CHAPTER 21

STORM DRAINAGE DESIGN

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INTRODUCTION

Drainage systems are divided into two categories: minor and major. The minor system, which consists of swales, small ditches, gutters, small pipes, and the other various types of inlets and catch basins, collects and conveys runoff to a discharge area or impoundment. Components in the minor system are sized to carry runoff generated by the more frequent, short-duration storm events. The major drainage system includes natural streams, channels, ponds, lakes, retention and detention facilities, large pipes, and culverts. Design criteria for the major system are based on significant amounts of rainfall produced by the less frequent, long-duration storms and are further governed by the hydraulic concepts related to bridges and large conveyance structures.

Storm drain design requires two basic types of analyses: the hydrologic aspect of estimating runoff and the hydraulic aspect of sizing the components. Although there are numerous techniques to estimate runoff, the one selected for design of a particular component depends on the following factors:

- What is being designed
- Type of input data available
- Type of output data required
- Cost effectiveness of the technique
- Required accuracy
- Size of watershed
- Accepted design standards of the approving agency(ies)

Generally, the design of minor-system components requires only the determination of peak runoff discharges, whereas major-system components require not only the peak discharges but the time variation of runoff for an effective design. Analysis of major systems may include detailed hydrology and hydraulic modeling using one- and two-dimensional modeling software. This chapter includes a brief discussion of the available programs and methodology used in analyzing major systems; however, the primary focus is on computing runoff for small drainage areas for the purpose of sizing minor-system components, such as culvert design and analysis and surface inlets and pipe systems. Analysis of minor systems includes hydraulic head loss through the system and hydraulic grade line (HGL) computations. (See Figure 21.1.)

PAVEMENT DRAINAGE

Stormwater runoff on roads is a safety hazard in numerous ways. Hydroplaning, reduced visibility, and icy conditions are dangerous to vehicle occupants and pedestrians. In an urban setting, these hazardous conditions are substantially magnified due to the increased traffic and pedestrian density. It is imperative to effectively remove the stormwater runoff from roads quickly and efficiently to mitigate the safety risks. Effective removal of runoff is affected by the geometric characteristics of the roadway, such as longitudinal slope, cross slope, type of curb and gutter section, and ditch section. These geometric characteristics dictate the location and spacing of inlets and catch basins.

Flow in Curb and Gutter

Most urban and suburban land development projects of moderate to high density use curb or curb and gutter. For drainage purposes, the gutter is considered as part of the pavement width. In many cases, streets are constructed with composite gutter and pavement sections, where the gutter

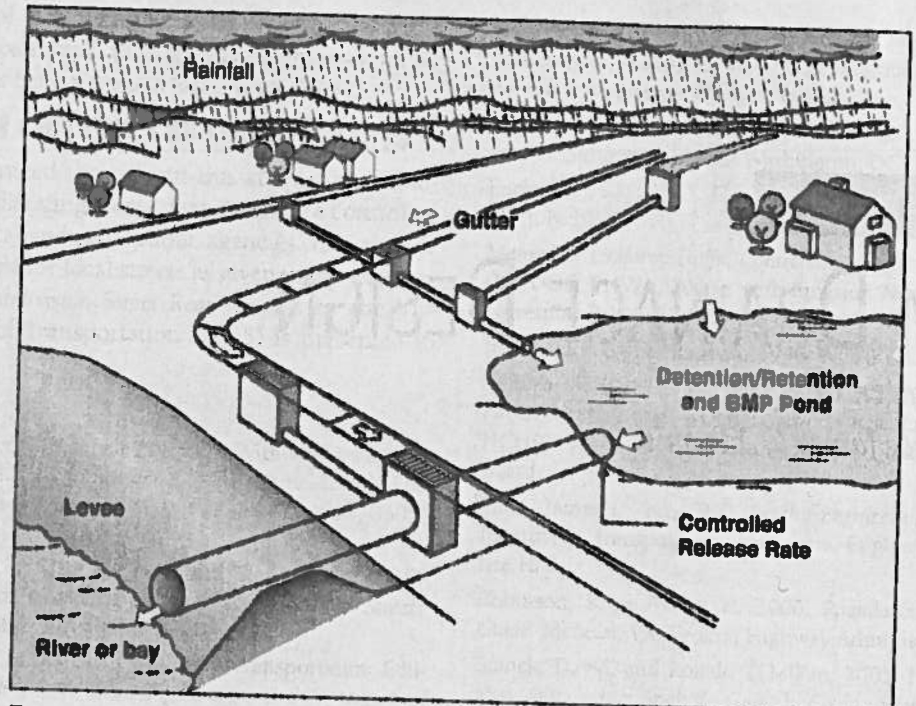

FIGURE 21.1 Typical urban drainage system.

portion has a greater cross slope than the pavement. A composite gutter section results in a decrease in elevation of the flow line at the face of the curb relative to where the flow line would be if the pavement were extended to the face of the curb at the designed cross slope. In these instances, the gutters typically vary between 1 foot and 3 feet wide, depending on local standards. The steeper cross slope of the gutter concentrates the flow toward the gutter to improve inlet efficiency and minimize the width of the runoff as it encroaches onto the travel lanes (spread).

There are some examples, such as on the inside (left) lane adjacent to a raised median, where composite gutters are not provided and the roadway surface and slope tie directly into the face of the curb. These are sometimes referred to as *header curbs*, and the engineer should consider the lack of the gutter section when calculating flow along curb and gutter sections. The Federal Highway Administration's HEC-12, *Drainage of Highway Pavements*, is the basis for many local guidance design standards and criteria.

Flow conveyed in gutter sections with uniform cross slope is:

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3} \qquad (21.1)$$

which is a modification of the Manning equation and accounts for the disproportionate ratio of depth to top width of water surface. In the equation, Q = discharge (cfs), n = Manning's n coefficient, S_x = pavement cross slope (ft/ft), S = longitudinal slope of the gutter (ft/ft), T = width of flow or spread (ft), which is the distance from the face of the curb to the water line limits in the pavement. Values for Manning's n coefficient for various pavement types are given in Table 21.1. The nomograph in Figure 21.2 is the solution to this equation for either V-shaped or triangular gutter configurations.

TABLE 21.1 Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, *increase* above values by n of	0.002

Note: Estimates are by the Federal Highway Administration.
Reference: USDOT, FHWA, HDS-3 (1961).

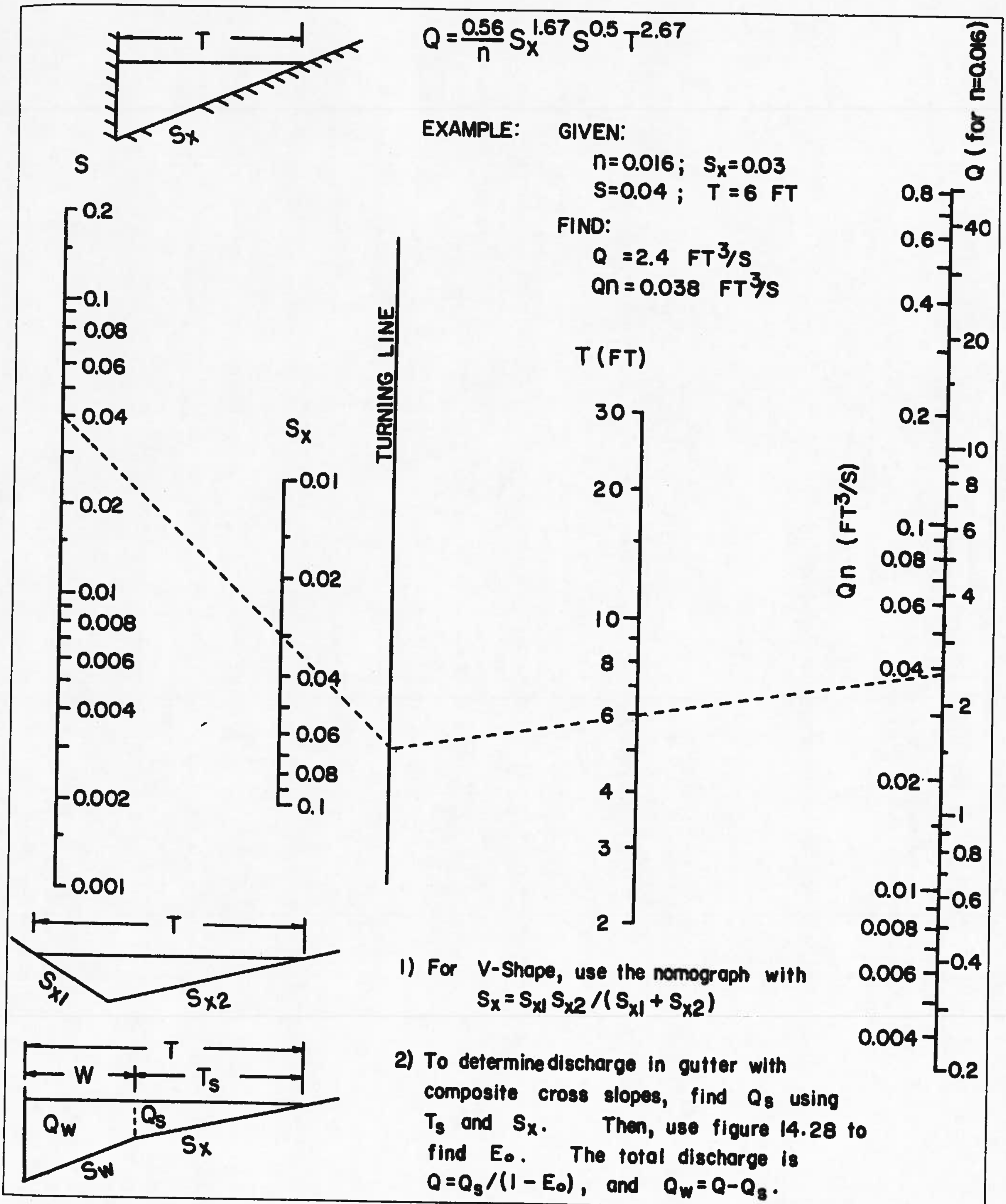

Figure 21.2 Nomograph for flow in triangular gutter sections.

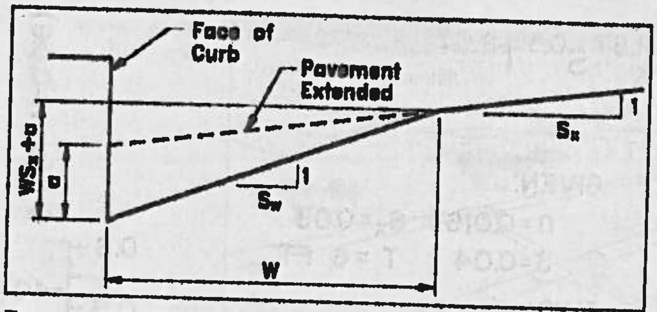

FIGURE 21.3 Composite gutter section nomenclature.

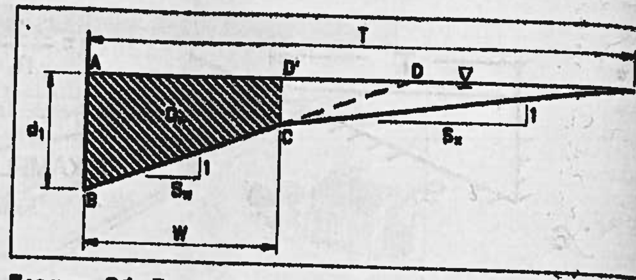

FIGURE 21.5 Flow in gutter section.

The cross slope of the depressed gutter section measured relative to the pavement cross slope is designated as S'_w and is equal to the depression divided by the width of the gutter:

$$S'_w = \frac{a}{W} \tag{21.2}$$

Hence, the actual cross slope S_w is:

$$S_w = S'_w + S_x \tag{21.3}$$

The relationship between these terms is illustrated in Figure 21.3.

Flow in composite gutter sections is computed by separating the flow into two prisms: (1) the flow in the pavement section and (2) the flow in the depressed gutter area. The flow in the pavement section is:

$$Q_s = \frac{0.56\, d_2^{8/3} S^{1/2}}{nS_x} \tag{21.4}$$

where $d_2 = (T - W)\, S_x$, as shown in Figure 21.4; d_2 is the depth of flow at the break point of the gutter-pavement interface; and W is the width of the gutter.

The flow in the gutter section is given as:

$$Q_w = \frac{0.56\,(d_1^{8/3} - d_2^{8/3}) S^{1/2}}{nS_w} \tag{21.5}$$

where d_1 is the depth of flow at the curb and S_w is the cross slope of the gutter. Note that Equation 21.5 computes the flow in prism ABCD' by subtracting the flow in triangle CDD' from the flow in triangle ABCD of Figure 21.5.

Because of the numerous variables involved in the computations of flow in composite gutters, a relationship between the gutter flow ratio $Q_w/Q(= E_0)$, W/T, and S_w/S_x has been developed. Figure 21.6 is a nomograph depicting this relationship and assumes that the Manning's roughness coefficient for the gutter and pavement are the same.

In most urban areas, the allowable spread of the flow into the street is given in local design manuals. The general rule of thumb for urban-type secondary streets is to limit the spread to one-half the lane width as measured from the curb. Some primary road designs, such as interstates, require that the spread not encroach at all on the travel lane. The spread is a function of the longitudinal and transverse slope of the pavement. Typically, drainage design of streets involves the sizing and spacing of the inlets based on limitations of the spread, with a given geometry of the street. However, to determine the spread T, given other parameters, involves a trial-and-error procedure, since no direct solution for T is available from Equations 21.4 and 21.5.

EXAMPLE 1

Find the spread of flow for a street section with a gutter width W of 2 feet, curb height = 6 inches, S_w = 1 inch per foot, pavement cross slope of ¼ inch per foot

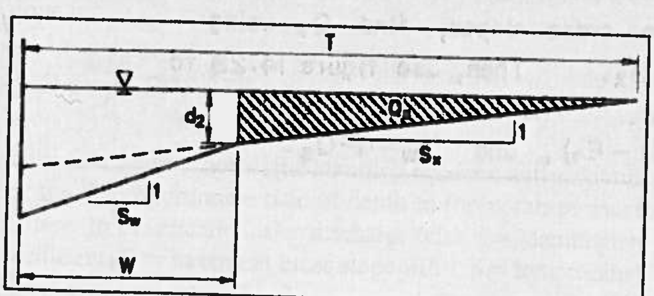

FIGURE 21.4 Flow in pavement section.

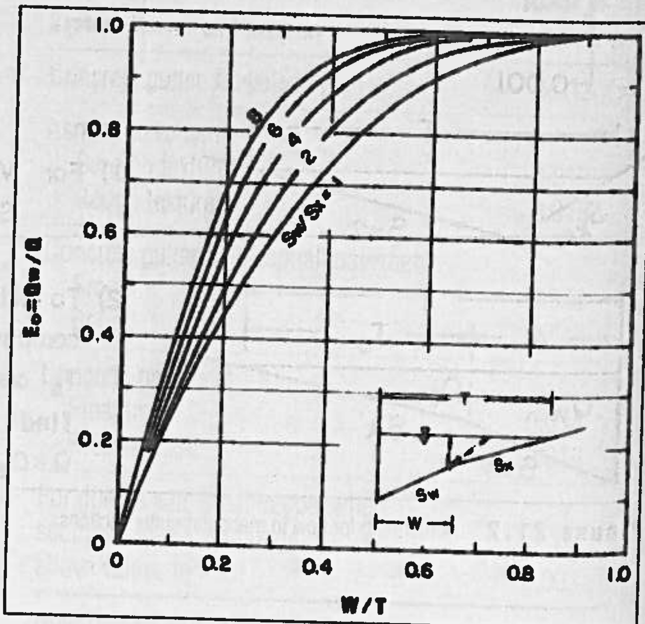

FIGURE 21.6 Ratio of frontal flow to total gutter flow.

