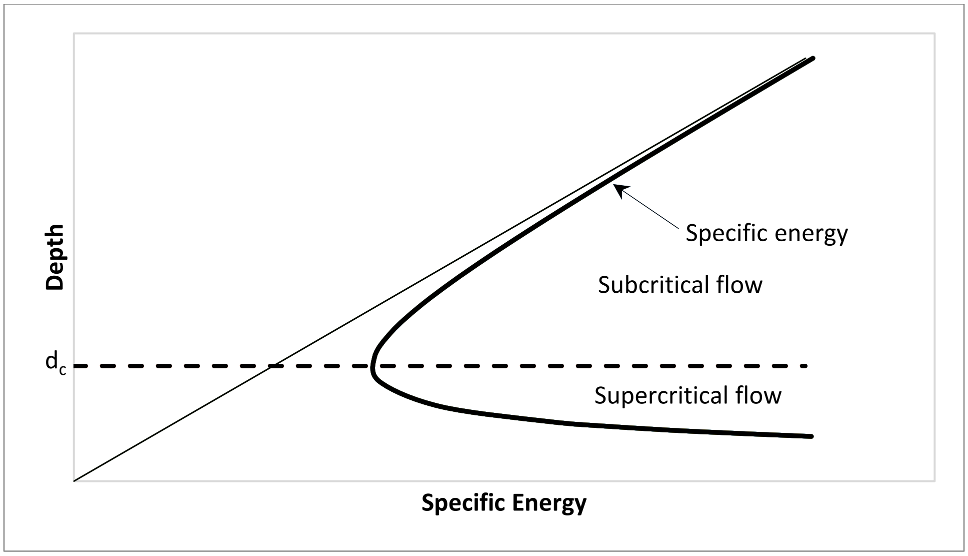


Figure 6.5: Specific energy diagram

Critical depth can also be found by setting the Froude Number, *F*, to unity. The Froude Number is given in Eqn. 6.8 as:

$$F = \frac{v}{\sqrt{gD_h}} \qquad \text{(Eqn. 6.8)}$$

where:

F = Froude Number, dimensionless;
v = velocity, ft/s (m/s);
g = gravitational acceleration, 32.2 ft/sec^2 (9.8 m/s^2); and,
D_h = Hydraulic Depth, ft (m), defined as flow area/channel top width.

Froude Numbers that are less than one are indicative of subcritical flow, while Froude Numbers that are greater than one are indicative of supercritical flow.

Bernoulli Equation

The energy equation is based on the principles of conservation of energy; namely, that energy can be neither created nor destroyed. In terms of flow in a conduit or channel, the total energy at one point along the conduit is equivalent to any other point along the conduit, once all of the energy losses between two points are considered. Bernoulli determined that for a frictionless, incompressible fluid, the conservation of energy results in the equation shown in Eqn. 6.9:

$$z_1 + d_1 + \left(\frac{v_1^2}{2g}\right) = z_2 + d_2 + \left(\frac{v_2^2}{2g}\right) + h_l \qquad \text{(Eqn. 6.9)}$$

where:

z_i = Distance to an arbitrary datum, ft (m);
d_i = depth of flow, ft (m);
v_i = velocity, ft/s (m/s);
g = gravitational acceleration, 32.2 ft/sec^2 (9.8 m/s^2); and,
h_l = head loss.

The Bernoulli Equation is used to determine the depth of flow in a channel or conduit when there is a change in the geometry or slope of the section.

Figure 6.6 is a representation of the change in the head along a channel section. Important parameters include the velocity head, $v^2/2g$, and the specific energy. The line consisting of the points of static head along the length of the channel is the hydraulic grade line. When the velocity head is added to the plot of static head, the result is known as the energy grade line.

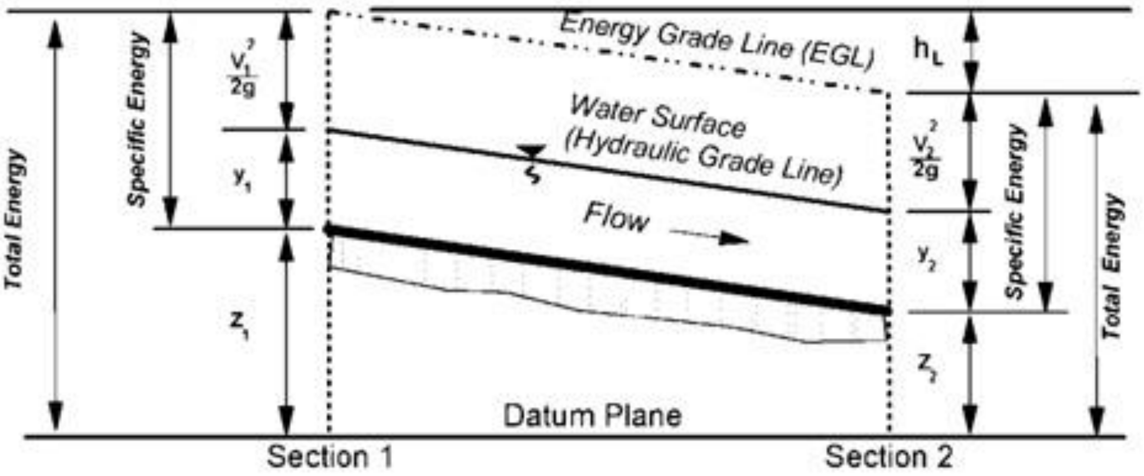


Figure 6.6: Change in head in open channel flow (FHWA HDS No.4 1996)

Energy Losses

From an inspection of the Bernoulli equation, it is evident that an important factor for consideration is the head loss, or energy loss, through the system. Head loss can be generally classified as friction losses and minor losses, such as bend loss, expansion and contraction losses, and manhole and inlet losses.

Friction Losses

The drop in head resulting from friction losses equate to the head losses resulting from the interaction between the water and the conduit or channel wall. This is a function of the area and roughness of the conduit and the flow rate of water through the conduit. It is convenient to calculate the total friction loss, h_f, between two points by solving for the friction slope, S_f, or the slope of the hydraulic grade line, and multiplying this by the length of the conduit. The friction slope is determined by rearranging the Manning's equation to solve for slope. The resulting equation, Eqn. 6.10 is:

$$S_f = \frac{Q^2 n^2}{1.486 A^2 R^{4/3}} \qquad \text{(Eqn. 6.10)}$$

Minor Losses

Minor losses are head losses resulting from bends, expansions, contractions, manholes and other appurtenances. Bend losses are typically quite small and can be calculated by Eqn. 6.11:

$$h_b = 0.0033\Delta\frac{v^2}{2g} \qquad \text{(Eqn. 6.11)}$$

where:

h_b = bend loss;
Δ = angle of curvature of bend, degrees;
v = velocity, ft/s (m/s); and,
g = gravitational acceleration, 32.2 ft./sec^2 (9.8 m/s^2).

Expansions and contractions are locations where the conduit or channel experiences a change in cross-sectional area. Typically, such transitions occur across a manhole or inlet. However, in larger storm systems, the need for such transitions may occasionally occur within the barrel of the pipe. For gradual transitions in open-channel flow, the contraction loss coefficient is approximately one-half of the expansion loss coefficient. Table 6.4 introduces values related to gradual expansion loss coefficients. Expansion or contraction losses can be calculated using Eqn. 6.12:

$$h_m = K_m\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right) \qquad \text{(Eqn. 6.12)}$$

where:

h_m = expansion or contraction loss;
K_m = minor loss coefficient;
v_1 = velocity in upstream conduit, ft/s (m/s);
v_2 = velocity in downstream conduit, ft/s (m/s); and,
g = gravitational acceleration, 32.2 ft/sec^2 (9.8 m/s^2).