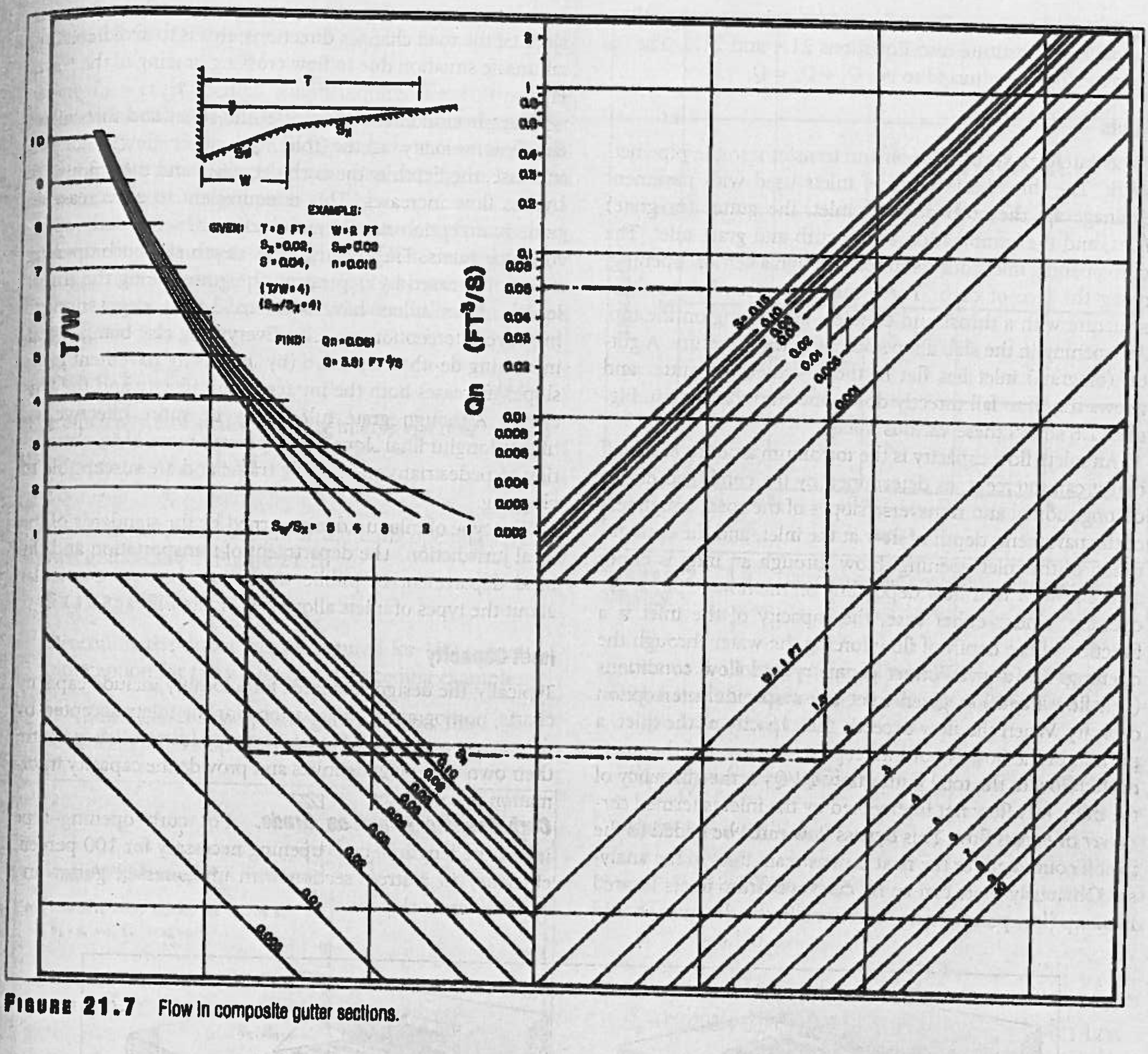

Figure 21.7 Flow in composite gutter sections.

($S_x = 0.0208$), longitudinal slope of 3 percent, and a design discharge of 3.5 cfs (assume $n = .015$ for both pavement and gutter).

Equations 21.4 and 21.5 can be used simultaneously to solve for the two unknowns T and Q_w (recall that $Q = Q_w + Q_s$). Because a direct solution is difficult due to the exponential form of the equations, the nomograph of Figure 21.7 can aid the computations.

Extend a horizontal line from 0.053 [(= 3.5)(0.015)] on the Q_n to intersect the $S = 0.03$ line. From this point, draw a vertical line to intersect the $W = 2$ feet line. Continue horizontally and intersect the $S_x = 0.02$ at the bottom left quadrant of the nomograph. Extend a vertical line to $S_w/S_x = 0.083/0.020 = 4.1$. A horizontal line drawn from this point to the T/W scale gives a value of 4.0. This translates to a spread of $T = (2)(4.0) = 8.0$ feet.

Check the depth at the curb:

$$d_1 = TS_x + W(S_w - S_x) \tag{21.6}$$

$$= (8.0)(0.02) + 2(0.083 - 0.02)$$

$$= 0.29 \text{ feet}$$

Since 0.29 feet < 0.5 feet, the design flow will not overtop the curb.

Often the values for S_x, S_w, and S must be interpolated on the nomograph, which may contribute to some error in reading the T/W scale. In these situations the estimated value for T can be used as a beginning trial value to substitute into Equations 21.4 and 21.5. The value for T is adjusted to get $Q_s + Q_w = Q$.

Inlets

Inlets intercept surface runoff and transfer it to the pipe network. The three basic types of inlets used with pavement drainage are the curb opening inlet, the gutter (or grate) inlet, and the combination of the curb and grate inlet. The curb opening inlet collects runoff through a vertical opening, along the face of curb. These inlets consist of a manhole structure with a throat and concrete slab fitting on the top. An opening in the slab allows access to the structure. A gutter (or grate) inlet lies flat in the pavement or gutter and allows runoff to fall directly down and into the system. Figure 21.8 shows these various inlets.

An inlet's flow capacity is the maximum amount of runoff that it can intercept, as determined by the combined effects of longitudinal and transverse slopes of the road, roughness of the pavement, depth of flow at the inlet, and the configuration of the inlet opening. Flow through an inlet is either orifice flow or weir flow depending on the flow depth at the opening. Under either case, the capacity of the inlet is a function of the depth of flow forcing the water through the opening. For a given street geometry and flow conditions (i.e., flow depth), a given inlet has a specific interception capacity. When the flow exceeds the capacity of the inlet, a portion of the flow is not intercepted. The ratio of the intercepted flow to the total gutter flow Q_i/Q_T is the efficiency of the inlet. Any flow not intercepted by the inlet is termed *carryover* or *bypass* flow. This bypass flow must be added to the runoff computed for the next downstream inlet in the analysis. Obviously there can be no carryover from inlets located in sump (low point) areas. Additionally, the designer should limit the carryover flows in pavement areas where the cross slope of the road changes directions; this is to avoid creating an unsafe situation due to flow crossing or icing of the travel lanes.

The physical characteristics of the street and inlet affect the flow velocity at the inlet. As gutter flow velocities increase, the depth at the curb decreases and the amount of bypass flow increases. This is equivalent to a decrease in both interception capacity and efficiency for curb opening–type inlets. The effective flow depth at a curb opening inlet is increased by depressing the gutter along the throat length. Most inlets have a 2- to 3-inch depression to improve interception capacity. Everything else being equal, increasing depth at the curb (by increasing pavement cross slope) increases both the interception capacity and the efficiency. Although grate inlets may be more effective on higher longitudinal slopes, they present more of an obstruction to pedestrians and bicycle traffic and are susceptible to clogging.

The type of inlet used is governed by the standards of the local jurisdiction. The department of transportation and the local department of public works can be very particular about the types of inlets allowed in rights-of-way.

Inlet Capacity

Typically, the design standards for a locality include capacity charts, nomographs, or equations for the inlets accepted by the review agencies. Manufacturers of inlets often perform their own tests on grate inlets and provide the capacity information upon request.

Curb-Opening Inlets on Grade. For curb opening–type inlets, the length of curb opening necessary for 100 percent efficiency in a street section with undepressed gutter and constant cross slope is:

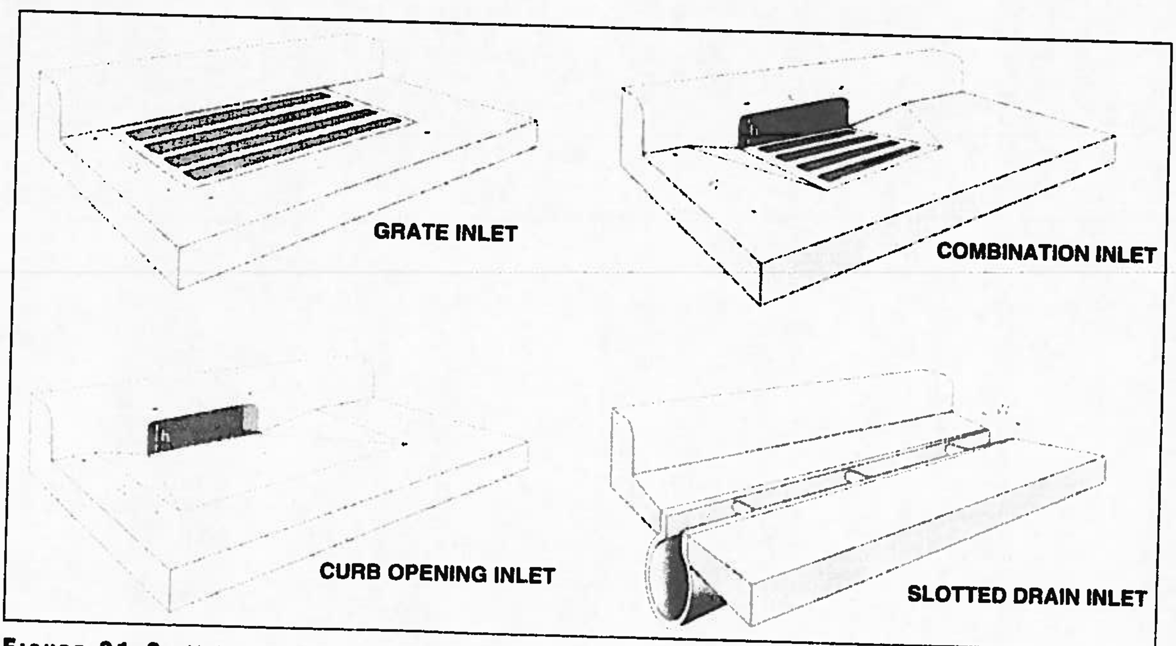

FIGURE 21.8 Various types of curb inlets and gutter inlets.

$$L_T = 0.6\,Q^{0.42} S^{0.3} \left(\frac{1}{nS_x}\right)^{0.6} \qquad (21.7)$$

where L_T = curb opening length required for 100 percent interception (ft), Q = gutter flow (cfs), S = longitudinal street slope (ft/ft), S_x = cross slope (ft/ft), and n = Manning's n coefficient for the pavement. The nomograph of Figure 21.9 can be used in lieu of the equation.

For a depressed curb opening inlet, the length of opening is the same as Equation 21.7, with the S_x term replaced by the equivalent cross-slope term S_e. This term is used to account for the gutter depression. The equivalent cross slope is given as:

$$S_e = S_x + S'_w E_o \qquad (21.8)$$

The efficiency E of a curb opening inlet is given as:

$$E = \frac{Q_i}{Q} = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \qquad (21.9)$$

where L is the actual length of the curb opening. This is shown graphically in Figure 21.10.

EXAMPLE 2

Determine the throat length required for 100 percent interception for the values of the preceding example.

1. Since this is a composite gutter section, the equivalent cross slope is used as calculated in Equation 21.8. Given $S_w/S_x = 4.1$ and $W/T = 0.25$, Figure 21.6 reads a value of $E_0 = 0.70$; S'_w is still required to solve Equation 21.8.

$$S'_w = \frac{a}{W} = \frac{0.125}{2} = 0.063 \qquad (21.10)$$

where $a = w(S_w - S_x)$

$$= 2(0.0833 - 0.0208) \qquad (21.11)$$

$$= 0.125$$

2. The equivalent cross slope is:

$$S_e = 0.0208 + 0.063(0.70) \qquad (21.12)$$

$$= 0.065$$

3. Substituting the values into Equation 21.7:

$$L_T = 0.6(3.5)^{0.42}(0.03)^{0.3}\left(\frac{1}{(0.015)(0.065)}\right)^{0.6} \qquad (21.13)$$

Since most throat lengths are available in 2-foot increments, a 24-foot throat length would be specified for this situation.

Most manufacturers do not make inlets with throats longer than 20 feet. Therefore, the designer would have to consider the overflow from this location to be included at the next downstream location. Or, if 100 percent interception is required, moving the inlet upstream or adding another inlet upstream to reduce

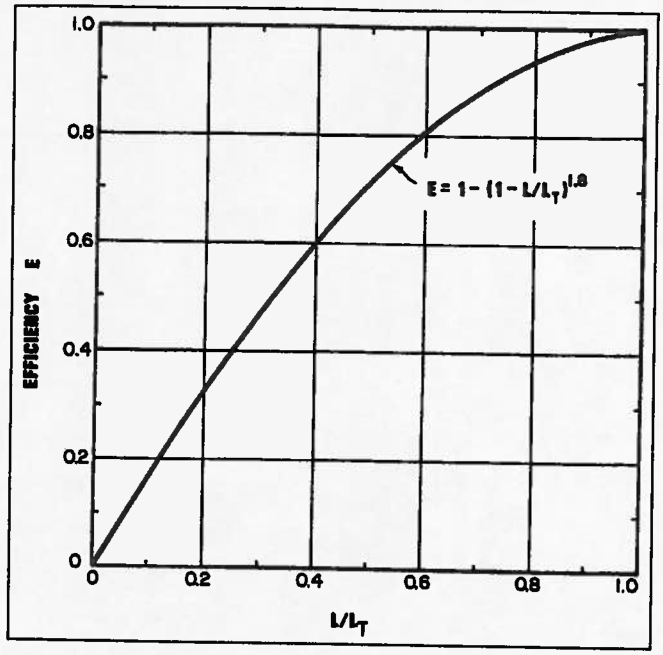

FIGURE 21.10 Curb opening and slotted-drain inlet interception efficiency.

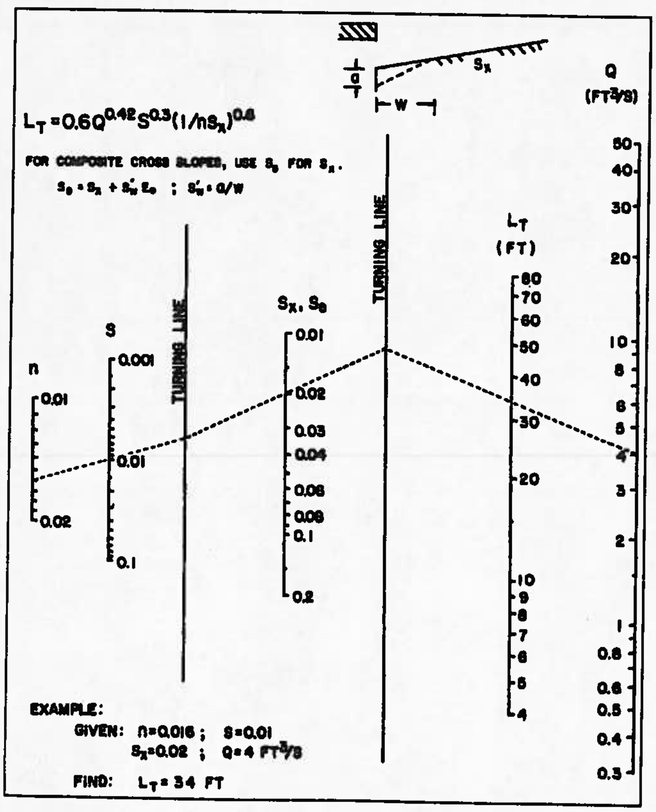

FIGURE 21.9 Curb opening and slotted-drain inlet length for total interception.

the flow to this structure should be considered. Note that once S_e was determined, the required throat length could have been found using Figure 21.9. Also, the answer does not account for the residual storage effects of an additional inlet or local depression, which is standard in many jurisdictions.

Curb Opening Inlets in Sag Locations. These act as a weir for gutter flow depths less than or equal to the height of the opening. At depths greater than 1.4 times the height of the opening, the inlet operates as an orifice. Between these depths, the flow is in transition. The interception capacity for a depressed inlet as given by the weir equation is:

$$Q_i = C_w (L + 1.8W)\, d^{1.5} \qquad (21.14)$$

where C_w is the weir coefficient (≈ 2.3), L is the length of the opening (e.g., throat length), W is the lateral width of the depression, and d is the depth of flow at the curb as measured from the normal cross-slope gutter flow line. For an undepressed inlet, W is equal to 0.

When flow in the gutter exceeds the height of the curb opening inlet by a factor of 1.4, the opening acts as an orifice with the capacity given as:

$$Q_i = C_o h L \sqrt{2gd_o} = C_o A \sqrt{2g\left(d_i - \frac{h}{2}\right)} \qquad (21.15)$$

where C_o is the orifice coefficient (usually 0.67), h is the height of the opening (feet), L is the length of the opening (feet), d_o is the effective head on the center of the orifice throat, A is the area of the opening (square feet), and d is the depth at the lip of the curb opening (feet). Typical opening configurations for curb inlets are shown in Figure 21.11. The computed capacity for the inlet depends on proper interpretation of the height of the opening, since the height depends on the configuration of the throat opening.

Grate Inlets. The capacity of a grate inlet depends on pavement geometry, amount of flow, and the configuration of the grate. Flow through the grate is affected by the size and spacing of the bars as well as their shape and orientation with respect to flow direction.

Any jurisdiction's design standards identify preferred types of grates. Each type of grate has its own hydraulic capacity chart. The Federal Highway Administration (FHWA) identifies several types of grates and their hydraulic characteristics in HEC-12. For those occasions where a standard grate will not suffice, readers are referred to manufacturers of grates for dimensions and hydraulic characteristics.

Grate Inlets in Sag Locations. Flow through grates located at the lowpoint of a sag vertical curve operates as weir flow for relatively shallow depths. As the depth increases, the flow through the inlet transitions to orifice flow. There is no definitive depth where weir flow stops and orifice flow begins. The range of depth for which the transition between weir and orifice flow occurs depends on geometric and hydraulic characteristics of the grate. The weir equation for grate inlets is:

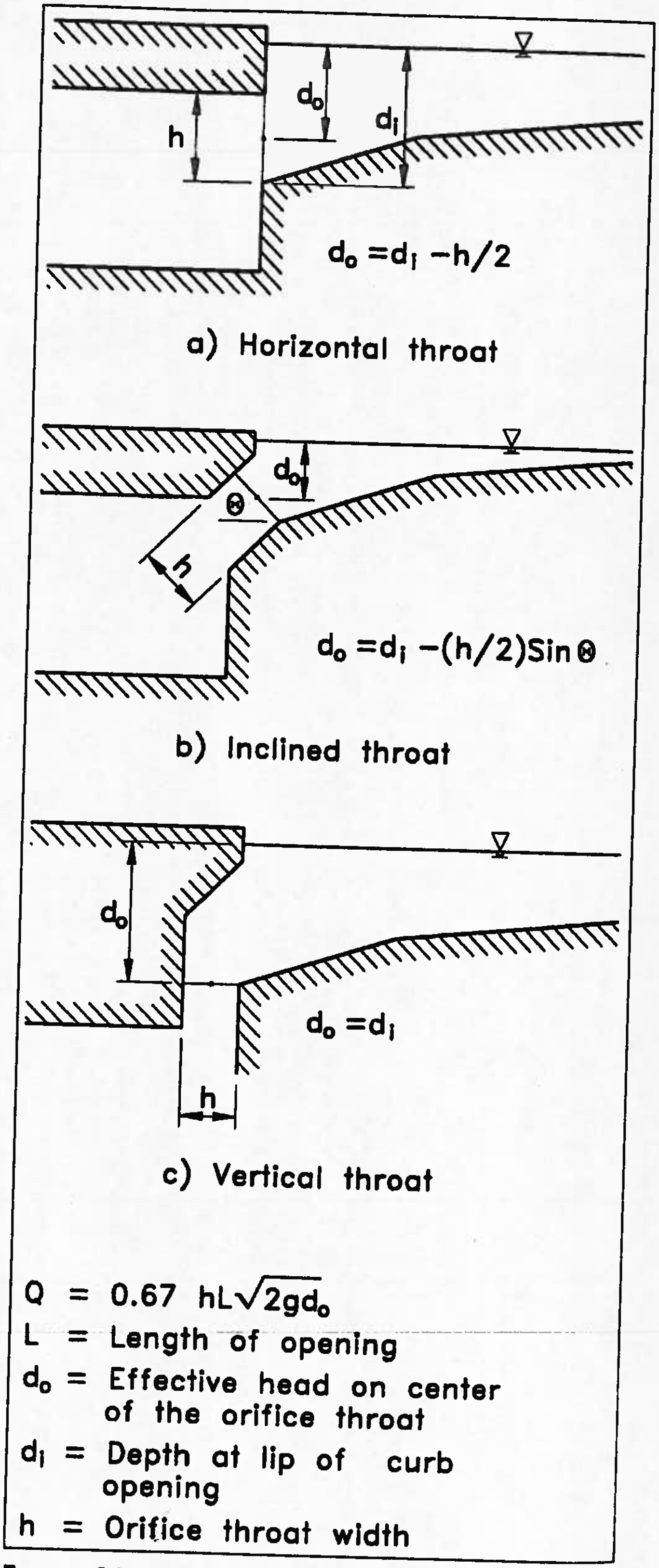

FIGURE 21.11 Throat configuration of curb opening inlets.

$$Q = C_w P d^{3/2} \qquad (21.16)$$

where C_w is a weir coefficient dependent on geometry of the opening ($C_w \approx 3.0$), P is the effective length of opening where weir flow occurs (grates located along the face of curb have only three sides where discharge passes through as weir flow), and d is the depth of flow.

The orifice equation for grate inlets is:

$$Q = C_d A \sqrt{2gd} \qquad (21.17)$$

where C_d is the coefficient of discharge (≈ 0.67), A is the clear opening area of the grate (ft^2), $g = 32.17$ ft/sec^2, and d is the depth of flow. Orifice discharge through the grate depends on the clear open area, which does not include space occupied by bars, vanes, and other obstructions. Since sag locations are highly susceptible to clogging by debris and trash, many localities require an allowance of 50 percent for clogging.

STORM SEWER AND INLET LAYOUT

The storm sewer design, like most design processes, is an iterative process. The results from one iteration provide the details necessary for further refinement of the design in the next iteration. Each iteration attempts to optimize the design in order to provide adequate drainage control, without an excessive number of inlets or length of pipe. In most cases, design of the storm sewer system requires between two and four iterations. The design of the minor system involves:

- Locating the inlets
- Hydrologic analysis to size the inlets
- Determining the pipe network
- Hydrologic and hydraulic analysis to size the pipes
- Checking to see that all constraints are met, which includes dealing with hydrology and hydraulics, compliance with local criteria and standards, and any impacts to other components of the project

Requisite information to begin design of the storm drain system includes on-site and off-site topographic maps, current and projected land use maps, soil identification maps, floodplain delineation maps, floodplain reports, existing site plans of surrounding property, stream and channel outfall information, and the plans of the proposed conditions of the project. Worksheets of the proposed project in plan view, at a scale of 1 inch = 50 feet or larger are preferable. The worksheets should show proposed and existing topography; layout of the proposed streets, buildings, and parking areas; profiles of the road alignment; horizontal and vertical location of all existing utilities; and any proposed utilities. Much of this existing information should already be in project files, having been accumulated during the course of the project.

Location of Inlets in Streets

Inlets are placed at all low points in sag vertical curves. In sag locations, where there is a higher propensity for clogging of grates, the spread of ponded water presents a traffic hazard. Flanking inlets should be considered, if they are not required by code, in the sump area of a sag vertical curve. The addition of flanking inlets, which limit the spread and ponding at the low point, provides relief when the sump inlet is clogged. Table 21.2 is the Federal Highway Administration's recommendation for spacing of the flanking inlets. This recommendation is based on the desired depth at the curb and the vertical curve length defined by $K = L/A$, where L is the length of the vertical curve and $A = g_2 - g_1$ is the algebraic difference of the tangent slopes of the vertical curve.

TABLE 21.2 Distance to Flanking Inlets in Sag Vertical Curve Locations Using Depth at Curb Criteria

Speed (mph):	20	25	30	35	40	45	50	55	60		65	70
$K(= L/A)$:	20	30	40	50	70	90	110	130	160	167	180	220
Depth @ curb												
0.1	20	24	28	32	37	42	47	51	57	58	60	66
0.2	28	35	40	45	53	60	66	72	80	82	85	94
0.3	35	42	49	55	65	73	81	88	98	100	104	115
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	71	84	95	105	114	126	129	134	148
0.6	49	60	69	77	92	104	115	125	139	142	147	162
0.7	53	65	75	84	99	112	124	135	150	153	159	176
0.8	57	69	80	89	106	120	133	144	160	163	170	188

Reference: USDOT FHWA, *Drainage of Highway Pavements*, HEC-12, 1984.

For an example, what is the recommended distance to the flanking inlets for a 300-foot-long vertical curve with longitudinal slopes of $g_1 = -3\%$ and $g_2 = +3\%$, and the depth of flow at the curb is 0.4 feet?

Solving for K:

$$K = \frac{L}{g_2 - g_1} = \frac{300}{(+3)-(-3)} = 50 \qquad (21.18)$$

Entering in Table 21.2 for depth at curb value = 0.4 feet and $K = 50$ gives a distance of 63 feet.

On normal crown street sections two inlets are needed, one on each side of the street at the sump location. Two inlets are also needed if the low point (sump) of a sag vertical curve occurs on a section of road that is superelevated, since the standard gutter on the superelevated side forces the water against the face of curb. Although some localities have a "spill," or reverse gutter, that dispenses the water away from the curb, this is not common in public rights-of-way. In cases where a reverse gutter is an option, only one inlet is needed at the low point of the vertical curve on the low side of the superelevation.

Be aware, however, that in certain situations the location of a low point on the outer edge of the pavement is not always the same as that of the lowpoint on the PGL (profile grade line). For example, the low-point locations may not coincide when a superelevation transition[1] occurs at or near the low point of the PGL. Other potential situations in which this might occur typically involve a superelevation transition and/or pavement widening, as when adding a right-turn lane. To properly locate the drainage inlet, the design engineer should check the elevations of the outer edge of pavement along the superelevation transition segment to determine the location of any false low points.[2] Most urban streets have curbed or curb and gutter sections. In those situations, where the urban street is required to have superelevation through horizontal curves, certain conditions create the situation where a low point occurs and is not in an obvious sump. To attain full superelevation or to go from full superelevation to normal crown (or reverse superelevation in the case of compound curves) requires a length of transition. If the rate of superelevation is greater than the longitudinal rate of grade change, a low point in the gutter area results. Although superelevation transitions through horizontal curves are not considered good practice, this situation, in some instances, cannot be avoided due to other constraints. The engineer should be aware of the conditions that present this predicament and locate the not-so-obvious low point.

Inlet placement in an intersection requires detailed analysis of the intersection street's profiles and cross sections. Intersections have the potential for creating unusual drainage patterns and collection networks. The design of an intersection involves matching the cross slope of the intersecting street with the longitudinal profile of the through street. Curb returns (the horizontal curve portion of the curb used to join the curb of the two intersecting streets) and the cross sections of the intersecting streets, coupled with any horizontal and vertical curves, can create sump areas in the most inconspicuous and least favorable locations.

Drainage patterns can be further complicated by deviations from the cross slopes shown on the typical section, which may be relaxed through the intersection to fit the pavement from the intersecting street to the pavement of the through street. Additionally, the profile of the curb return itself may not follow a known mathematical function—for example, a parabolic curve—and typically does not exactly coincide with the street profile. The intersection's contour grading plan usually provides clues to the drainage pattern and the potential location of sump areas. An experienced engineer will design the intersection with drainage as a priority consideration. For example, locating an intersection within the low area of a vertical curve creates drainage design difficulties. Avoiding a very flat cross slope at or near the lowpoint of a sag vertical curve also facilitates the drainage design in the intersection area.

Often, local design criteria prescribe an upper limit to the amount of flow that can bypass an intersecting street. Typically, the amount of flow crossing the intersection depends on the functional street classification. The higher the functional classification, the lower the amount of discharge allowed to pass the intersection. Occasionally, design criteria may even prescribe that no flow can pass the intersection regardless of the street's functional classification; therefore, the inlet should be located at the ends of the curb returns. The designer should consider the location of any handicap ramps or pedestrian crossings prior to placing inlets at the end of the curb returns. Depending on the longitudinal street slope and the gutter flow, an inlet just upstream of this location may also be necessary. Locating inlets at the ends of curb returns also reduces flows around the curb returns, which may be a consideration if the longitudinal gradient of either street is very steep or if the area is a high-pedestrian-traffic area.

It is difficult to install curb inlets in curb returns with short radii. Therefore, the engineer may decide to show the profile of the curb return to ensure no sump area exists and to guide the contractor that is installing the curb return. Inlets with relatively short throats or short grates may be installed on curb returns with larger radii without disrupting the continuity of the curve. Occasionally, curb returns of large radii (e.g., 100 feet or more) that are longer than normal may be designed to intentionally have a sump area to keep runoff from spilling into the travel area of the intersection.

After inlets have been placed at the required locations, the location of any and all remaining inlets is dictated by the

[1] A superelevation transition is a segment of the road used to gradually change the cross slope of the pavement from a normal crown section (straight section of road) to a superelevation section (curved section of road, like a racetrack curve that is banked), and vice versa.

[2] The term *false low point* is used to indicate a low-point location difference from the low-point location given on the PGL.

limitations on spread of flow into the street. Frequently, the inlet is intentionally sized to intercept less than the total design flow approaching the inlet, thus allowing some flow to carry over and be added into the design discharge for the next downstream inlet. One study concludes that an inlet's capacity is maximized if some water is permitted to bypass the inlet. Therefore, the size and location of street inlets on grade depends on how the design discharge in the street is divided among the inlets.

In parking areas, the grading plan determines the drainage pattern and inlet location. In commercial building sites, pedestrian traffic is an important consideration for grading and drainage. Sump areas should be avoided in pedestrian travel ways such as crosswalks or near entrances to the building. Additionally, sump areas should be avoided in areas where passengers discharge from vehicles such as bus stops or walkways. Although there is typically no limitation for the spread of flow into privately maintained travel lanes and parking areas, good judgment is needed to limit the spread in cold regions, since the formation of ice sheets is a concern. Handicap ramps and parking spaces also need to be considered when locating drainage inlets and sump areas.

Location of Inlets Outside of Paved Areas

The drainage system should be carefully integrated into all projects. Ideally, the drainage system should accommodate the site or roadway engineer's plan, and not be the driving factor behind the remaining design components. In single-family detached projects, drainage patterns favor the rear and side of the houses. Swales and inconspicuous ditches convey the water across the rear of the lots. The rate of flow being conveyed across a lot is limited to nonerosive velocities. No more than 2 to 4 cfs should concentrate and flow across the lots without being intercepted or conveyed in a well-defined and stabilized channel. Maintenance and ownership of swales and ditches is a factor in drainage design in suburban developments. Drainage easements may be necessary. The desirability of a lot is reduced if a drainage easement runs through the middle of a rear yard. In most cases, the drainage system should be kept within the public right-of-way.

Manholes

Manholes are typically precast circular concrete barrel sections, in 3- to 4-foot lengths, that stack on top of each other. The elevation of the top of the manhole is adjusted to meet grade with spacer rings. The top is covered with an iron ring fitted with an iron cover. Manholes are used to change the horizontal and/or vertical direction of pipes, while also acting as a junction, allowing the convergence of several incoming pipes. Many types of inlets use the manhole barrel sections, but have a precast throat or grate that fits to the top instead of the frame and cover.

It is common practice to match crowns of an incoming and outgoing pipe. When several pipes converge at a manhole, matching crowns may not be feasible due to the width of the manhole, the diameters of the incoming pipes, and the angles of approach. The diameter of the manhole depends on the size of the pipes connecting to it. Standard manhole diameters are approximately 4 feet.

In Figure 21.12*a*, the size and angles of the pipes are such that they can be connected to the manhole without interfering with each other. Compare this to Figure 21.12*b*, where the pipe sizes and angles cannot fit into the manhole at the same elevations (or nearly the same elevations). In the second case, the elevations of the pipe should be staggered enough to provide for the necessary clearance. In each case, the thickness of the pipe walls should be considered when determining the necessary manhole size. Note: A rule of thumb for the wall thickness of a standard reinforced con-

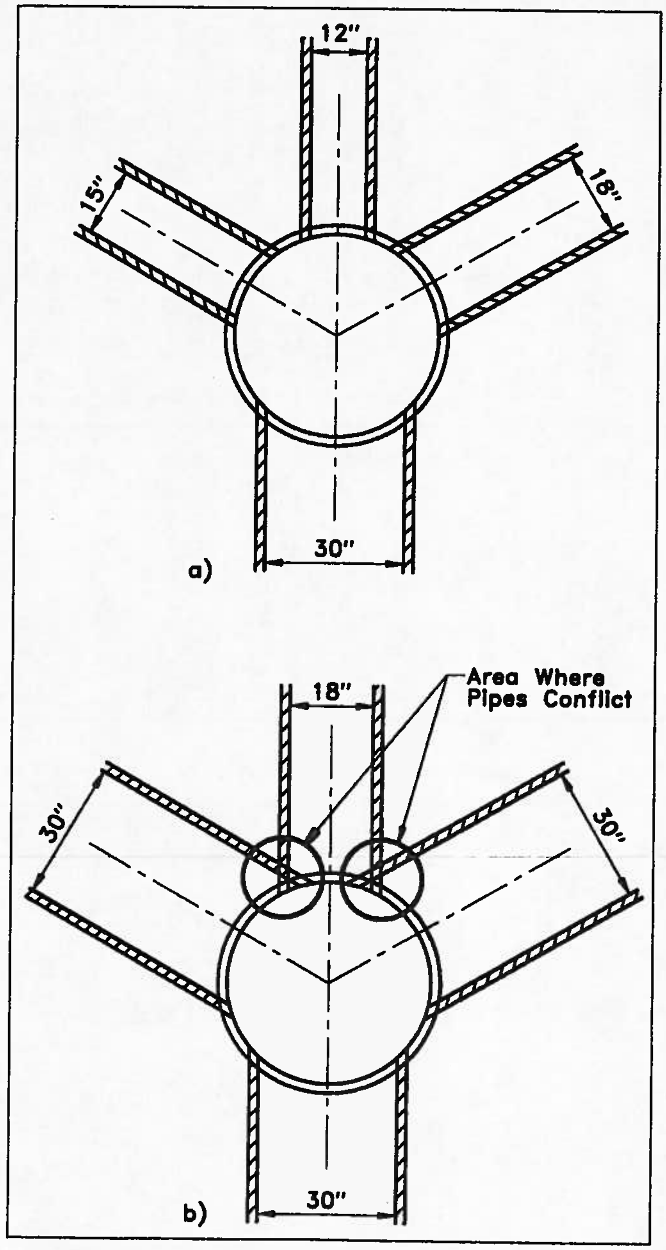

Figure 21.12 Pipes connecting at manhole at same elevation.

crete pipe is as follows. The wall thickness (in inches) is equal to the diameter of the pipe (in feet) plus 1. For example, a 48-inch pipe will have a thickness of 5 inches (4-foot diameter + 1).

Similar to the preceding discussion, where several pipe connections to the manhole must be checked for proper fit, the skew angle of the pipe connection at a rectangular structure must be checked for fit. Figure 21.13*a* shows an incoming 36-inch-diameter pipe properly fitted to a 3-foot by 5-foot (outside dimensions) structure. In Figure 21.13*b*, the pipes are skewed and do not properly connect to the structure. In this example, the center of the structure is aligned with the centerline of the pipe, and the skew angle causes part of the incoming pipe to overlap onto the 3-foot side of the structure.