Table 6.4: Typical values for K_m for gradual enlargement of pipes in non-pressure flow, modified from Water Resources Engineering, 2nd Ed. with permission

Typical Values for K_m for Gradual Enlargement of Pipes in Non-Pressure Flow.							
D_2/D_1*	Angle of cone						
	10^0	20^0	45^0	60^0	90^0	120^0	180^0
1.5	0.17	0.40	1.06	1.21	1.41	1.07	1.00
3	.017	0.40	0.86	1.02	1.06	1.04	1.00

* D2 /D1 = Ratio of diameter of smaller pipe to larger pipe

Manhole and Inlet Losses

Manhole and inlet losses, commonly referred to as junction losses, can be estimated using Eqn. 6.13:

$$h_j = K_j \frac{v^2}{2g} \qquad \text{(Eqn. 6.13)}$$

where:

h_j = junction loss;
K_j = minor loss coefficient, taken from Table 6.5;
v = velocity, ft/s (m/s); and,
g = gravitational acceleration, 32.2 ft/sec^2 (9.8 m/s^2).

For a more complete discussion and advanced equations for determining minor losses in manholes and inlets, the reader is directed to the FHWA publication HEC No. 22, "Urban Drainage Design Manual."

Table 6.5: Head loss coefficients for inlets and manholes (FHWA HEC-22 2001)

Structure Configuration	K_j
Inlet – straight run	0.50
Inlet – angled through	
90^0	1.50
60^0	1.25
45^0	1.10
22.5^0	0.70
Manhole – straight run	0.15
Manhole – angled through	
90^0	1.0
60^0	0.85
45^0	0.75
22.5^0	0.45

System Velocity

A minimum velocity in a storm conduit is often specified by owner agencies to ensure that the system is self-cleaning. This minimum clean-out velocity is generally specified as 3.0 ft/s (0.9 m/s).

Extremely high velocities should also be considered. In situations when a conduit is operating under supercritical flow, a downstream tailwater (i.e., a less steep conduit run) may lead to a hydraulic jump within the conduit. This may result in a negative pressure head within the conduit. This condition should be mitigated through venting, stepping the conduit incrementally down a steep slope, or through the use of a larger conduit size. Extremely high velocities (greater than 20 ft/s (6 m/s)) result in significantly higher minor losses. The accumulation of the minor losses throughout a storm system can result in significant pressure head in the system. Analysis of the hydraulic grade line is required to understand the implication of these minor losses.

Surcharged Pressure Conditions

The flow in a storm system is generally designed to operate as open channel flow. Many owner agencies specify that the storm system be designed for “just-full” capacity (i.e., approximately 93% to 95% of full flow capacity) for the design year event. For discharges higher than the design year event, it is possible that the storm system will operate under pressure flow. Such rare instances of pressurized flow are typically not an issue; however, a storm system that is subjected to repeat occurrences of pressurized flow must be designed using conduit and joint systems intended for pressure flow.

Stormwater Management

There are two main elements for stormwater management. These are water quantity control and water quality control. Water quantity controls are techniques used to reduce the rate or volume of runoff from a particular watershed or land parcel. Water quality controls are devices intended to reduce or remove particular pollutants from stormwater runoff.

The development of natural lands often results in the conversion of pervious areas to impervious areas, and the conversion of natural drainage paths to enclosed conveyance systems such as storm sewers. This conversion of pervious lands increases both the peak rate of runoff and the volume of runoff. This is because existing pervious areas allow for the infiltration and storage of runoff, whereas impervious areas do not. In addition, impervious areas are often directly connected to storm sewers and thereby eliminate the potential for infiltration. This conversion of land cover results in a decrease in the time of concentration for the watershed and potential increases in the volume discharged. Also, converting an open channel discharge to a conveyance system may increase discharge velocities. **Error! Reference source not found.** shows a typical increase in eak runoff rate and the volume that results from the conversion of pervious area to impervious area.

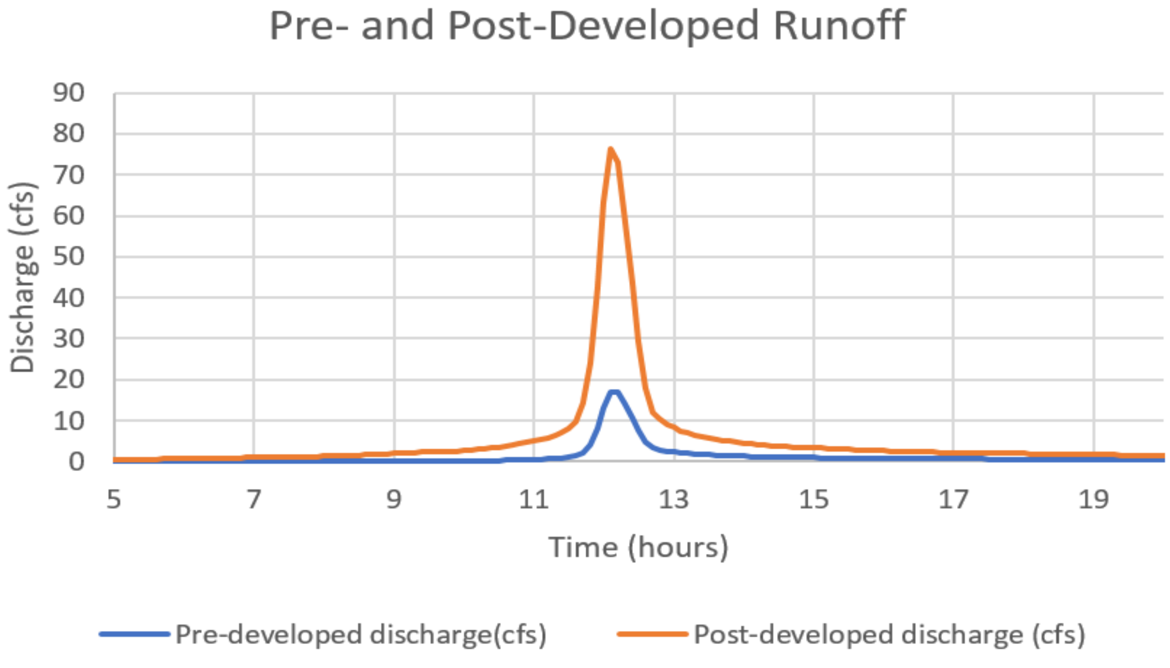


Figure 6.7: Comparison of pre- and post-developed discharges

The increase in impervious areas and the direct connection to conveyance systems often result in an increased pollutant loading in the stormwater runoff. Environmental compliance often requires the use of water quality control devices to reduce pollutant loading in stormwater runoff. Pollutants can include oil and grease, eroded soil or sediment, solid trash, and heavy metals.

Detention and Retention Systems

Detention and retention are often used to mitigate the negative impacts of an increase in the peak rate of runoff and runoff volume. Detention is the temporary storage of excess runoff and the controlled release of the runoff to mitigate negative downstream impacts caused by the increase in runoff rate or volume. Retention is also intended to reduce the impacts of increased runoff, but does so by retaining the water sufficiently long to allow for infiltration and evaporation. Often, the detention and retention are used in conjunction where large storm events are detained, but lesser events are retained. A wet storage basin, which maintains a permanent water pool, is one example of a combined facility. Figure 6.8 shows a comparison of a post-developed hydrograph and a detention facility outflow hydrograph that indicates how detention delays the release of stormwater and reduces the peak discharge.