In most moderate-density developments, manholes and inlet structures are spaced less than several hundred feet apart by necessity. Most localities set design criteria limiting the maximum distances between manholes, which often are a function of the pipe size. Table 21.3 is the recommended spacing provided by AASHTO. This distance is usually determined by maintenance and accessibility concerns; verify acceptable distances per locality.

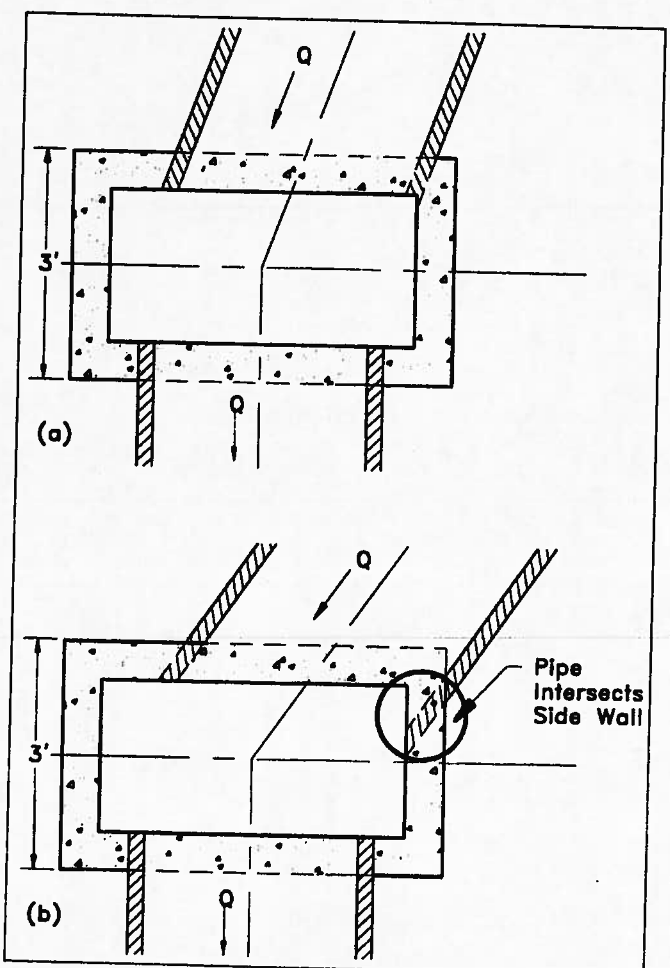

FIGURE 21.13 Skew angle for pipes connecting to rectangular structure.

TABLE 21.3 Recommended Manholes Spacing

PIPE DIAMETER	DISTANCE (FT)
12–24	350
27–36	400
42–54	500
≥60	1000

Stormwater Management Considerations

Oftentimes, the storm sewer and pipe design of a roadway drainage system are governed by the need to convey water to a stormwater management (SWM) facility. SWM facilities can include large ponds, basins, infiltration areas, grassed swales, or other devices meant to reduce peak flow runoff or to treat runoff for water quality purposes. Much of the focus on today's stormwater management design, for roadway and land development projects, is on water quality. Therefore, even if the designer can provide the necessary detention or prove that detention requirements are not warranted, more than likely water quality requirements will still be required. Given the site constraints or the SWM needs required for the development, most storm sewer systems and inlets are driven by the need to collect runoff and discharge it into an SWM facility.

With the growing demand for re-development and the emphasis on water quality, numerous manmade structures are being used to provide water quality. There are a variety of water quality inlets that help to remove oil, grit, sediment, and pollutants. Many of these devices are underground structures varying in size (from regular-sized manholes to large-vault boxes) that are connected directly to a storm sewer system. Given the amount of flow or drainage area accumulated in a storm sewer system, the engineer may decide to design a system that diverts a portion of the runoff to a water quality inlet. This will divert the first flush of the stormwater (in theory, the runoff with the most pollutants) into the water quality inlet, while allowing the remainder of the stormwater to flow through the pipe system. When stormwater is diverted within a system, the water quality inlet is said to function off-line. When the water quality inlet is placed within the storm sewer system itself (usually on small catchment areas only), the system is considered to be on-line.

Pipe Design

After inlet locations and sizes are established, the pipe conveyance system is determined. The pipe network typically converges to the outfall point. Occasionally, due to site conditions, the network is separated into several systems, each discharging at a different outfall point or connecting to another storm sewer network. Outfall points can be natural channels of adequate capacity, retention/detention areas, lakes, and rivers. The size, slope, and depth of the pipe in the network are controlled by the elevation of the outfall point. This is more of a problem in flat terrain than in rolling or hilly terrain. In flat terrain, the engineer may have design problems trying to match outfall elevations, whereas in steep terrain, high velocity and high-energy losses may create a potential problem. In general, the slope and size of the pipe are kept to the minimum required to carry the design flows at near full capacity, which helps ensure proper cleanout of the storm sewer systems. Recognize, however, that many public agencies have a requirement for a minimum allowable pipe size and slope. For storm sewer systems in steep terrain, the minimum pipe size may have excess capacity.

The pipe network, wherever possible, should be located within the public right-of-way. Most storm sewer systems are maintained by a government agency. Therefore, any part of the conveyance system on private property, requires an easement to allow access. Additionally, it is desirable to locate the pipe parallel to a property line, offset to one side. This allows for maximum use of the property (i.e., no encumbrances through the middle of the lot) and also allows an owner to build a fence without having to dismantle it, if the pipe needs to be repaired.

Common practice is to set the pipe at minimum depth (i.e., minimum cover or minimum inlet depth) or at the shallowest depth possible (to ease in constructability) and select the pipe size and slope that convey the design discharge at near full capacity. The pipe network is typically designed such that the HGL is at or below the crown of the pipe (i.e., nonpressure flow). Under such conditions, the pipe network is surcharged for storms that generate runoff greater than the design storm. The HGL should be checked against this secondary, or check, storm—the local design standards may even require it. The designer should determine the overland relief path for the larger secondary storm surcharges for safety or property damage concerns. Site grading or alignment, which would result in these surcharges damaging private property, should be rethought.

Discharge velocity in a pipe should be kept within a nominal range to prevent scour and eliminate sediment buildup. Although efforts are made to keep sediment from entering the system, all sediment cannot be kept out. Sediment transported by very high velocities causes abrasion damage to the pipe. Low velocities cause sediment to settle out and reduce the pipe capacity. A recommended velocity range is 2 to 12 feet per second (fps).

Avoiding conflicts with other underground utilities is a major objective in the design of a storm sewer system. Since storm sewer flow is driven by gravity, any conflicts with other proposed non-gravity-dependent utilities are usually resolved by redirecting those utilities. Attempts are made to locate the proposed storm sewer system around any existing gravity or non-gravity-dependent utilities. It can be costly to redirect existing utilities, and only in the unavoidable case is this done.

Pipe Materials

Pipes can be manufactured from many different materials, including concrete, reinforced concrete, aluminum, steel, and various other synthetic materials. Some of the synthetic materials, such as high-density polyethylene (HDPE) pipe, come in smooth lines or corrugated, depending on engineering needs. HDPE pipe has become a favorite among developers due to the material and installation costs. It is important to determine the availability of pipe material selection per locality specifications and site constraints. The pH level and resistivity of the soil, whether it is a high- or low-fill area, corrosive environmental conditions, and tidal areas are some of the site constraints. Others may be the preference of contractor, the proximity to manufacturing plants, and the cost of the different pipe materials—the engineer must be careful to consider the pipe material when designing a storm sewer system. The various different pipes available on the market can have an enormous range of n values (from as little as 0.007 to as high as 0.033). Since n value plays an important role in pipe capacity and hydraulic grade line calculations, the selection of the pipe material could dramatically affect the design.

Some localities provide guidance on the types of materials preferred. Many other jurisdictions allow the contractor to choose the pipe material. When the contractor is given the option of pipe materials, it is up to either the contractor or the engineer to determine whether the material selected still meets the design requirements of the construction documents. Therefore, it is important that the engineer understand the requirements of the local industry before determining pipe materials to be used for design and analysis.

Underdrain Design

The inlet and storm sewer design focused on drainage pavements, which have surface water runoff. However, there are many instances where water collects underneath the pavement section, either in locations of cut/fill transitions or just by infiltration. Water usually collects in the subgrade portion of the pavement, especially in areas where aggregate or stone is used in the design. This collection of water underneath the pavement can be extremely problematic, leading to loss of stability and strength of the pavement structure, fatigue based on freeze/thaw conditions, or loss of cohesion and bearing pressures of the surrounding soil.

Given certain situations, this excess water underneath the pavement requires that a secondary drainage system be constructed to reduce the water collected underneath roadways. This is usually done by placing a combination of small plastic pipes (sometimes perforated or wrapped in geotextile fabric) along the aggregates section of the pavement box and discharging the water into either a ditch or a structure (i.e., inlet or manhole) associated with the roadway drainage system. These devices are called *underdrains*.

In most urban situations, underdrains are placed along the edge of the pavement, below the curb and gutter section.

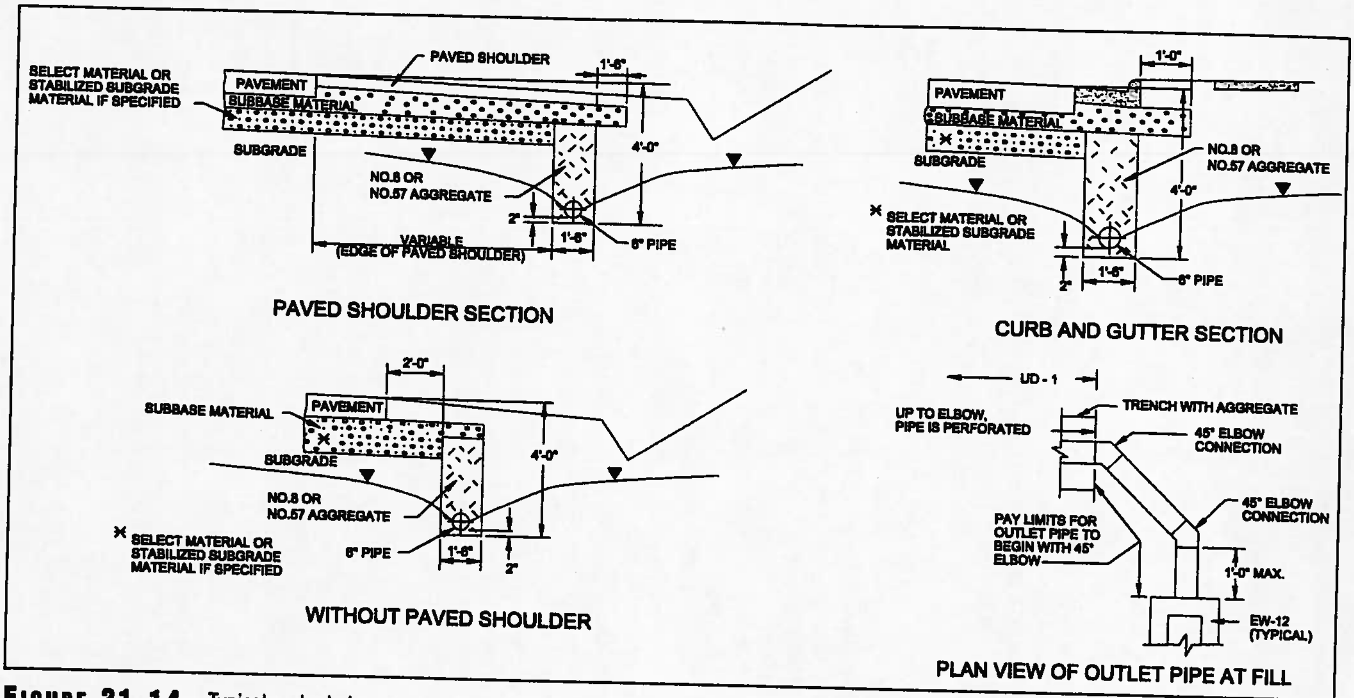

Figure 21.14 Typical underdrain.

Typical roadway design requires that the pavement be sloped to the edge to facilitate drainage. The same applies to pavement design as well, such that all layers of the pavement box (top surface coarse, base coarse, subgrade, etc.) drain to the outside. Curb and gutter sections typically are constructed on stone or aggregate bases, meaning that water can flow easily along the corridor. Underdrains placed longitudinally below the curb line, within the aggregate, can capture the majority of water trapped beneath the pavement and discharge it directly into the roadway drainage system. Figure 21.14 shows a typical underdrain.

OPEN CHANNEL FLOW IN PIPES AND CHANNELS

The two basic forces that act on flow in an open channel are gravity (pulling it downward) and friction (retarding its movement). These factors then have to be combined with the basic concept of continuity, which is that the flow in one discrete section of channel or pipe must equal the flow out of that section. This is the basic premise for estimating open channel flow in channel and pipes. Therefore, open channel flow Q is estimated by combining the continuity equation $Q = VA$ with Manning's equation for velocity.

$$Q = \left(\frac{1.49}{n} \times R^{2/3} S^{1/2}\right) \times A \tag{21.19}$$

where the term in parentheses represents Manning's velocity for open channel flow. In Equation 21.19, n = Manning's coefficient of roughness; R = hydraulic radius (ft), defined as the ratio of the cross-sectional area to the wetted perimeter; S is the slope of the energy grade line (ft/ft); and A is the cross-sectional area of the flow (ft^2). In the equation, the term in parentheses represents the velocity of flow as per the Manning equation. Natural conveyance systems (i.e., swales, ditches, and channels) are increasingly popular in residential applications and/or slower-speed roadway environments. Natural channels are subject to the same theories reviewed in open channel flow; however, they are discussed more thoroughly in the "Stormwater Management Considerations" section, as they have additional benefits beyond just conveyance that are more relevant to quantity/quality control.

The crosshatched area of Figure 21.15 shows the cross-sectional area of flow for a circular section and a trapezoidal section. The wetted perimeter for the circular section is arc ABC. Line segments ABCD show the wetted perimeter on the trapezoidal section.

The roughness coefficient represents an estimate of the resistance of flow. Large n values correspond to high resistance to flow. Table 21.4 provides n coefficients for commonly occurring channel materials.

Following are some of the factors that affect flow resistance:

- Surface roughness, due to size and shape of the grains of the material forming the wetted perimeter, increases the roughness coefficient as the coarseness of the grains increases.
- Vegetation is also a form of surface roughness. The degree to which vegetation impacts flow resistance depends on density, height, and type of vegetation.
- Channel irregularities such as those found in natural streams. Channel irregularities include depressions in the channel bed, humps, sandbars, ridges, and so on, which change the wetted perimeter size and shape of

cross section. Abrupt channel changes have a larger effect on the n value than small or gradual irregularities.

- Channel alignment and curvature have some impact on flow resistance. A large radius of curvature has less effect than a short radius.
- Obstructions such as logs, stumps, debris, trash, rocks, and other items that interrupt and severely alter the flow path increase the flow resistance.

In a circular pipe flowing full with diameter D, the hydraulic radius is $D/4$. In a pipe flowing less than full, the relationship between depth y, cross-sectional flow area A, wetted perimeter P, and hydraulic radius R is illustrated in Figure 21.16 and given by the following:

$$y=\frac{D}{2}(1-\cos\theta) \tag{21.20}$$

$$A=\frac{D^2}{4}(\Theta-\sin\Theta\cos\Theta) \tag{21.21}$$

$$=\frac{D^2}{4}(\Theta-1/2\sin 2\Theta)$$

$$P=D\Theta \tag{21.22}$$

$$R=\frac{A}{P}=\frac{D}{4}\left(1-\frac{\sin\Theta\cos\Theta}{\Theta}\right) \tag{21.23}$$

which simplifies to

$$R=\frac{D}{4}\left(1-\frac{\sin 2\Theta}{2\Theta}\right)$$

Energy Losses in Pipe Systems

Flow in a conduit is retarded by resistance and turbulence. Resistance and turbulence are measured in terms of the energy consumed to overcome them, which is mostly in the form of frictional losses along straight sections of the conduit. Turbulence is also created where there is an abrupt

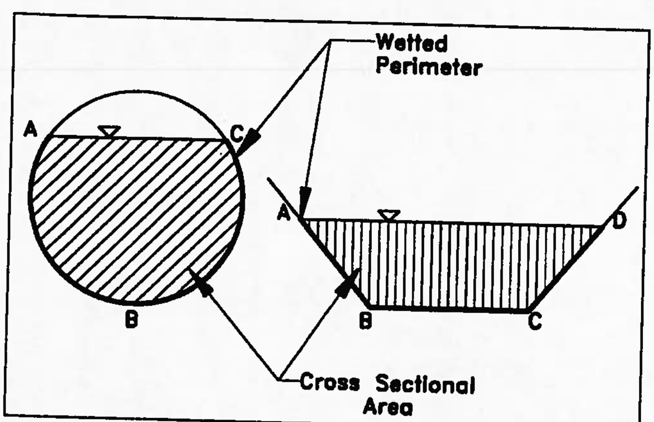

FIGURE 21.15 Cross-sectional area and wetted perimeter for circular section and trapezoidal section.

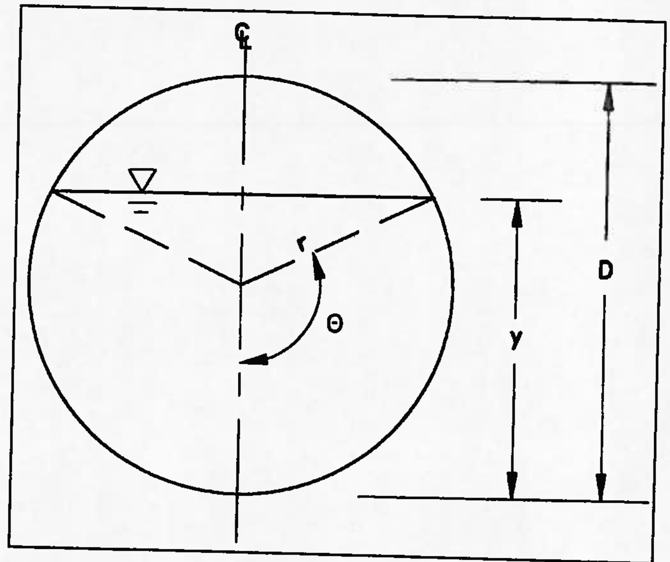

FIGURE 21.16 Nomenclature for pipe flowing partially full.

change in cross section of the flow (e.g., sudden contraction or sudden expansion) or by other interferences of the flow. The concept of uniform flow in open channels is a balance between the available energy that is generated by gravity and the consumed energy of the flow resistance.

Uniform flow occurs whenever the depth, water area, velocity, and discharge at every section in a channel reach remain constant. For such stringent conditions to exist, not only must channel geometry be consistent, but the drop in potential energy (due to the fall in elevation in the channel bed) must be equal to the energy consumed through boundary friction and turbulence. Hence, if the channel geometry and slope remain constant and the reach is sufficiently long, then for a given discharge there will exist one, and only one, depth at which the flow will be uniform. This depth is referred to as *normal depth*.

The energy at a particular location in a pipe is given by Bernoulli's equation:

$$E=\frac{p}{\gamma}+\frac{V^2}{2g}+z \tag{21.24}$$

where p/γ is the pressure head,[3] $V^2/2g$ is the velocity head, and z is the elevation head. A plot of points given by Equation 21.24 at every section along a length of a pipe is known as the *energy grade line* (EGL). By utilizing the conservation of energy principle along with the Bernoulli equation, the energy at point B, downstream of A, is less than the energy at point A by an amount equal to the energy lost between points A and B. Mathematically, this is expressed as:

$$\frac{P_A}{\gamma}+\frac{V_A^2}{2g}+z_A=\frac{P_B}{\gamma}+\frac{V_B^2}{2g}+z_B+\Sigma h_L \tag{21.25}$$

[3]In hydraulics, the term *head* is synonymous with energy.

TABLE 21.4 Values of the Roughness Coefficient n

Type of Channel and Description	Minimum	Normal	Maximum
A. Closed Conduits Flowing Partly Full			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlets, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminate, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

TABLE 21.4 (Continued)

Type of Channel and Description	Minimum	Normal	Maximum
B. Lined or Built-up Channels			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom bloat-finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	—	0.500

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where the Σh_L term accounts for the total energy lost between points A and B.

The energy gradient (hydraulic slope) is defined as the energy loss per length of channel:

$$S = \frac{h_L}{L} \tag{21.26}$$

In closed conduits under pressure flow, the potential energy plus the pressure energy in the system is given as $z + P/\gamma$. This term, also referred to as the *static head* or *piezometric head*, represents the level to which the liquid will rise if a piezometer tube were installed at this location. A continuous line drawn through the tops of these piezometer columns is the *hydraulic grade line* (HGL). In open channel flow, the water surface itself is the hydraulic grade line. In uniform flow, the slope of the channel, the slope of the water surface, and the slope of the hydraulic gradient are all parallel.

Head loss is attributed to pipe friction, abrupt changes in the cross-sectional area (due to entrance and exit of flow to or from a reservoir or manhole), or other types of appurtenances and obstructions within the flow path. The two most commonly used friction loss equations for determining hydraulic resistance are the Darcy equation and the Manning equation. The Darcy-Weisbach equation is given as:

$$h_f = f\left(\frac{L}{D}\right)\left(\frac{V^2}{2g}\right) \tag{21.27}$$

where h_f = friction energy loss, f = Darcy-Weisbach friction factor, L = length of conduit, D = diameter of the conduit, V = mean velocity in the conduit, and g = gravitational acceleration.

The Darcy-Weisbach friction factor is obtained from the Moody diagram (see Figure 25.33), which relates f to the Reynolds number and the relative roughness of the conduit.[4] Conversion of the Darcy-Weisbach friction factor f to the Manning roughness coefficient n is given by:

$$n = 0.0926\, R^{1/6} f^{1/2} \tag{21.28}$$

where R is the hydraulic radius.

In the Manning equation, the slope of the energy line S is given as $S = h_f/L$, where h_f is the friction loss and L is the length of conduit. Substituting for S and solving for h_f in the Manning equation gives:

$$h_f = \left(\frac{29\, n^2 L}{R^{4/3}}\right)\frac{V^2}{2g} \tag{21.29}$$

In addition to pipe friction energy losses, there are minor energy losses through manholes, bends, and other appurtenances. Energy losses through such appurtenances are proportional to the velocity head in the appurtenance and are accounted for as follows:

$$h_L = k\frac{V^2}{2g} \tag{21.30}$$

where k is an empirical coefficient that accounts for the energy loss for a particular appurtenance or configuration. Table 25.13 gives k values for various conditions).

Minor energy losses through manholes and junctions are the result of the deflection angle of the flow, entrance losses as the flow enters the discharge pipe from the manhole, and exit loss as the flow leaves the incoming pipe. As indicated by Equation 21.30, high velocities result in high energy losses. However, the energy loss through the manhole can be significantly reduced through inlet shaping. Inlet shaping provides a flow trough at the bottom of the manhole to direct the flows from the incoming pipes to the downstream pipe. Exit loss coefficients, energy loss coefficients, and other loss coefficients are typically identified in the local standards criteria. Figure 21.17 gives some values for k as prescribed by the American Association of State Highway and Transportation Officials (AASHTO).

In addition, losses for deflection of flows from the incoming pipes to the outgoing pipe have to be considered. When a junction or manhole has more than one incoming pipe, each pipe is analyzed for the total head loss of the combined effects (entrance, exit, deflection, etc.). The pipe producing the greatest head loss is assumed to be the controlling pipe, and the EGL and HGL are based on this greatest value.

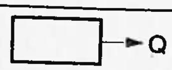
Energy losses also occur at inlets that collect surface runoff and discharge directly into the storm sewer system. The water dropping into the bottom of the manhole produces turbulence within the water that is already flowing through the manhole, thus disturbing (even further) the flow from the upstream pipes. These energy losses are added to the conventional losses attributed to the exit and entrance losses from the discharge ends of the incoming pipes. Other losses, which are less frequently encountered in the pipe network, include diffusor/confusor losses, losses associated with converging or diverging wyes, and bends. Figure 21.17 summarizes the various minor energy losses through structures.

Hydraulic Grade Line in Pipe Systems

Most storm sewers are intended to operate as gravity flow for the design discharge. Occasionally, due to unavoidable field conditions such as high flow rates or low-lying flat areas where little fall is available, the system is surcharged (i.e., operates as pressure flow). Even though the pipes were sized to function under gravity flow, accumulated head losses can cause surcharge in the system if, during design, sufficient allowances were not made to compensate for the accrued head losses. Surcharge in the system occurs when the hydraulic grade line rises above the crown of the pipe.

Most localities prefer the storm drainage system to operate as gravity flow for the design discharge. However, surcharge of the system for the design flow is tolerable if the hydraulic grade line is maintained below a certain limit. Typically, this is a specified distance above the crown of the pipe

[4] The Reynolds number is a dimensionless parameter that is a ratio of inertial forces to viscous forces. Relative roughness is the ratio of element roughness size to conduit size. Consult basic hydraulic texts for further discussion.



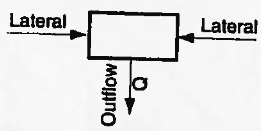
$$H_{tm} = \frac{V^2}{2g} \qquad \textbf{(a)}$$

Terminal Junction Losses
(at beginning of run)

Where g = gravitational constant, 32.2 feet per second per second

$$H_e = 0.5\frac{V^2}{2g} \qquad \textbf{(b)}$$

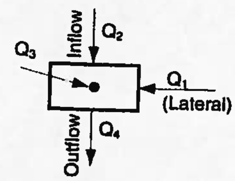
Entrance Losses
(for structure at end of run)
Assuming square-edge



$$H_{jl} = \frac{V^2}{2g}\text{(Outflow)} \qquad \textbf{(c)}$$

Junction Losses

Use only where flows are identical to above, otherwise use H_{j2} Equation.

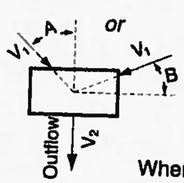


$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4} \qquad \textbf{(d)}$$

Junction Losses
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90° (H_b).

Where K = Bend loss factor
Q_3 = Vertical dropped-in flow from an inlet
V_3 = Assumed to be zero



Where B = 90 - A

$$H_b = \frac{KV_1^2}{2g} \qquad \textbf{(e)}$$

Bend Losses
(changes in direction of flow)

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

Friction Loss (H_f)

$$H_f = S_f \times L \qquad \textbf{(f)}$$

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

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

Where Q = discharge of conduit
n = Mannings coefficient of roughness (use 0.013 for R.C. pipes)
A = area of conduit
R = hydraulic radius of conduit ($^D/_4$ for round pipe)

Total Energy Loss at Each Junction

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

FIGURE 21.17 Summary of energy losses through structures.

PROJECT: __________ DESIGNED BY: __________ CHECKED BY: __________

DESIGN STORM: __________ DATE: __________ DATE: __________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
				Friction Loss in Pipe						Terminal Structure Losses		Junction Losses									
Struct.	D/S	h_v	D/S	D	Q	V	L	S_f	H_f	H_{ex}	H_{entr}	H_0	H_1	H_b	H_T	1.3 H_T	0.5 H_T	U/S	h_v in	U/S	Rim
No.	EGL	out	HGL	in	cfs	fps	ft	ft/ft	ft	ft	ft	ft	ft	ft	ft	ft	ft	EGL	ft	HGL	Elev.

Column
1 Structure identification label
2 EGL elevation at downstream side of structure = preceding column 19 + preceding column 10
3 Velocity head at the discharge side of the structure = $V^2/2g$
4 HGL elevation = column 2 − column 3
5 Pipe diameter
6 Discharge
7 Discharge velocity
8 Pipe length between structures
9 Friction slope = $\left(\dfrac{Vn}{1.49\,R^{2/3}}\right)^2$
10 Friction head loss = column 8 × column 9 (fig. 14.20-f)
11 Exit head loss at terminal structure on D/S end of run (fig. 14.20-a)
12 Entrance head loss at terminal structure on U/S end of run (fig. 14.20-b)

Column
13 Contraction head loss for outflow from a nonterminal structure = $k_{ex}(V^2_{out}/2g)$
14 Expansion head loss for inflow into nonterminal manhole = $k_{entr}(V_{in}^2/2g)$
15 Bend head loss due to deflection of flow through manhole (fig. 14.20-e)
16 Total head loss through the junction = column 13 + column 14 + column 15
17 Head loss for contributing flow from surface inlet = column 16 × 1.3 if surface discharge is greater than 10% of the mainline flow
18 Reduction in head loss of column 16 for inlet shaping = column 16 or column 17 × 0.5
19 EGL on the upstream side of the manhole structure = column 2 + column 11 or column 16, 17, or 18
20 Velocity head of the incoming pipe = $V_{in}^2/2g$
21 HGL on the upstream side of the manhole structure = column 19 − column 20
22 Elevation of top of manhole or inlet structure

FIGURE 21.18 Hydraulic grade line computation form.

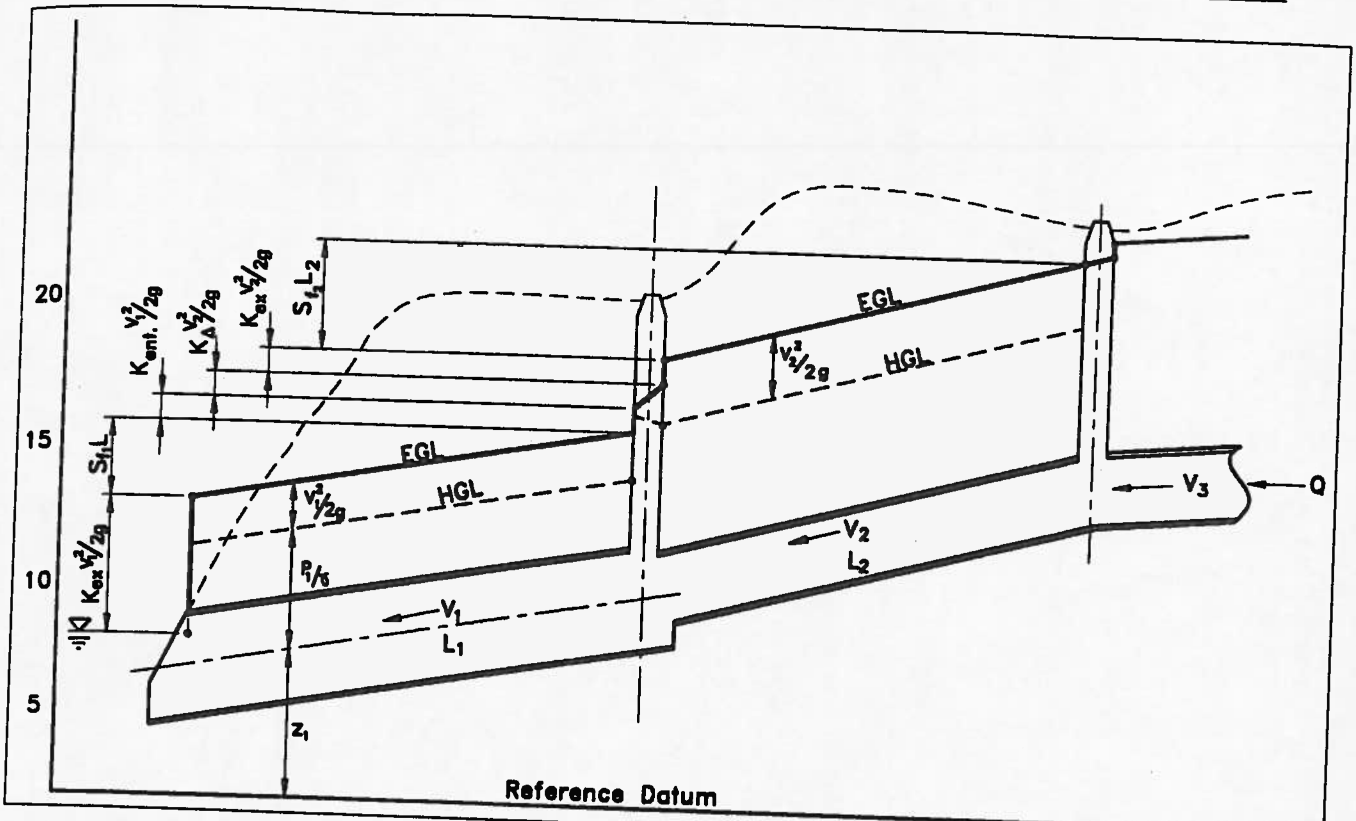


FIGURE 21.19 Schematic diagram for energy grade line and hydraulic grade line.

and below the ground surface elevation (e.g., the inlet inflow elevation or rim elevation). Some localities have developed criteria requiring the storm sewer system to be evaluated for specified storms larger than the design storm, such as in low-lying areas where there is little to no overland relief should runoff surcharge out of the system.

Hydraulic grade line calculations are easily summarized with the aid of an HGL calculation form as shown in Figure 21.18. The hydraulic grade line is parallel to and below the energy grade line along the length of conduit a distance equal to the velocity head $V^2/2g$. Energy losses through manholes and other appurtenances are graphically shown as abrupt jumps in the EGL. Figure 21.19 is a schematic representation of the HGL calculation form.

Computations for the HGL begin at a position where the water surface elevation is known or can be calculated. A known water surface elevation is where the HGL elevation can easily be determined, such as with the end of a pipe discharging into a lake. In this case, assuming the velocity head of the water at the lake's surface near the discharge end of the pipe is negligible, the starting EGL elevation is equal to the water surface elevation of the lake as well as the HGL of the lake. Should the end of the pipe discharge into a channel, the beginning water surface elevation can be found from backwater computations in the stream channel.

Once you have established the starting point, the computations proceed in the upstream direction by adding the energy losses attributed to exit, entrance, and junction losses to the EGL.

If storm sewer systems are assumed to operate with velocities in the 3–6-fps range, the HGL is approximated by the EGL. For normal velocities in the storm drain system, the difference between the EGL and the HGL elevations is less than 0.5 feet (i.e., for $V = 5$ fps, $V^2/2g = 0.4$ ft). Although the HGL is actually lower than the EGL by this amount, a relatively small velocity head is considered negligible, and many localities assume that the EGL approximates the HGL. A common design practice to anticipate and compensate for these velocity heads is to match crowns of the outgoing and incoming pipes of the manhole when the outgoing pipe is larger. If the two pipes are of equal size, common design practice would drop the outgoing pipe approximately 0.25 to 0.50 feet to account for losses that occur within the structure.

However, recognize that the occasion occurs, perhaps rarely, where such approximations can be substantially different. One case occurs when the discharge velocity in the incoming pipe is significantly greater than the velocity of the outgoing pipe in a manhole. This is illustrated at structure number 3 in the following example.

EXAMPLE 3

Determine the HGL for the data given in Table 21.5.