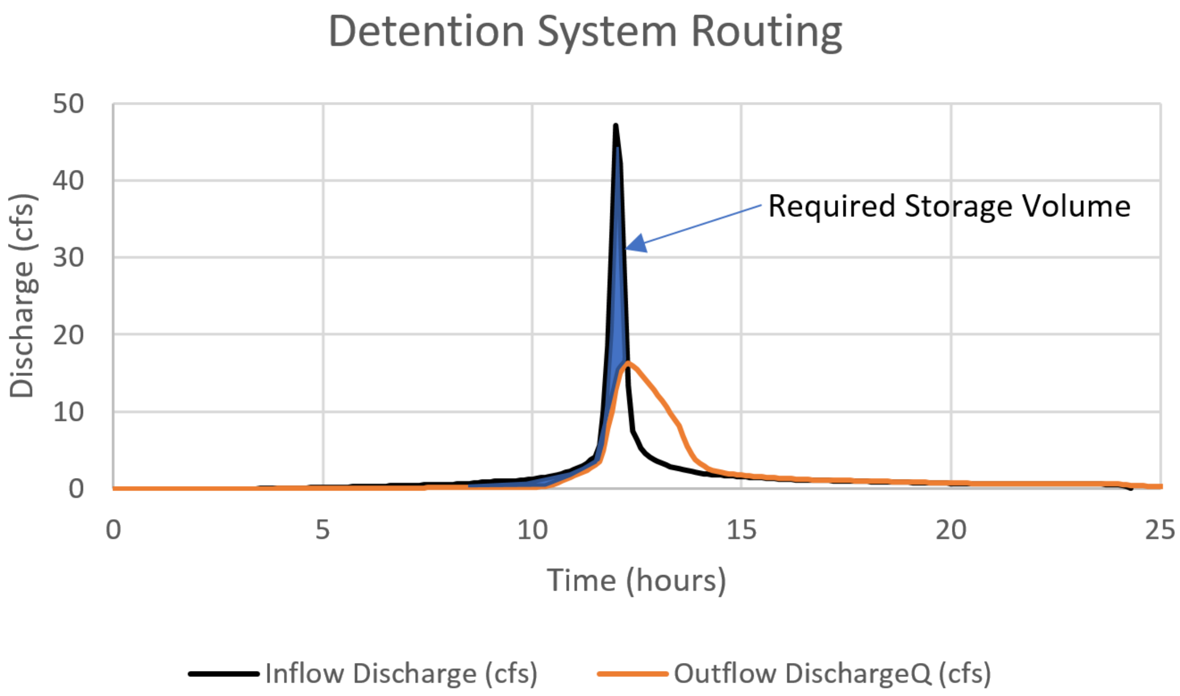


Figure 6.8: Hydrographs of post-developed discharge and detention facility discharge showing reduction in peak discharge

Underground Retention/Detention Systems

Where land costs are particularly high, or where available land is scarce, the use of underground detention/retention systems is common. Underground detention/retention systems can be comprised of a single run of large diameter conduit or a series of conduits joined via a header manifold. The constituent pipes can be either perforated or non-perforated. Perforated pipe may allow for the recharge of the groundwater system and a reduction in runoff, or allows for the infiltration of runoff that has been absorbed into the ground. Some corrugated plastic pipe manufacturers also produce chambers with open-bottoms, which allow for the infiltration of runoff into the surrounding soil. An image of an underground detention/retention system is shown in **Error! Reference source not found.**.

Figure 6.9: Underground enclosed detention chamber (courtesy of Prinsco)

Underground detention/retention systems, designed to infiltrate into the ground or to make use of the void space of the stone backfill around the systems for additional storage, often require sediment removal capabilities upstream. By removing trash, sediment, and/or hydrocarbons from the storm water discharge, the void space in the backfill is preserved and periodic maintenance can occur in a predetermined location upstream of the detention/retention system. Large diameter corrugated plastic pipe can be fabricated with baffles and filter media to provide water quality benefits by removing petro-chemicals and suspended solids.

Detention/retention systems are unique designs based on the storage needs, parcel dimensions, environmental regulations (e.g., maintaining a certain distance above groundwater), and other restrictions. As such, they can be complex and time consuming to design. Corrugated plastic pipe producers have greatly simplified the process by providing components, such as headers and fittings, made specifically for detention/retention systems.

Because components are specially made for the detention/retention system they are made to fit together. Field modifications are seldom required. The lightweight feature of the pipe and components assist in speedy handling and placement. Quality joints that are easy-to-assemble help to facilitate installation.

Runoff may be stored in a detention/retention system for extended periods. If the runoff contains harsh chemicals or road salts, the pipe material must be suitable to resist chemical degradation. It is widely accepted that corrugated plastic pipe is highly resistant to chemical corrosion and to abrasion.

Underground detention/retention systems are typically installed to as shallow a depth as possible to limit the excavation costs and to maintain clearance between the system and the groundwater. Properly installed corrugated plastic pipe can withstand AASHTO HS-25 vehicular loads with at least 1 ft (0.3 m) of cover for pipe 48-inch (1200 mm) and smaller, or 2 ft (0.6 m) of cover for larger pipe diameters. This allows for driveways, access roads, parking lots, and similar structures to be built above the system.

Underground detention/retention systems utilizing perforated pipe often make use of the granular backfill surrounding the system for additional runoff storage volume. AASHTO #57 stone has a void ratio of approximately 40% and a resulting porosity of 28%. This means that 28% of the total backfill volume can be utilized to store additional runoff. However, appropriate reductions to the available void space must be made to account for the presence of groundwater.

Design

The design of detention/retention facilities, commonly referred to as routing, is based on the storage-indicator method, or Modified Puls method, which is an implementation of the conservation of mass. The fundamental equation given in Eqn. 6.14 is:

$$\frac{\Delta S}{\Delta t} = \frac{I_1+I_2}{2} - \frac{O_1+O_2}{2} \qquad \text{(Eqn. 6.14)}$$

where:

ΔS = Change in storage during time step Δt, ft^3 (m^3);
Δt = time step, min;
I = inflow, ft^3 (m^3); and,
O = outflow, ft^3 (m^3).

It is convenient to rearrange the above equation such that all known values are combined on the left-hand side of the equation. The result is shown in Eqn. 6.15:

$$\frac{I_1+I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad \text{(Eqn. 6.15)}$$

This equation is then solved for each time step in the routing procedure. An inflow hydrograph can be developed using the NRCS TR-55 methodology described in Chapter 5 of this manual.

A stage storage curve, which provides the relationship between the depth of water in the detention/retention facility and the volume of water stored. A typical stage-storage relationship is shown in Figure 6.10.

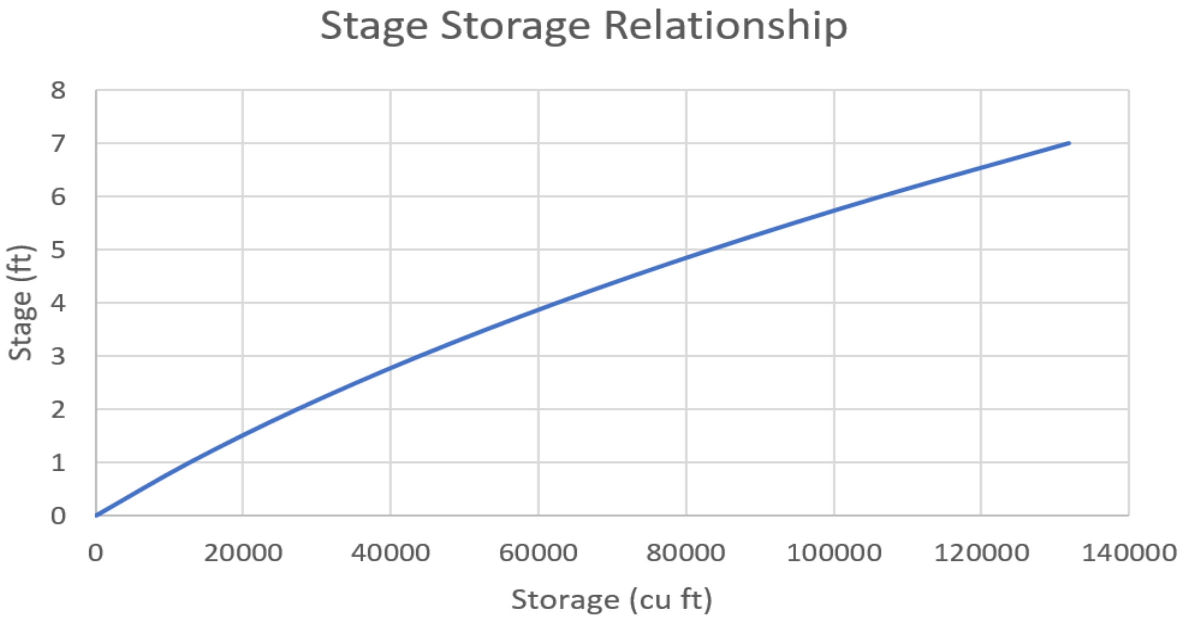


Figure 6.10: Typical stage storage relationship

A stage discharge curve that provides the relationship between the depth of water in the detention/retention facility and the discharge of water from the outflow structure. Outflow structures can include orifices, weirs, evaporation, or infiltration. A typical stage-discharge relationship is presented in Figure 6.11.

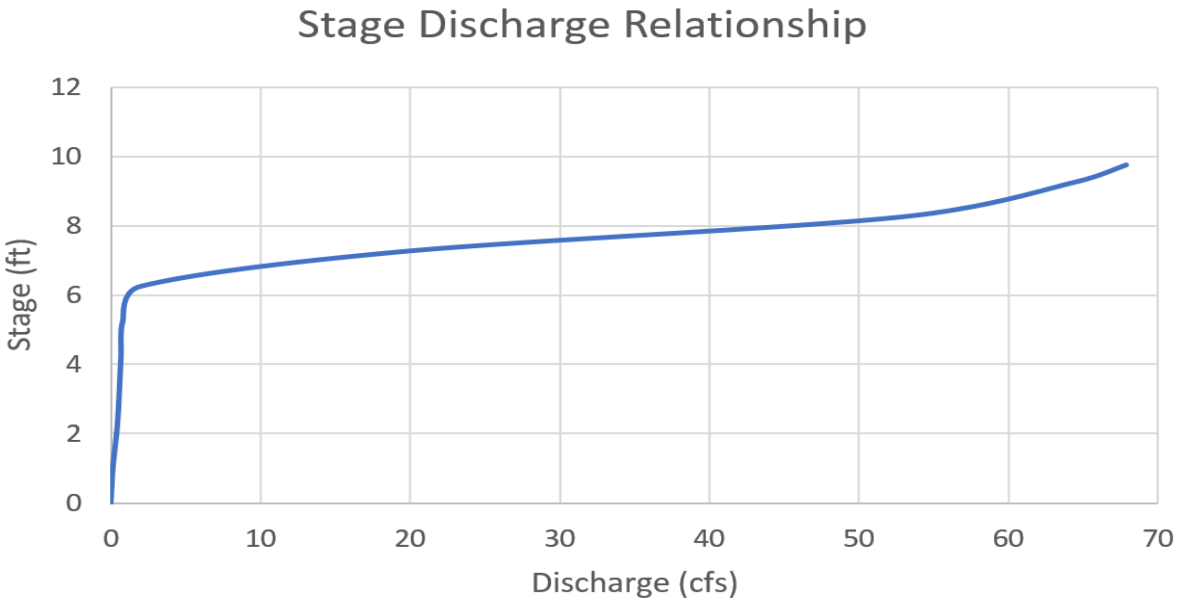


Figure 6.11: Typical stage discharge relationship

There are numerous commercial software programs available that significantly simplify the routing procedure and can be used to develop all of the required inputs.

Inspection and Maintenance

A maintenance and inspection plan is an important part of a stormwater management facility. The majority of stormwater control systems require only periodic removal of accumulated sediment or routine upkeep, such as mowing. The periodic inspection of the system will ensure that the system is performing as intended. A yearly inspection cycle is recommended unless otherwise dictated, based on the application.

Inspections should focus on a number of aspects. For example, the inlet and outlet structures should be inspected for signs of clogging or deterioration. The water storage zone, including backfill if used as a water storage zone, should be inspected for sediment and debris buildup. This is particularly important near the inlet and outlet structures. The source of any standing water should be identified. Standing water may lead to unwanted insects or weeds. Finally, any signs of structural distress in conduits, outlet structures, or facility berms and embankments should be identified and documented.

6.4 Example Problems

Culvert design

Given:

Consider the culvert shown in Figure 6.12. Use the following information provided to complete the most economical culvert design:

Culvert is to be smooth-lined HDPE with standard manufactured ends (i.e., no bell)

Design year event (Q_{25}) = 150 cfs (4.2 cms)
Check year event (Q_{100}) = 240 cfs (6.8 cms)
Design year tailwater depth of flow = 3.7 ft (1.1 m)
Check year tailwater depth of flow = 4.1 ft (1.2 m)

The headwater must be limited to the following allowable headwater criteria:

1. 1.2D for the design year event;
2. 2D for the check year event; and,
3. 1 ft (0.3 m) of freeboard from the edge of roadway for the design year event.

All design data and design trials will be recorded on the Culvert Design Form shown in Figure 6.16 obtained from the Ohio Department of Transportation, Location and Design Manual (2019). Similar forms are widely available from State DOT and local political jurisdictions.

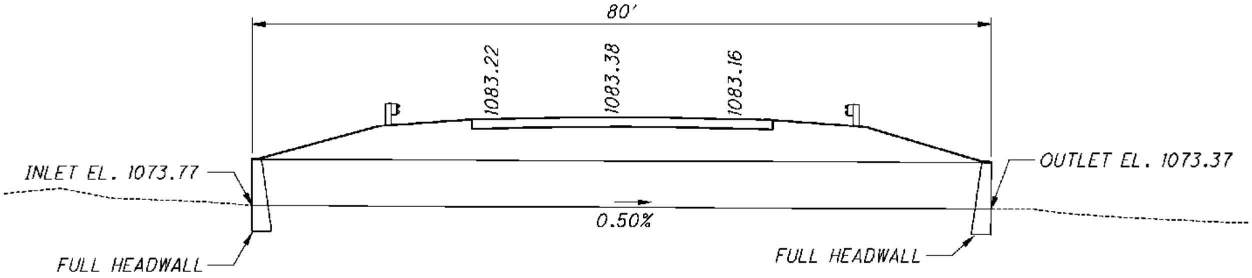


Figure 6.12: Culvert profile for design example

Inlet control:

1. Select a trial culvert size of 54 in. (1370 mm). Enter HDS-5 Chart 1B (Figure 6.13) for smooth culverts operating under inlet control;
2. Construct a line beginning at 54 on the Diameter of Culvert Scale and extend the line through the Design Discharge scale at 150 cfs. Finally, terminate the line at the first scale for Headwater Depth in Diameters. The first scale is utilized for the given entrance configuration; and,
3. Read the HW/D value of 1.48 value from the scale.

The 1.48 value exceeds the allowable criterion of 1.2D. Therefore, select the next larger culvert size:

1. Select a second trial size of 60 in. (1500 mm);
2. Construct a line beginning at 60 on the Diameter of Culvert Scale and extend the line through the Design Discharge scale at 150 cfs. Finally, terminate the line at the first scale for Headwater Depth in Diameters. The first scale is utilized for the given entrance configuration; and,
3. Read the HW/D value of 1.16 from the scale.