All pipe is reinforced concrete pipe (RCP), $n = 0.015$. A plan/profile view of the pipe system is shown in Figure 21.20.

In this example, the water surface is known at the downstream side of structure 1. The head loss through

TABLE 21.5 Data for HGL/EGL Example 3

From	To	Diameter (in)	Length (ft)	Slope (ft/ft)	Discharge (cfs)
4	3	15	200	0.045	15
3	4	21	400	0.010	15
2	1	24	300	0.0075	21

the structure is computed to find the EGL on the upstream side. The head loss through the section of pipe is added to this to find the EGL on the downstream side of the next upstream structure. The process is repeated until the HGL falls below the crown of the pipe or until the end of the run. Detailed computations follow with reference to the HGL computation form shown in Table 21.6. The form is utilized best if each horizontal line is computed sequentially.

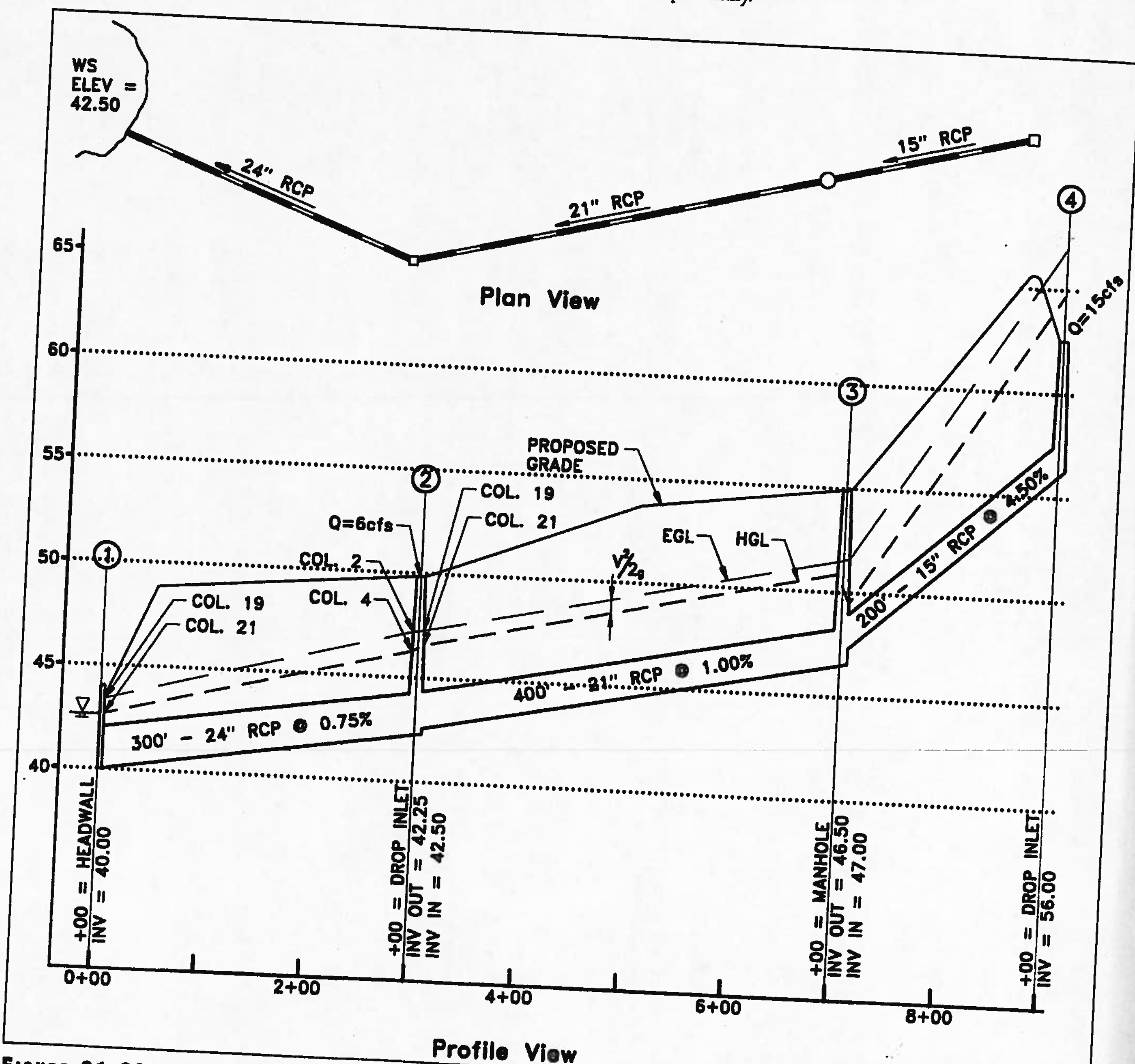


FIGURE 21.20 Plan and profile view of pipe system for Example 3.

TABLE 21.6 Hydraulic Grade Line Computation Form

PROJECT: EXAMPLE PROBLEM   DESIGNED BY: ________   CHECKED BY: ________

DESIGN STORM: ________   DATE: ________   DATE: ________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
				FRICTION LOSS IN PIPE						TERMINAL STRUCTURE LOSSES		JUNCTION LOSSES									
STRUCTURE No.	D/S EGL	H_v OUT	D/S HGL	D (IN.)	Q (CFS)	V (FPS)	L (FT)	S_f (FT/FT)	H_f (FT)	H_{ex} (FT)	H_{ent} (FT)	H_o (FT)	H_i (FT)	H_b (FT)	H_T (FT)	1.3 H_T (FT)	0.5 H_T (FT)	U/S EGL	h_v IN (FT)	U/S HGL	RIM ELEV.
1	42.50	0	42.50							0.69								43.19	0.69	42.50	
				24	21	6.69	300	0.011	3.30												
2	46.49	0.69	45.80									0.17	0.21	0.28	0.66	0.86	0.43	46.92	0.60	46.32	49.50
				21	15	6.24	400	0.012	4.80												
3	51.72	0.60	51.12									0.15	0.81		0.96		0.48	52.20	2.31	49.89	54.50
				15	15	12.2	200	0.072	14.3												
4	66.50	2.31	64.19								1.16						67.66		67.66	62.00	

Column
1 Structure identification label
2 EGL elevation at downstream side of structure = preceding column 19 + preceding column 10
3 Velocity head at the discharge side of the structure = $V^2/2g$
4 HGL elevation = column 2 − column 3
5 Pipe diameter
6 Discharge
7 Discharge velocity
8 Pipe length between structures
9 Friction slope = $(Vn/1.49\ R^{2/3})^2$
10 Friction head loss = column 8 × column 9 (Figure 21.17*f*)
11 Exit head loss at terminal structure on D/S end of run (Figure 21.17*a*)
12 Entrance head loss at terminal structure on U/S end of run (Figure 21.17*b*)
13 Contraction head loss for outflow from a nonterminal structure = $k_{ex}(V_{out}^2/2g)$
14 Expansion head loss for inflow into nonterminal manhole = $k_{ent}(V_{in}^2/2g)$
15 Bend head loss due to deflection of flow through manhole (Figure 21.17*e*)
16 Total head loss through the junction = column 13 + column 14 + column 15
17 Head loss for contributing flow from surface inlet = column 16 × 1.3 if surface discharge is greater than 10 percent of the mainline flow
18 Reduction in head loss of column 16 for inlet shaping = column 16 or column 17 × 0.5
19 EGL on the upstream side of the manhole structure = column 2 + column 11 or column 16, 17, or 18
20 Velocity head of the incoming pipe = $V_{in}^2/2g$
21 HGL on the upstream side of the manhole structure = column 19 − column 20
22 Elevation of top of manhole or inlet structure

Structure 1

Columns 2, 3, and 4. Structure 1 is a headwall with pipe discharging into a reservoir at water surface (WS) elevation = 42.5. The velocity of the water surface of the reservoir is assumed to be zero if we ignore the disturbance in the immediate vicinity of the headwall caused by the momentum of the incoming flow. The EGL at the downstream side of the headwall is:

$$\mathrm{EGL} = \frac{p}{\gamma} + \frac{V^2}{2g} + z \tag{21.31}$$

If, as assumed, $V = 0$, and we use gauge pressure as the reference, then $p/\gamma = 0$ and the EGL is equal to the water surface elevation z. Because $V = 0$, the HGL is also the water surface elevation (i.e., HGL = EGL − $V^2/2g$).

Column 11. Assume a loss coefficient k for a square-edged headwall = 1.0. The head loss through the headwall is:

$$H_{ex} = (k_{ex})\frac{V^2}{2g} = 1\left(\frac{6.69^2}{2g}\right) = 0.69\ \mathrm{ft} \tag{21.32}$$

where V is the average discharge velocity in the pipe ($V = Q/A$).

Column 19. On the upstream side of the headwall, the EGL is equal to the EGL at the downstream side plus the head loss through the structure.

$$(U/S\ \mathrm{EGL})_1 = 42.50 + 0.69 = 43.19\ \mathrm{ft} \tag{21.33}$$

Columns 20 and 21. The HGL at the upstream side of the headwall is equal to the EGL minus the velocity head in the pipe.

$$(U/S\ \mathrm{HGL})_1 = 43.19 - \frac{6.69^2}{2g} = 42.50\ \mathrm{ft} \tag{21.34}$$

Columns 6 and 7. From Manning's equation, the maximum discharge through the 24-inch pipe is:

$$Q_{max} = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{0.67}(0.0075)^{0.5}\left(\frac{2^2\pi}{4}\right) = 17.0\ \mathrm{cfs} \tag{21.35}$$

The pipe is carrying 21 cfs and is therefore under pressure, that is, the HGL is above the crown of the pipe. The velocity in the pipe is:

$$V = \frac{Q}{A} = \frac{21}{\frac{\pi 2^2}{4}} = 6.69\ \mathrm{fps} \tag{21.36}$$

Columns 9 and 10. The friction slope of the water is:

$$S_f = \left(\frac{(6.69)(0.015)}{(1.49)\left(\frac{2}{4}\right)^{0.67}}\right)^2 = 0.011\ \mathrm{ft/ft} \tag{21.37}$$

The total head loss in the 24-inch pipe is:

$$H_f = (0.011)(300) = 3.30\ \mathrm{ft} \tag{21.38}$$

Structure 2

Columns 2, 3, and 4. The EGL on the downstream side of structure 2 is equal to the EGL on the upstream side of structure 1 plus the head loss in the pipe between structures 1 and 2.

$$(D/S\ \mathrm{EGL})_2 = 43.19 + 3.30 = 46.49\ \mathrm{ft} \tag{21.39}$$

The HGL on the downstream side of structure 2 is one velocity head below the EGL.

$$(D/S\ \mathrm{HGL})_2 = 46.49 - \frac{6.69^2}{2g} = 45.80\ \mathrm{ft} \tag{21.40}$$

Columns 13, 14, 15, and 16. The head loss through the structure is the combined effects from sudden contraction, sudden expansion, and momentum loss as the flow is deflected through the 45°. Proceeding in the upstream direction, the head loss due to sudden contraction as the flow leaves structure 2 and enters the pipe is:

$$H_o = k_o\left(\frac{V^2}{2g}\right) = 0.25\left(\frac{6.69^2}{2g}\right) = 0.17\ \mathrm{ft} \tag{21.41}$$

The head loss for sudden expansion for incoming flow into structure 2 is:

$$H_i = 0.35\left(\frac{6.24^2}{2g}\right) = 0.21\ \mathrm{ft} \tag{21.42}$$

The bend loss coefficient found in Figure 21.17 is 0.47. Using the incoming velocity,

$$H_b = 0.47\left(\frac{6.24^2}{2g}\right) = 0.28\ \mathrm{ft} \tag{21.43}$$

The total head loss through the structure $H_T = 0.17 + 0.21 + 0.28 = 0.66$ ft. The head loss coefficients for sudden expansion and sudden contraction vary according to the standard design criteria of the local jurisdiction.

Column 17. Surface water entering through the top of the structure disrupts the flow in the mainline and thus creates head loss. The recommendation by AASHTO is to increase the total head loss through the structure by 30 percent to account for this additional head loss. This adjustment is applicable only for surface water entering the structure in the amount equal to or greater than 10 to 20 percent of the flow in the mainline of the pipe, based on local criteria. In this case, 6 cfs ≧ 0.1(15 cfs) and the adjustment is made; 1.3 (0.66) = 0.86 ft.

Column 18. Efficiency of flow through the manhole can be improved by constructing a trough through the bottom of the structure to direct the flow. This is known as *inlet shaping*. Most design standards for storm sewer manholes incorporate inlet shaping. If the storm sewer structure incorporates inlet shaping, the total head loss through the structure, in this case, is reduced 50 percent (this value varies according to local criteria). Column 16 (or column 17 for surface inlet structures) is decreased. For this structure, column 18 = 0.5(0.86) = 0.43 ft. However, hydraulic experts have questioned the effectiveness of inlet shaping for highly turbulent excessive surcharge flows.

Structure 3. Computations for structure 3 are similar to those for structures 1 and 2. Since structure 3 is a manhole-type structure and no surface water enters through the top, column 17 does not apply. Note that the EGL across structure 3 increases while the HGL decreases, due to the high incoming discharge velocity. This is one case where the HGL cannot be approximated to the EGL.

Structure 4. Structure 4 is a terminal structure and therefore only column 12 applies for computing the head loss. Notice the HGL elevation is above the rim elevation of the structure. This shows that for the given discharges and pipe design, the top of the structure is inundated by 3 feet of water. If the design discharges are for a frequent storm (i.e., recurrence interval ≤ 10 years) this pipe design, in all likelihood, would be unacceptable.

HGL Summary. HGL computations are vital to the proper design and construction of storm sewer systems. Keep in mind that the method described in the previous sample problem is typical; however, many localities have additional or different techniques. Some localities allow head losses at junctions to be reduced by inlet shaping, as suggested by column 18 in the HGL spreadsheet, while others do not. Some state departments of transportation also take conservative approaches to bend losses and junction losses, so always consult the local design criteria if available.

Another conservative approach practiced by many agencies is to assume that the minimum depth of the HGL along a pipe at the junction location (going upstream) can be no lower than a certain depth in the pipe, such as normal depth of $(D_c + D)/2$. For example, if calculated junction losses show that the elevation of the HGL is near the bottom of the upstream pipe, the user would start the HGL for the upstream pipe at $(D_c + D)/2$, not the calculated HGL value.

PROCEDURE FOR STORM SEWER DESIGN

Storm sewer is generally a gravity-based system (rather than pressurized): successful design is contingent upon thorough understanding of the outfall conditions, the contributing watershed characteristics (size, topography, land use, soils), and hydrology (discussed in Chapter 19). One of the more frustrating predicaments encountered by designers, after painstakingly designing the system, is to find that the design pipe invert elevation at the outfall point is too low. This can be avoided if preliminary pipe sizes are set based on rough estimates of the discharge and the first iteration of design is only a rough estimate. The general procedure for designing the storm sewer system is summarized as follows.

Data Collection

Obtain a plan of the project area showing existing and proposed features along with the existing topography and proposed grading. Profiles of roads, water and sanitary lines, invert elevations of the outfall points, topographic maps showing areas contributing off-site runoff and discharges from pipes, and typical sections of stream channels are all necessary information when designing the storm sewer system. This information should be assembled as part of the preliminary engineering efforts.

Evaluate Sump Inlet Locations

Inlets are set at all sump areas in streets, parking areas, and in nonpaved areas. Check all curb returns for low points. Using topographic information, establish the drainage areas to each inlet and compute the runoff. Include runoff and discharge from any areas outside of the project area. Use the appropriate capacity charts for the selected inlet to determine the ponding depth and/or spread at the inlet.

Supplementary Sump (Flanking) Inlets

If the inlets at the sump areas are inadequate either increase the size of the inlet or add another inlet upstream from the sump. If dual flanking inlets are not required such as recommended by FHWA, consider whether only one flanking inlet is necessary to reduce the ponding and spread in the sump. Similarly, if the drainage area on one side of the sump contributes most of the runoff, then consider only one flanking inlet. In residential areas, inlets should be located where they will not interfere with driveway entrances. In many cases final house and driveway locations are not known during the design of the storm sewer system. Usually, houses are centered on the lot and the driveway favors one side of the house. If the length of the flanking inlet is short, its location can be either the center of the lot or at the lot line extended. For longer-length inlets, the engineer must exercise judgment in anticipating the driveway location to locate the inlet. Inlets are considered in residential yards when the discharge velocity in the swale approaches erosive velocities or if flow depths are excessive. Few residents use the yard during a rainstorm; however, in poorly drained soils ponding and residual sogginess are a nuisance.

Other Inlet Locations

Other critical areas that warrant consideration are street intersections and curb returns. Many localities limit the amount of discharge that may cross through the entrance of an intersecting street. The amount of discharge may vary according to the functional designation of the street. Too much discharge flowing around a curb return causes the water to spread out as it moves around the curb. In lieu of any local restrictions, the engineer should consider adding inlets at curb returns to reduce the flow across the intersection when flows approach 3 to 4 cfs on lower-category streets and when flows are greater than 2 cfs on higher-category streets.