The 1.16 value meets the design-year allowable headwater criterion of 1.2D.

Next, it is necessary to ensure compliance with the 1 ft. (0.3 m) of freeboard from the edge of roadway:

1. Determine allowable headwater elevation as 1083.22 ft. – 1 ft. = 1082.22 ft. (330 m); and,
2. Determine headwater elevation by multiplying HW/D ratio by the 60 inch (5 feet) diameter.
   (5 ft)(1.16) = 5.80 ft. and then add to the culvert invert of 1073.77 ft. = 1079.57 ft. < 1083.22 ft. (329 m < 330 m).

The inlet control design headwater elevation meets the freeboard criterion, as well as the arbitrary 1.2D criterion.

Finally, it is necessary to calculate the inlet control headwater for the check year event:

1. Select a second trial size of 60 in. (1500 mm);
2. Construct a line beginning at 60 on the Diameter of Culvert Scale and extend the line through the design discharge of 240 cfs. Finally, terminate the line at the first scale for Headwater Depth in Diameters. The first scale is utilized for the given entrance configuration;
3. Read the HW/D value of 1.80 from the scale; and,
4. Determine headwater elevation by multiplying HW/D ratio by the 60-inch (5 feet) diameter.
   (5 ft)(1.80) = 9.0 ft. (2.7 m).

Therefore, select a 60-in. (1500-mm) smooth interior corrugated plastic pipe as the appropriate culvert.

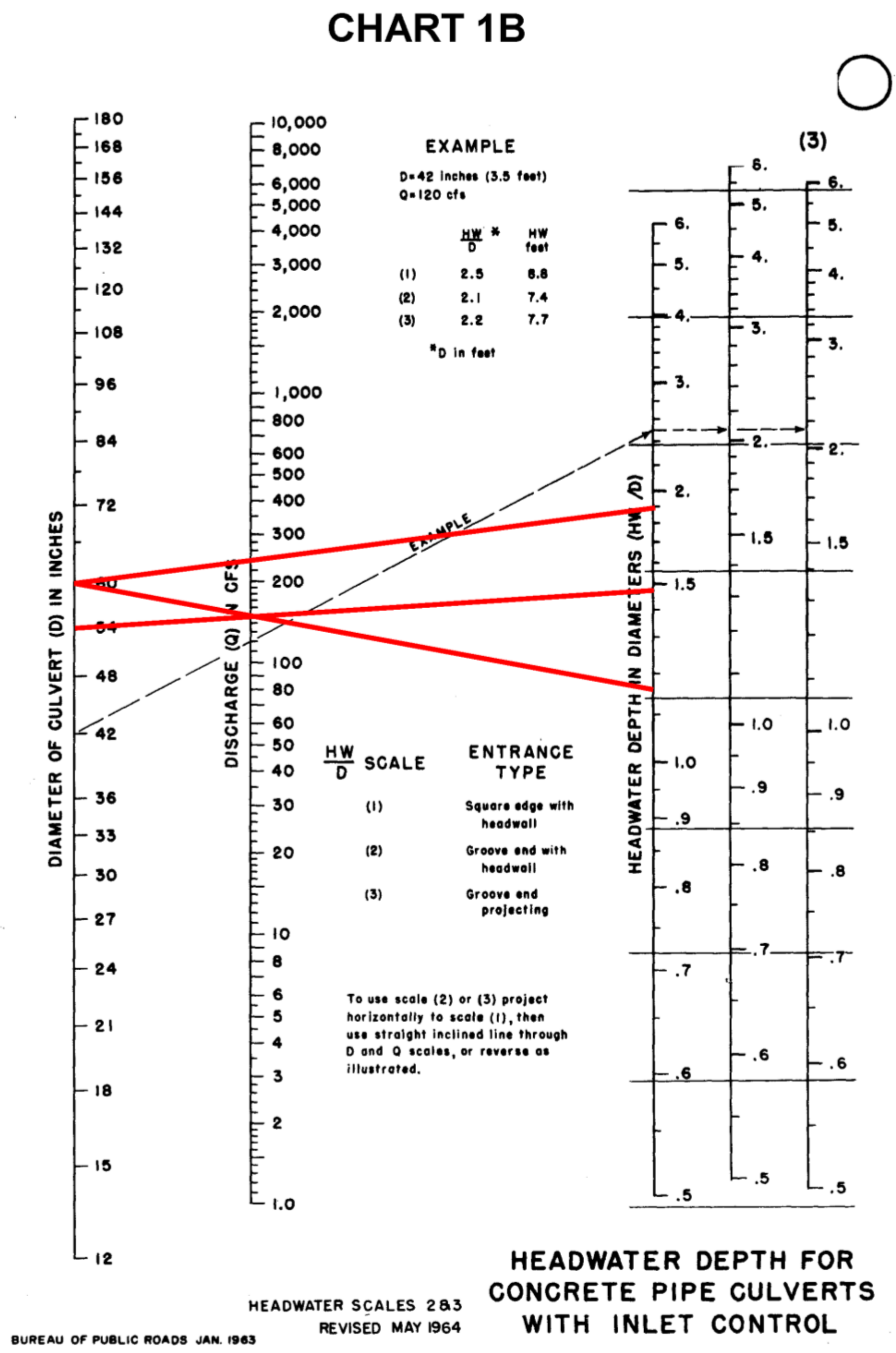


Figure 6.13: Inlet control headwater depth nomograph, several alternatives shown

Outlet control

Using the same design, determine the outlet control headwater for both the design- and check-year events. This is done by determining the water surface elevation at the outlet of the culvert and adding head loss and subtracting the datum change. Eqn. 6.15 is used to calculate the outlet control headwater:

$$HWO = H + h_o - LS_o \qquad \text{(Eqn. 6.15)}$$

where:

HWO = Outlet control headwater, fet (m);
H = Head loss, ft. (m);
h_o = Greater of the tailwater depth or $(d_c+D)/2$, ft. (m); where d_c is the critical depth of flow;
L = Culvert length, ft. (m); and,
S_o = Culvert slope, ft/ft (m/m).

1. With the previously determined 60-in. (1500 mm) culvert. Enter HDS-5 Chart 5B (Figure 6.14) for head for smooth barrel pipes flowing full;
2. Construct a line beginning at 60 on the Diameter of Culvert Scale and extend the line to 80 on the Length in Feet scale for k_e= 0.5. Make note of the intersection with the turning line.
3. Next, extend a line beginning at 150 on the Discharge in CFS scale extending through the intersecting point on the turning line and continuing to the Head in Feet scale.
4. Read the H value of 1.55 value from the scale;
5. Determine h_o as the greater of the tailwater depth or $(d_c+D)/2$. For the design year, the tailwater depth, TW is 3.7 ft. (1.1 m). Critical depth is calculated as 3.5 ft. (1.0 m). This can either be calculated or determined using charts provided in HDS-5 for standard culvert shapes.
6. $(d_c+D)/2$ is then calculated as $(3.5+5)/2=4.25 > 3.7$ ft. and h_o=4.25 ft. (1.3 m); and,
7. Calculate HWO as $HWO=H+h_o-LS_o = 1.55 + 4.25$ ft – $80(0.005) = 5.4$ ft. (1.6 m).

CHART 5B

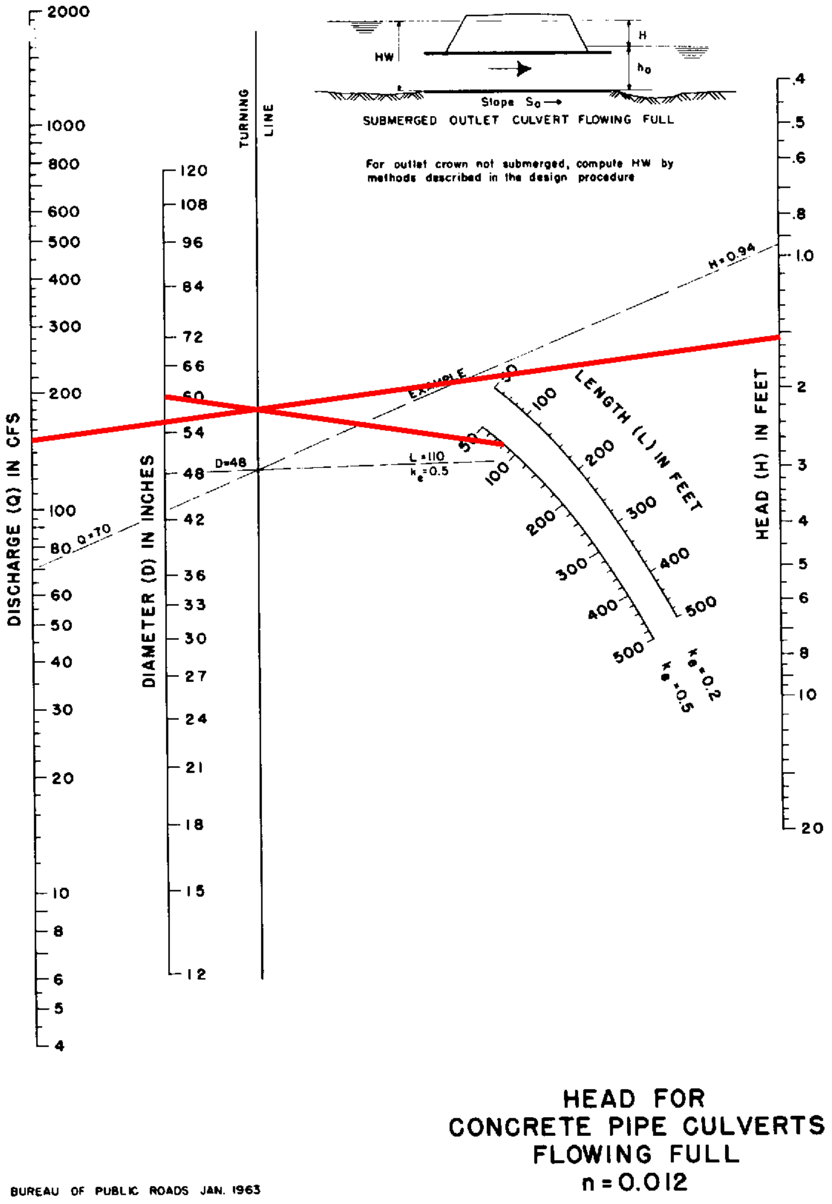


Figure 6.14: Outlet control head nomograph, design year discharge shown

The final step in the design process is to compare the calculated outlet control headwaters with the calculated inlet control headwaters. The larger of the two are conservatively put forth as the governing headwater. For this example, the inlet control headwater controls the design.

Next, calculate the check-year outlet control headwater:

1. With the previously determined 60-in. (1500 mm) culvert. Enter HDS-5 Chart 5B (Figure 6.15) for head for smooth barrel pipes flowing full;
2. Construct a line beginning at 60 on the Diameter of Culvert Scale and extend the line to 80 on the Length in Feet scale for k_e= 0.5. Make note of the intersection with the turning line. Next, extend a line beginning at 240 on the Discharge in CFS scale extending through the intersecting point on the turning line and continuing to the Head in Feet scale;
3. Read the H value of 2.6 value from the scale;
4. Determine h_o as the greater of the tailwater depth or $(d_c+D)/2$. For the design year, the tailwater depth, TW is 4.1 feet. Critical depth is calculated as 4.35 feet. This can either be calculated or determined using charts provided in HDS-5 for standard culvert shapes. $(d_c+D)/2$ is then calculated as $(4.35+5)/2=4.68 > 4.1$ feet. $h_o=4.7$ ft. (1.4 m); and,
5. Calculate HWO as $HWO=H+h_o-LS_o = 2.6 + 4.68 - 80(0.005) = 6.9$ ft. (2.1 m).

CHART 5B

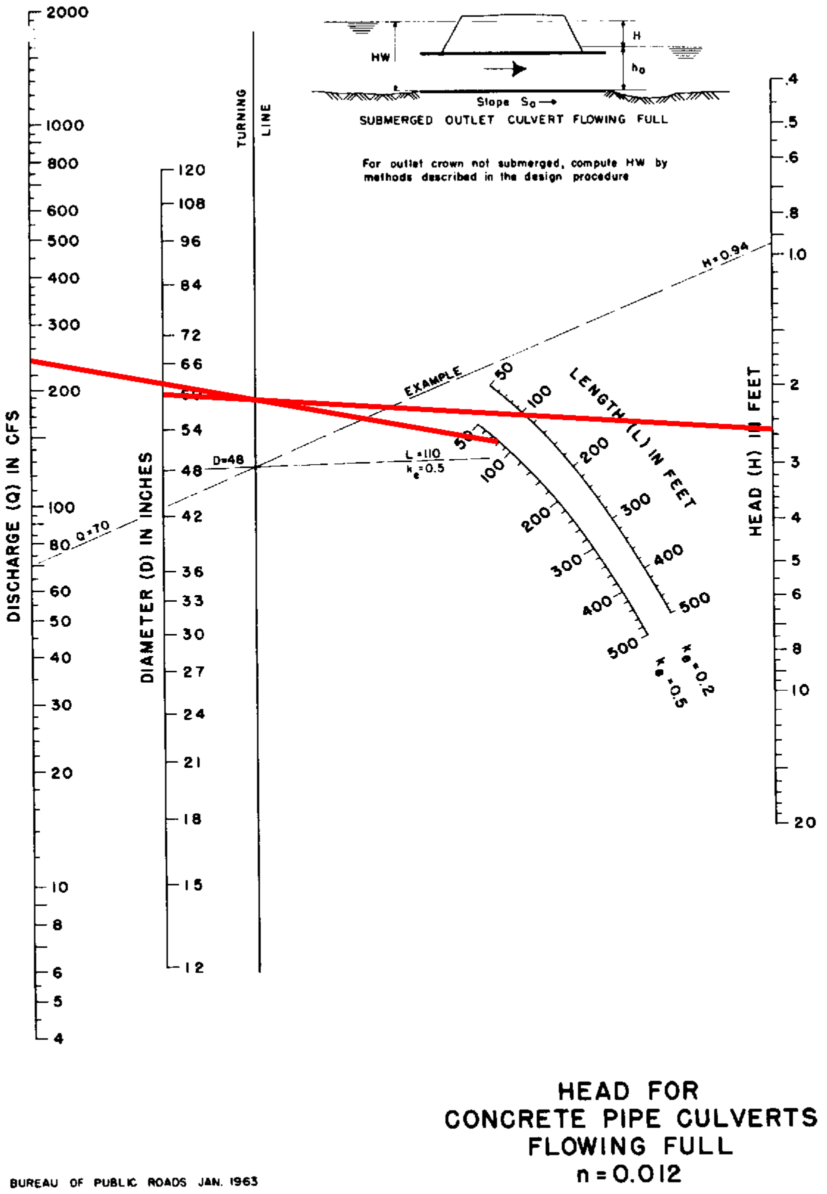
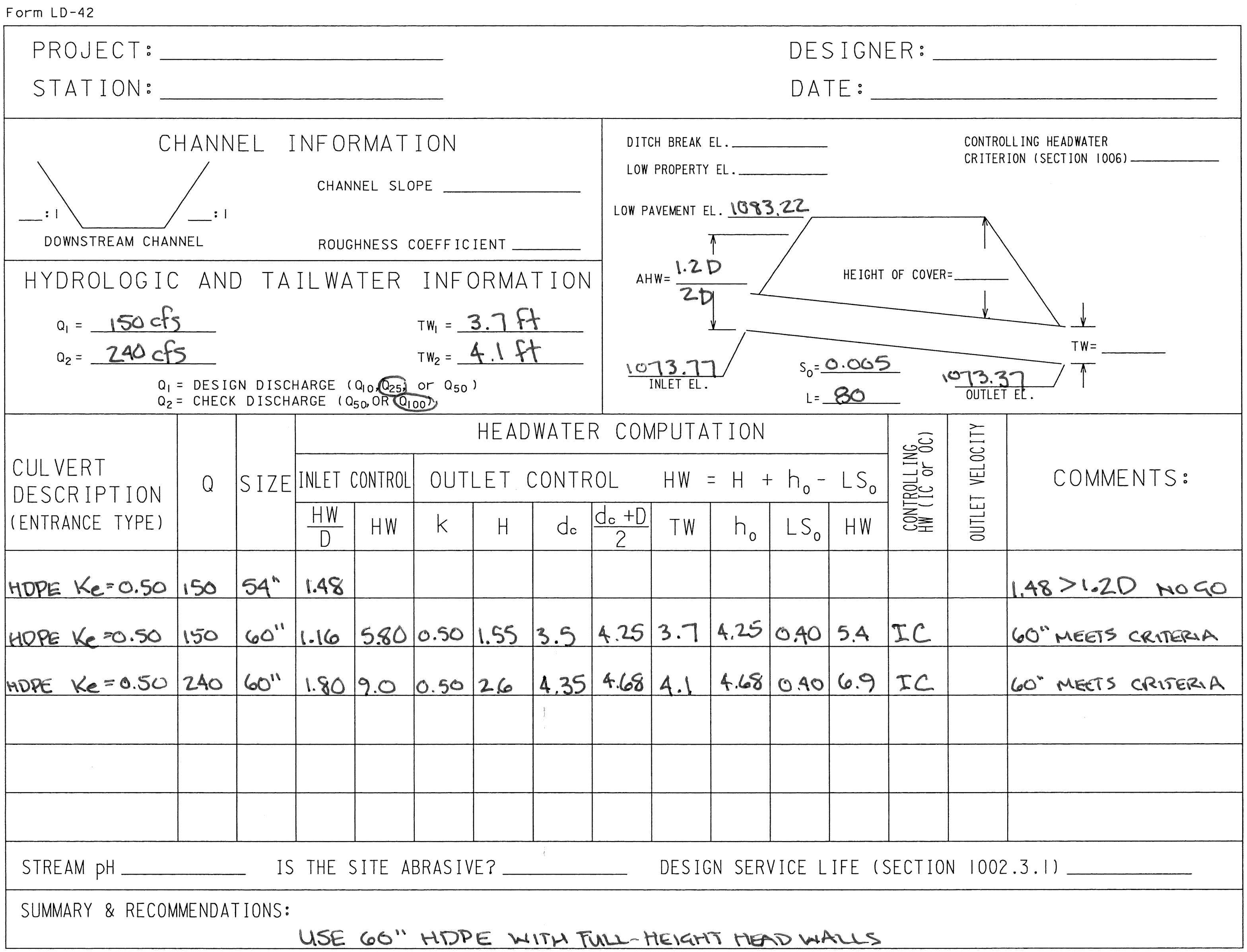