Placement of other inlets is then dictated by the spread limitations identified by design standards. Studies have shown that the efficiency of an inlet is increased if it is sized such that 5% to 10% of the flow is allowed to bypass. Therefore, consideration should be given for sizing the throat lengths so that they are about 90 percent efficient. In the

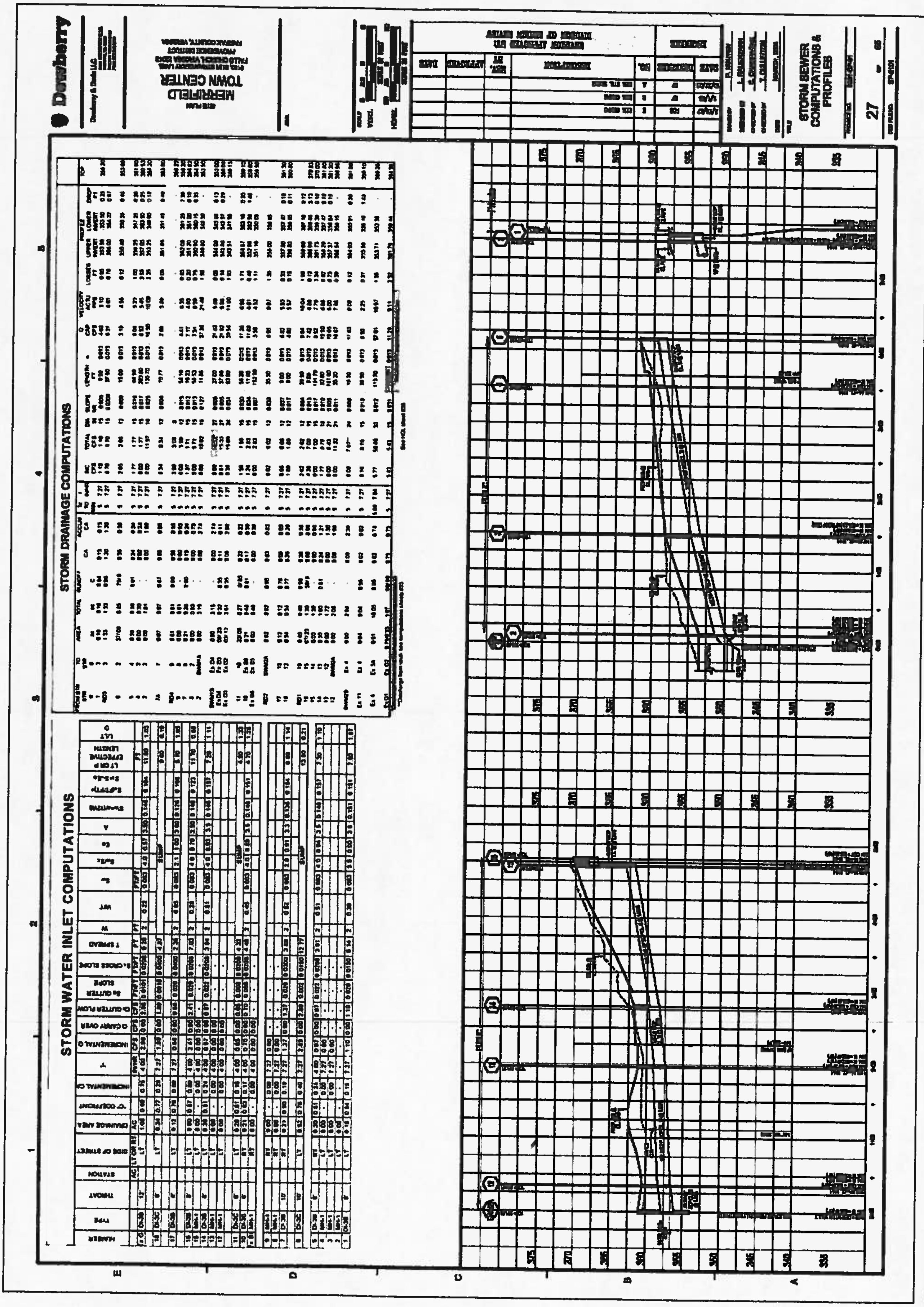


FIGURE 21.21 Storm drain computations and profile.

design, the bypass flow must be included in the runoff discharge for the next downstream inlet.

Pipe Layout

The inlets are then connected to show how the pipes convey the water to the outfall points. The pipe network usually directs the water in the prevailing direction of the slope of the proposed grading. However, there may be some instances where judgment supersedes rule-of-thumb guidelines. Pipe runs of excessive length (e.g., greater than the maximum allowable manhole spacing) require intermittent manholes. Additionally, any changes in horizontal direction or vertical slope of the pipe, (e.g., to go around a foundation or to minimize easements on lots) also require additional manholes. When additional manholes are necessary, the engineer should check whether additional inlets would be beneficial and substitute accordingly. This is not always possible or desirable, but the cost differential between the two is not significant.

Profile the System

Profile this tentative layout on a profile worksheet. Show the existing and proposed grades and mark the locations of the proposed inlets and manholes, as well as the existing (and proposed if available) utility crossings including sanitary sewer, water main, and duct bank crossings. The purpose of this profile is to ensure the pipes have sufficient cover to prevent excessive excavation and to prevent utility crossing conflicts. Utility conflicts need to be identified during the first phase of design so that they do not cause a major redesign of the system in a later design phase. A more accurate profile can be drawn later after the pipe design is complete. This step is highly recommended. Although some construction plans or documents do not require storm sewer profiles, the simple task of putting the design on paper can eliminate countless redesigns of a system and avoid conflicts.

Tentative Pipe Sizes

Estimate the size of each pipe by adding the flows from each inlet above the inlet at the design point. *Recognize that this is not the proper way to calculate flows, as pointed out in the section discussing the rational method.* However, this method, although it overestimates the flow, is adequate to tentatively estimate the pipe size. For small to moderate-sized sites (less than 50 acres) and pipe runs of 800 feet and less, the relative error in the discharge using this method versus the correct method is tolerable for tentatively estimating pipe sizes. Using a hydraulic calculator, or other means, select the smallest pipe size, which conveys the estimated discharge at full flow, at a slope that approximates the ground slope above the pipe. See Chapter 19 for rational method and closed pipe system computation. As the pipes are selected, write this information on the profile worksheet between the appropriate inlets. Draw a line between inlets that represents the invert of the pipe. The crown of the outgoing pipe is set at the same elevation, or higher, than the crown elevation of the lowest incoming pipe. When large junction losses are anticipated, the crown of the incoming pipe should be slightly higher. This increases the difference in elevation in inverts to allow for the large junction losses. Note that these last three statements affect where the invert line of the pipe is drawn.

Final Assessment

Check to see that the invert of the last pipe is above the invert of the outfall point. Confirm all existing and proposed utility crossings. If less-than-minimum clearance is provided where the storm sewer crosses a utility, concrete piers or cradles should be considered to support the storm sewer pipe at the utility crossing. If there are no utility conflicts, proper connections to outfall points are possible, and the engineer is comfortable with the cost effectiveness of the design, perform a more detailed design. That is, correctly compute the discharge through the pipes, fill out the pipe design and hydraulic grade line charts, and accurately draw the profiles. (See Figure 21.21.)

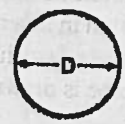
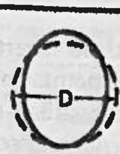
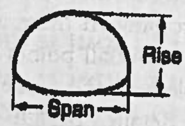
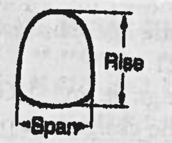
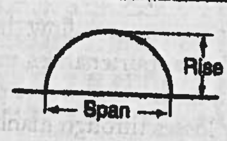
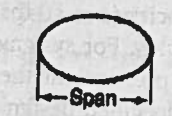
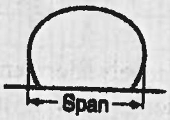
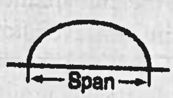
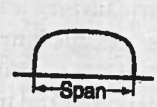
Hydraulic grade line (HGL) computations should be the last step in the design. See prior HGL discussion, this chapter. Although the pipe slopes, if properly designed and set for gravity (nonpressure) flow, head losses through structures and other appurtenances may cause pressure flow in the system. In most cases, where pipe velocities are less than 5 fps, energy losses through manholes are less than 0.5 feet; however, in systems with numerous manholes or in those pipes where velocities are 10 fps and higher, the head losses accrue very quickly. For systems with these characteristics, the potential for pressurizing the system becomes very likely. Even though a specified design storm is used to design the system, local criteria may require that the HGL be checked for other storms. Considerable engineering judgment is required throughout the storm drain design process in assessing the applicable regulations, determining and analyzing a myriad of solutions, and ultimately reviewing the final design to ensure public safety and welfare.

CULVERTS

A culvert is a relatively short length of conduit, typically less than 250 feet long, used to transport water through (or under) an embankment. A culvert, which acts as an enclosed channel through the embankment, serves as a continuation of the open stream. However, flow through culverts depends on entrance geometry and depth of flow at the downstream end. Consequently, flow computations for culverts are more complex than the open channel flow analysis associated with pipes and ditches. Culverts through roadway and railway embankments are designed to pass the design discharge without overtopping the embankment or causing extensive ponding or inundation at the upstream end. Local requirements may allow nominal depths over the embankments for lesser-frequency storm (greater recurrence interval) events.

Components

Major components of a culvert design include specifying the materials—the barrel; end treatments such as headwalls,

Shape		Range of Sizes	Common Uses
Round		6" – 26'	Culverts, subdrains, sewers, service tunnels, etc. All plates same radius. For medium and high fills (or trenches).
Vertically-elongated (ellipse) 5% is common		4' – 21' nominal; before elongating	Culverts, sewers, service tunnels, recovery tunnels. Plates of varying radii; shop fabrication. For appearance and where backfill compaction is only moderate.
Pipe-arch		Span x Rise 18" x 11" to 20' 7" x 13' 2"	Where headroom is limited. Has hydraulic advantages at low flows. Corner plate radius, 18 inches or 31 inches for structural plate.
Underpass*		Span x Rise 5' 8" x 5' 9" to 20' 4" x 17' 9"	For pedestrians, livestock or vehicles (structural plate).
Arch		Span x Rise 6' x 1' 9½" to 25' x 12' 6"	For low clearance large waterway opening, and aesthetics (structural plate).
Horizontal Ellipse		Span 20' – 40'	Culverts, grade separations, storm sewers, tunnels.
Pear		Span 25' – 30'	Grade separations, culverts, storm sewers, tunnels.
High Profile Arch		Span 20' – 45'	Culverts, grade separations, storm sewers, tunnels, Ammo ammunition magazines, earth covered storage.
Low Profile Arch		Span 20' – 50'	Low-Wide waterway enclosures, culverts, storm sewers.
Box Culverts		Span 3' – 20'	Low-Wide waterway enclosures, culverts, storm sewers.
Specials		Various	For lining old structures or other special purposes. Special fabrication.

* For equal area or clearance, the round shape is generally more economical and simpler to assemble.

FIGURE 21.22 Shapes and uses of corrugated conduits.

endwalls, and wingwalls; outlet protection; and inlet improvements such as debris control structures—as well as determining the environmental permitting requirements. Except for the barrel, these components are used as the specific situation warrants.

Barrels are available in various sizes, shapes, and materials. Figure 21.22 shows the commonly used culvert shapes as well as applications of the various shapes. Selection of shape depends on construction limitations, embankment height, environmental issues, hydraulic performance, and cost. The most commonly used culvert materials are corrugated steel, corrugated aluminum, and precast or cast-in-place concrete. Factors such as corrosion, abrasion, and structural strength determine the selection of material. In cases where the culvert is located in a highly visible area, the selection of shape and material may be based on aesthetics as well as the functional aspects.

End treatments, such as headwalls and wingwalls, protect the embankment from erosion, serve as retaining walls to stabilize the bank, and add weight to counter any buoyancy effects. Ideally, the centerline of the culvert should follow the alignment and grade of the natural channel. In many cases this cannot be done, and skewing headwalls and wingwalls helps transition the natural stream alignment to the culvert alignment. Figure 21.23 shows four types of inlet entrances.

Debris barriers are sometimes constructed on the upstream end to prevent material from entering and clogging the culvert. The barriers are placed far enough away from the entrance so that accumulated debris does not clog the entrance.

At the inlet and outlet ends of the culvert, endwalls and wingwalls serve as retaining walls and erosion protection for the embankment and help to inhibit piping along the outside surface of the culvert. Downstream wingwalls provide a smooth transition between the culvert and the natural stream banks.

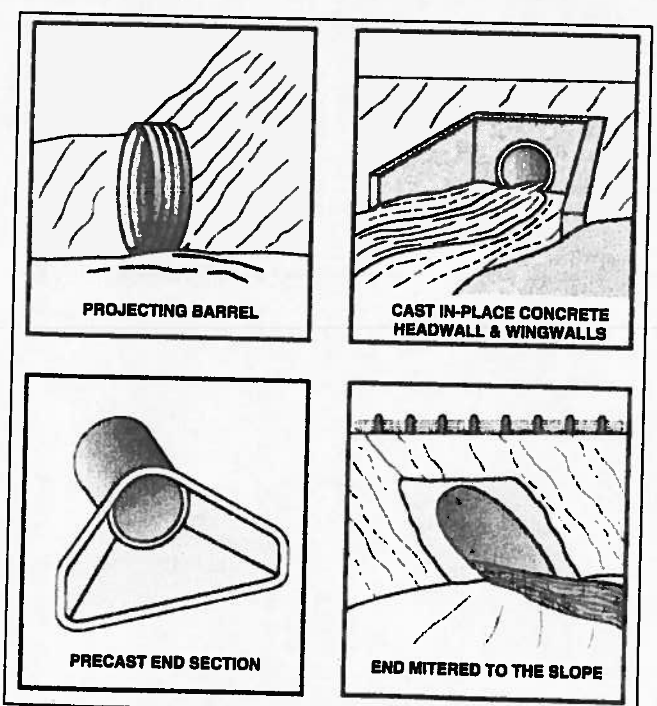


Figure 21.23 Four standard end inlet treatments.

All culverts that cross underneath roadway embankments inherently create potential stability concerns for the embankments. The same can be said of storm sewer pipe installations. This is a function of the construction of the culvert system. Compaction around a pipe, whether it is circular or rectangular, is more difficult than standard compaction of fill slopes. The contractor must take care in compacting around culvert systems, usually requiring several "lifts" (e.g., 6- to 12-inch layers of soil) to compact the surrounding fill material to specification. Consideration should always be given, by the designer and the contractor, that storm runoff will seep into roadway embankments or ground water tables will rise during flood events. If the compaction is inadequate, water can run along the voids in the soil of the uncompacted portion of the culvert. When water continues to flow along a pipe or culvert, the uncompacted material slowly washes away from the surrounding pipe, and failure of the embankment may occur. This is one of the most common causes of embankment failures at culvert installations and roadway pavement failures at storm sewer installations. This type of failure is referred to as *piping failure.*

Culverts and storm sewers are typically constructed with a stone base to provide support and flexibility. For most installations, this stone base is porous and allows any water inside the embankment to travel downstream, along the path of the culvert. Typically, this aggregate base aids in reducing piping failures, in that it concentrates the water flowing through the embankment. However, should there be excessive ponding of water behind an embankment, the standard bedding provided at culvert installations will not be adequate to pass the seepage flow safely along the conduit, and piping failures may occur. The designer should consult with a geotechnical engineer to determine appropriate countermeasures, such as concrete cradles, use of impervious materials, or embankment draining devices such as toedrains to minimize the potential for piping failures. Permitting requirements and natural channels require the designer to countersink the culvert 3 to 12 inches, depending on the local, state, and federal laws. The depth of countersinking may be dictated by the type of ground material. Countersinking the culvert provides a natural channel bottom through the culvert for aquatic habitat. The culvert analysis should consider the loss of flow area in the culvert and model the effective opening.

Culvert Hydraulics

In open channel flow, either of three turbulent flow regimes exists: subcritical flow, supercritical flow, or critical flow. The discharge energy, relative to the channel bed, is referred to as the *specific energy* and is mathematically defined as:

$$E = y + \frac{V^2}{2g} \qquad (21.44)$$

where y is the depth of flow, V is the average cross-sectional velocity, and E is the specific energy.

For one particular combination of depth and velocity, for a specified discharge, the specific energy is a minimum. This particular discharge is the critical flow. Alternatively, critical flow is the condition of flow when, for a given energy content of the water, the discharge is a maximum. For the same discharge, flows above this minimum energy may exist as either high depth–low velocity or low depth–high velocity. In the former case, flow is subcritical; the latter is supercritical flow. The parameter that distinguishes flow regimes, the *Froude number*, is a dimensionless number. The Froude number, as seen in Equation 21.45, is the ratio of inertial forces to gravitational forces of the flow:

$$F_R = \frac{V}{\sqrt{gd_h}} \tag{21.45}$$

In this equation, d_h is the hydraulic depth (the cross-sectional area of flow divided by the width of the channel at the water surface), V is the velocity, and g is gravitational acceleration. At critical flow $F_R = 1$, for subcritical flow $F_R < 1$, and at supercritical flow $F_R > 1$. The location in the channel section where flow changes from subcritical to supercritical flow—$F_R = 1$—is defined as a control section. At a control section, there exists a unique relation between depth and discharge, given as:

$$Q = \sqrt{g\frac{A^3}{B}} \tag{21.46}$$

where A is the cross-sectional area of flow and B is the width of the channel at the free surface of the water. Hydraulically, a control section restricts the transmission of the effect of changes in flow conditions, either in the upstream or downstream direction, depending on the state of flow.