Figure 6.15: Outlet control head nomograph, check year discharge shown



Form LD-42

PROJECT: ____________ DESIGNER: ____________

STATION: ____________ DATE: ____________

CHANNEL INFORMATION

__ : 1 __ : 1

DOWNSTREAM CHANNEL

CHANNEL SLOPE ________

ROUGHNESS COEFFICIENT ________

HYDROLOGIC AND TAILWATER INFORMATION

Q_1 = 150 cfs

Q_2 = 240 cfs

TW_1 = 3.7 ft

TW_2 = 4.1 ft

Q_1 = DESIGN DISCHARGE (Q_{10}, [✓ Q_{25}], or Q_{50})

Q_2 = CHECK DISCHARGE (Q_{50}, OR [✓ Q_{100}])

DITCH BREAK EL. ________

LOW PROPERTY EL. ________

LOW PAVEMENT EL. 1083.22

CONTROLLING HEADWATER CRITERION (SECTION 1006) ________

AHW= 1.2D / 2D

HEIGHT OF COVER= ________

TW= ________

1073.77 INLET EL.

S_0= 0.065

L= 80

1073.37 OUTLET EL.

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW (IC or OC)	OUTLET VELOCITY	COMMENTS:
			INLET CONTROL		OUTLET CONTROL HW = H + h_o - LS_o										
			$\frac{HW}{D}$	HW	k	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW			
HDPE Ke=0.50	150	54"	1.48												1.48 > 1.2D NO GO
HDPE Ke=0.50	150	60"	1.16	5.80	0.50	1.55	3.5	4.25	3.7	4.25	0.40	5.4	IC		60" MEETS CRITERIA
HDPE Ke=0.50	240	60"	1.80	9.0	0.50	2.6	4.35	4.68	4.1	4.68	0.40	6.9	IC		60" MEETS CRITERIA

STREAM pH ________ IS THE SITE ABRASIVE? ________ DESIGN SERVICE LIFE (SECTION 1002.3.1) ________

SUMMARY & RECOMMENDATIONS:

USE 60" HDPE WITH FULL-HEIGHT HEAD WALLS

Figure 6.16: Culvert design computation sheet

Storm Sewer Design

Given:

Consider the roadway and the associated storm sewer plan shown in Figure 6.17, the drainage area summary from Table 6.6, and the intensity-duration information from Table 6.7. The storm sewer is a smooth interior corrugated plastic pipe with a Manning's n value of 0.012. The minimum pipe size is 12 in. (300 mm).

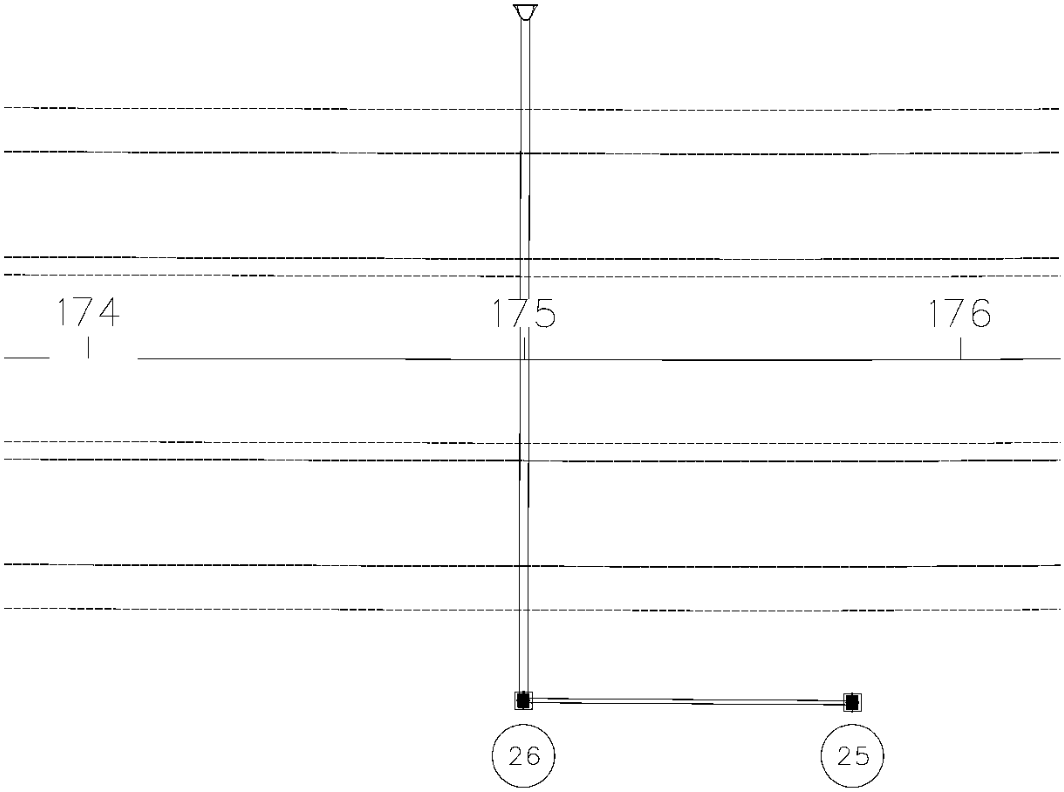


Figure 6.17: Roadway and storm sewer plan for design example

Table 6.6: Drainage area summary for design example

Inlet Number	Drainage Area (acres)	Runoff Coefficient, "C"	Time of Concentration, t_o (min)	Ground Elevation (feet)
25	0.50	0.85	5	2116.16
26	3.50	0.70	10	2116.22

Table 6.7: Intensity-duration information for design example

Time (min)	5	10	15	20	30
Intensity (in./hr.)	7.6	7.3	6.6	5.8	4.8

For Inlet 25, determine the design discharge using the rational equation. From Table 6.6 and Table 6.7, find the drainage area, runoff coefficient, and rainfall intensity. The design discharge is calculated as:

$$Q = 0.5(7.6\ in./hr)(0.50\ acres)$$

$Q = 1.9\ cfs$ (0.05 cms)

Arbitrarily select the minimum allowed pipe size of 12 in. (300 mm) and a pipe slope of 1% for the run from 25 to 26. The selection of the pipe slope is largely based on the elevation of the ground surface above the pipe. The 1% slope will provide good capacity. The pipe slope can be adjusted as the design proceeds.

Using Manning's equation, determine the just-full capacity of a 12-in. (300-mm) pipe on a 1% slope.

$$Q = (0.77)\frac{1.486}{0.012}(0.29^{2/3})(0.01^{1/2})$$

$Q = 4.17\ cfs$ (0.19 cms)

This is greater than the run discharge of 1.9 cfs (0.05 cms), so the selected alternative is acceptable.

For Inlet 26, a common mistake is to simply add discharges from subsequent inlets to determine the total system discharge. One of the fundamental assumptions of the rational method is that the entire watershed is contributing at the time of concentration. For this assumption to hold true, the entire system drainage area must be contributing at the system time of concentration. Therefore, the calculation for discharge at any inlet within a storm network is shown in Eqn. 6.16:

$$Q = i_{system} \sum CA \qquad \text{(Eqn. 6.16)}$$

The system time of concentration is determined by summing the time of concentration of each segment in the system. The system time of concentration calculation proceeds as follows.
Inlet 25 has a beginning time of concentration of 5 minutes. Then, add the time of flow in the conduit. 75 feet @ 4.9 ft/s = 0.26 minutes.

Therefore the system time of concentration is 5.26 minutes. However, Inlet 26 must now be considered. Inlet 26 has a time of concentration of 10 minutes. This is greater than the calculated system time of 5.26 minutes. Thus, 10 minutes become the system time of concentration.

1. From Table 6.7, the rainfall intensity is obtained. The summation of the runoff coefficient and area is (0.5)(0.5)+(3.5)(0.70) = 2.70. The design discharge is calculated as:

   $Q = 7.3(2.7)$
   $Q = 19.7\ cfs$ (0.56 cms);

2. Arbitrarily select a pipe size of 18 in. (450 mm). A pipe slope of 2.5% is selected based on an estimate of the system depth at Inlet 26 and the existing ground elevation. The pipe slope can be adjusted as the design proceeds.

3. Using Manning's equation, determine the just-full capacity of an 18-in. (450-mm) pipe on a 2.5% slope.

$$Q = (1.73)\frac{1.486}{0.012}(0.43^{2/3})(0.025^{1/2})$$

$$Q = 19.3\ cfs\ (0.55\ cms)$$

This is less than the run discharge of 19.7 cfs (0.56 cms), so the selected alternative is rejected and the next larger conduit size is evaluated.

4. Using Manning's equation, determine the just full capacity of a 24 in. (600 mm) pipe on a 2.5% slope.

$$Q = (3.08)\frac{1.486}{0.012}(0.57^{2/3})(0.025^{1/2})$$

$$Q = 41.6\ cfs\ (1.17\ cms)$$

This is greater than the run discharge of 19.7 cfs (0.56 cms), so the selected alternative is acceptable.

5. A system profile is then created, based on the calculated pipe sizes. The profile is largely a function of providing sufficient cover over the storm sewer runs. This is because it is generally most cost-effective to minimize the sewer depth. A final profile of the system is provided in Figure 6.18 and the final slope of the outfall conduit is 2.53%. The final calculations should be revised to match this slope. Minor revisions such as those in this example are a normal part of designing a storm sewer system. The calculations are typically summarized on a tabulation form such as that shown in Figure 6.19.

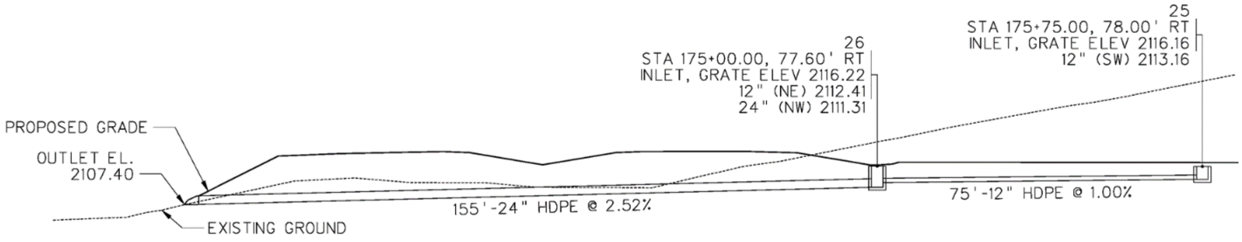


Figure 6.18: Storm sewer profile for design example

STORM SEWER COMPUTATION SHEET																			
STRUCTURE DATA		DRAINAGE AREA DATA							RAINFALL INTENSITY FOR DESIGN STORM (IN INCHES / HOUR)	STORM DISCHARGE (IN CFS)	Pipe Data								
Inlet Structure Number	Outlet Structure Number	DRAINAGE AREA (IN ACRES)		RUNOFF COEFFICIENT	CA		TIME OF CONCENTRATION (IN MINUTES)				SIZE OF PIPE (IN INCHES)	LENGTH OF PIPE (IN FEET)	INVERT ELEVATIONS		SLOPE OF PIPE (IN FT / FT)	JUST FULL CAPACITY (IN CFS)	MEAN VELOCITY (IN FT / SEC)	GRATE ELEVATION
		INLET	SYSTEM	C	INLET	SYSTEM	INLET	SYSTEM					INLET	OUTLET				
Structure 26 to 8																			
25	26	0.500	0.500	0.50	0.250	0.250	5.00	5.00	7.60	1.90	12	75.0	2113.16	2112.41	0.0100	4.17	4.9	2116.16
26	OUT	3.500	4.000	0.70	2.450	2.700	10.00	10.00	7.30	19.7	24	155.0	2111.31	2107.39	0.0253	41.6	12.4	2116.22

Figure 6.19: Storm sewer computation sheet

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