The amount of water entering a culvert is determined by the location of the control section. If the control section is at the inlet (i.e., inlet control), the amount of flow into the culvert is restricted by entrance conditions and the flow into the culvert is less than what the culvert might actually carry. If the control section is at the downstream end (i.e., outlet control), the amount of flow through the barrel is controlled by the combination of conditions on the downstream side of the culvert, inlet configuration, and hydraulic properties of the barrel itself.

Inlet Control

When the control section is near the inlet, only the headwater depth and inlet configuration determine the amount of water entering the culvert. In most cases, the amount of water entering the culvert is less than what the barrel is capable of carrying. Consequently, the barrel is flowing less than full. Conceivably, for a given inlet configuration, the culvert

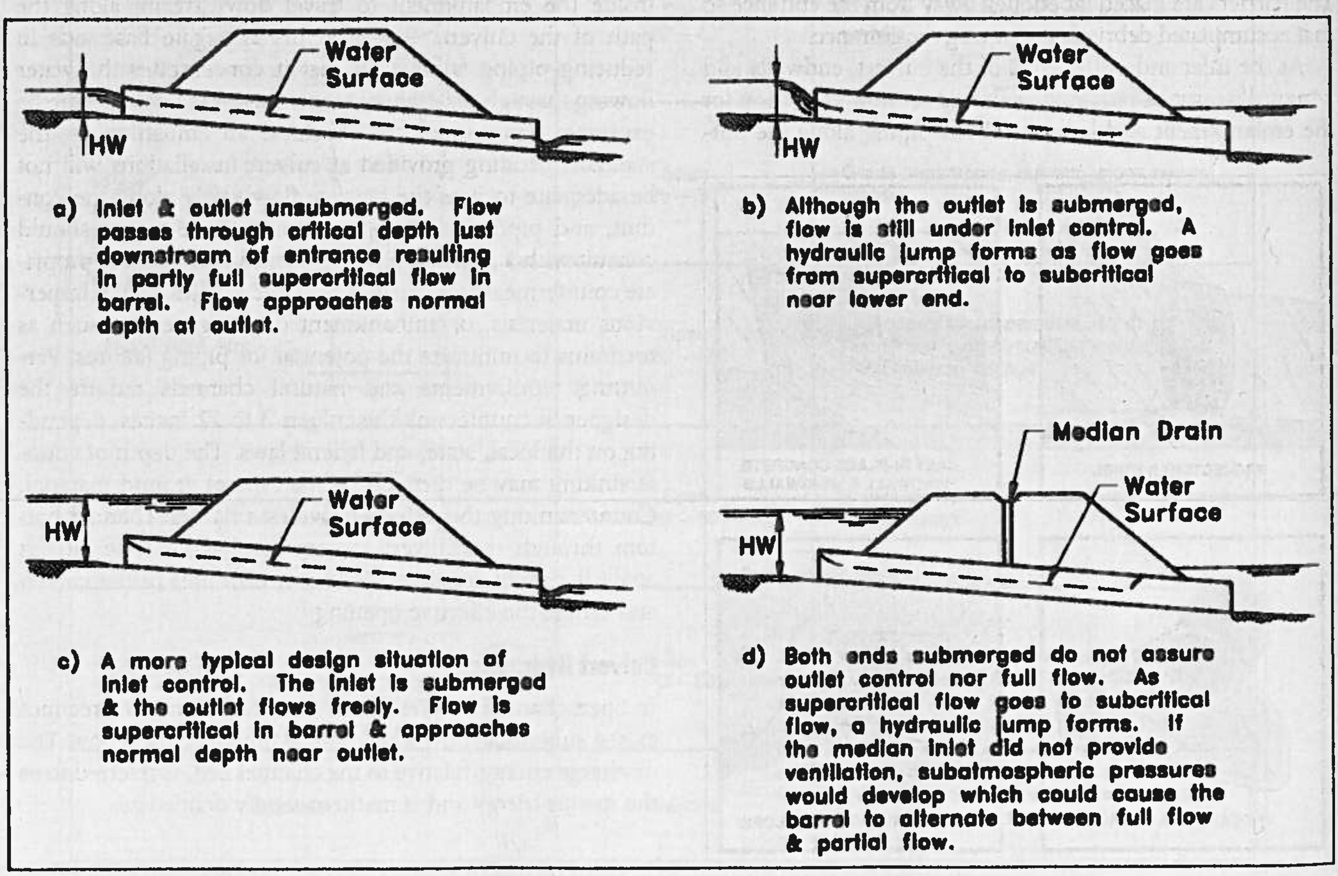


FIGURE 21.24 Types of inlet control.

TABLE 21.7 Constants for Inlet Control Design Equations

				Unsubmerged			Submerged		
Chart No.	Shape and Material	Monograph Scale	Inlet Edge Description	Equation Form	K	M	C	Y	Reference
1	Circular concrete	1	Square edge w/headwall	1	0.0098	2.0	0.0398	0.67	(56) (57)
		2	Groove end w/head wall		.0078	2.0	.0292	.74	(56) (57)
		3	Groove end projecting		.0045	2.0	.0317	.69	(56) (57)
2	Circular CMP	1	Headwall	1	.0078	2.0	0.379	.69	(56) (57)
		2	Mitered to slope		.0210	1.33	0.463	.75	(57)
		3	Projecting		.0340	1.50	0.553	.54	(57)
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	(57)
		B	Beleveled ring, 33.7° bevels		.0018	2.50	.0243	.83	(57)
8	Rectangular box	1	30–75% wingwall flares	—	.026	1.0	.0385	.81	(56)
		2	90° and 15° wingwall flares	1	.061	.75	.0400	.80	(56)
		3	0° wingwall flares		.061	.75	.0423	.82	(8)
9	Rectangular box	1	45° wingwall flare $d = .043D$	2	.510	.667	.0309	.80	(8)
		2	18°–33.7° wingwall flare $d = .083D$		.486	.667	.0249	.83	(8)
10	Rectangular box	1	90° headwall w/¾· chamfers	2	.515	.667	.0375	.79	(8)
		2	90° headwall w/45° bevels		.495	.667	.0314	.82	(8)
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865	(8)
11	Rectangular box	1	¾· chamfers; 45° skewed headwall	2	.522	.667	.0402	.73	(8)
		2	¾″ chamfers; 30° skewed headwall		.533	.667	.0425	.705	(8)
		3	¾″ chamfers; 15° skewed headwall		.545	.667	.04505	.68	(8)
		4	45° bevels; 10–45° skewed headwall		.498	.667	.0327	.75	(8)
12	Rectangular box ¾″ chamfers	1	45° nonoffset wingwall flares	2	.497	.667	.0339	.803	(8)
		2	18.4″ nonoffset wingwall flares		.493	.667	.0361	.806	(8)
		3	18.4″ nonoffset wingwall flares 30° skewed barrel		.495	.667	.0386	.71	(8)
13	Rectangular box Top bevels	1	45° wingwall flares, offset	2	.497	.667	.0302	.835	(8)
		2	33.7° wingwall flares, offset		.495	.667	.0252	.881	(8)
		3	18.4° wingwall flares, offset		.493	.667	.0227	.887	(8)
16–19	CM boxes	1	90° headwall	1	.0083	2.0	.0379	.69	(57)
		2	Thick wall projecting		.0145	1.75	.0419	.64	(57)
		3	Thin wall projecting		.0340	1.5	.0496	.57	(57)

TABLE 21.7 (Continued)

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Unsubmerged: Equation Form	Unsubmerged: K	Unsubmerged: M	Submerged: C	Submerged: Y	Reference
29	Horizontal	1	Square edge with headwall	1	0.0100	2.0	0.0398	0.67	(57)
	ellipse	2	Groove end with headwall		.0018	2.5	.0292	.74	(57)
	concrete	3	Groove end projecting		.0045	2.0	.0317	.69	(57)
30	Vertical	1	Square edge with headwall	1	.0100	2.0	.0398	.67	(57)
	ellipse	2	Groove end with headwall		.0018	2.5	.0292	.74	(57)
	concrete	3	Groove end projecting		.0095	2.0	.0317	.69	(57)
34	Pipe arch	1	90° headwall	1	.0083	2.0	.0496	.57	(57)
	18″ corner	2	Mitered to slope		.0300	1.0	.0463	.75	(57)
	Radius CM	3	Projecting		.0340	1.5	.0496	.53	(57)
35	Pipe arch	1	Projecting	1	.0296	1.5	.0487	.55	(56)
	18″ corner	2	No bevels		.0087	2.0	.0361	.66	(56)
	Radius CM	3	33.7° bevels		.0030	2.0	.0264	.75	(56)
36	Pipe arch	1	Projecting	1	.0296	1.5	.0487	.55	(56)
	31″ corner		No bevels		.0087	2.0	.0361	.66	(56)
	Radius CM		33.7° bevels		.0030	2.0	.0264	.75	(56)
40–42	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69	(57)
		2	Mitered to slope		.0300	2.0	.0463	.75	(57)
		3	Thin wall projecting		.0340	1.5	.0496	.57	(57)
54	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.89	(3)
		2	Rough tapered inlet throat		.519	.64	.0289	.90	(3)
55	Elliptical	1	Tapered inlet, beveled edges	2	.536	.622	.0368	.83	(3)
	Inlet face	2	Tapered inlet, square edges		.5035	.719	.0478	.80	(3)
		3	Tapered inlet, thin edge projecting		.547	.80	.0598	.75	(3)
56	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	(3)
57	Rectangular	1	Side tapered, less favorable edges	2	.56	.667	.0466	.85	(3)
	concrete	2	Side tapered, more favorable edges		.56	.667	.0378	.87	(3)
58	Rectangular	1	Slope tapered, less favorable edges	2	.50	.667	.0466	.65	(3)
	concrete		Slope tapered, more favorable edges		.50	.667	.0378	.71	(3)

Reference: FHWA, *Hydraulic Design of Highway Culverts*, HDS-5, 1985.

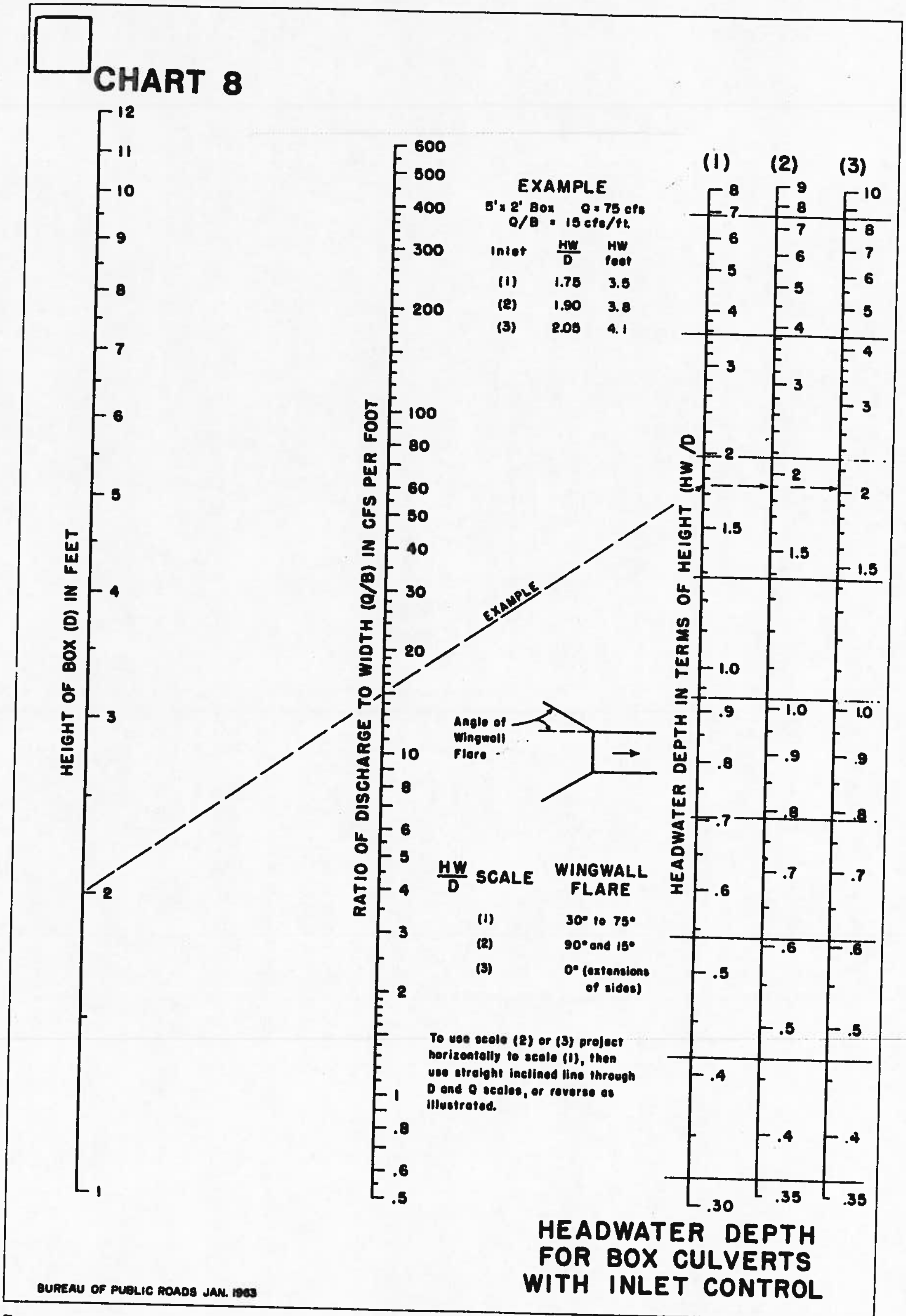


FIGURE 21.25 HDS-5 Chart 8, nomograph for headwater depth for box culvert with inlet control.

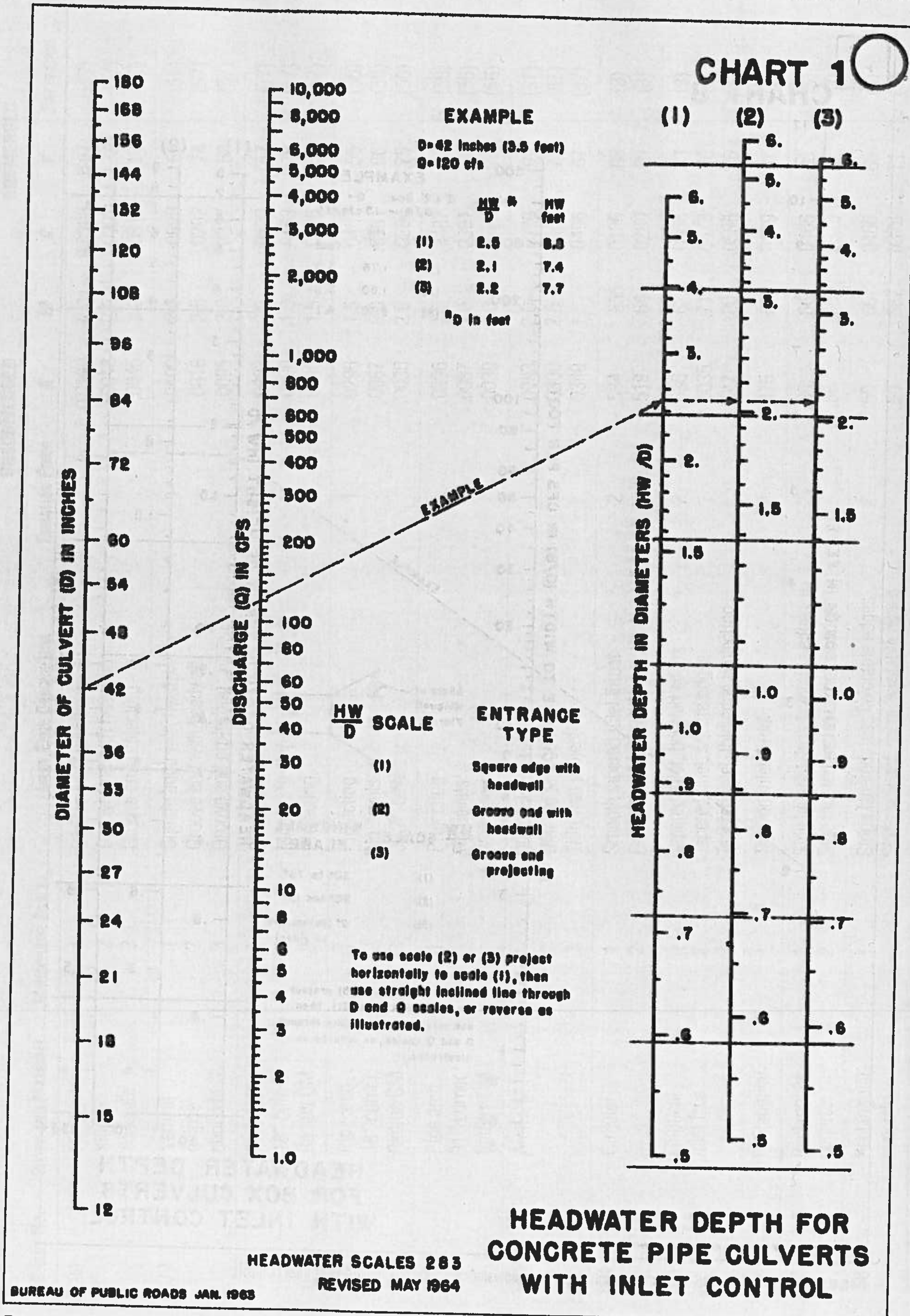


FIGURE 21.26 HDS-5 Chart 1, nomograph for headwater depth for concrete pipe culvert with inlet control.

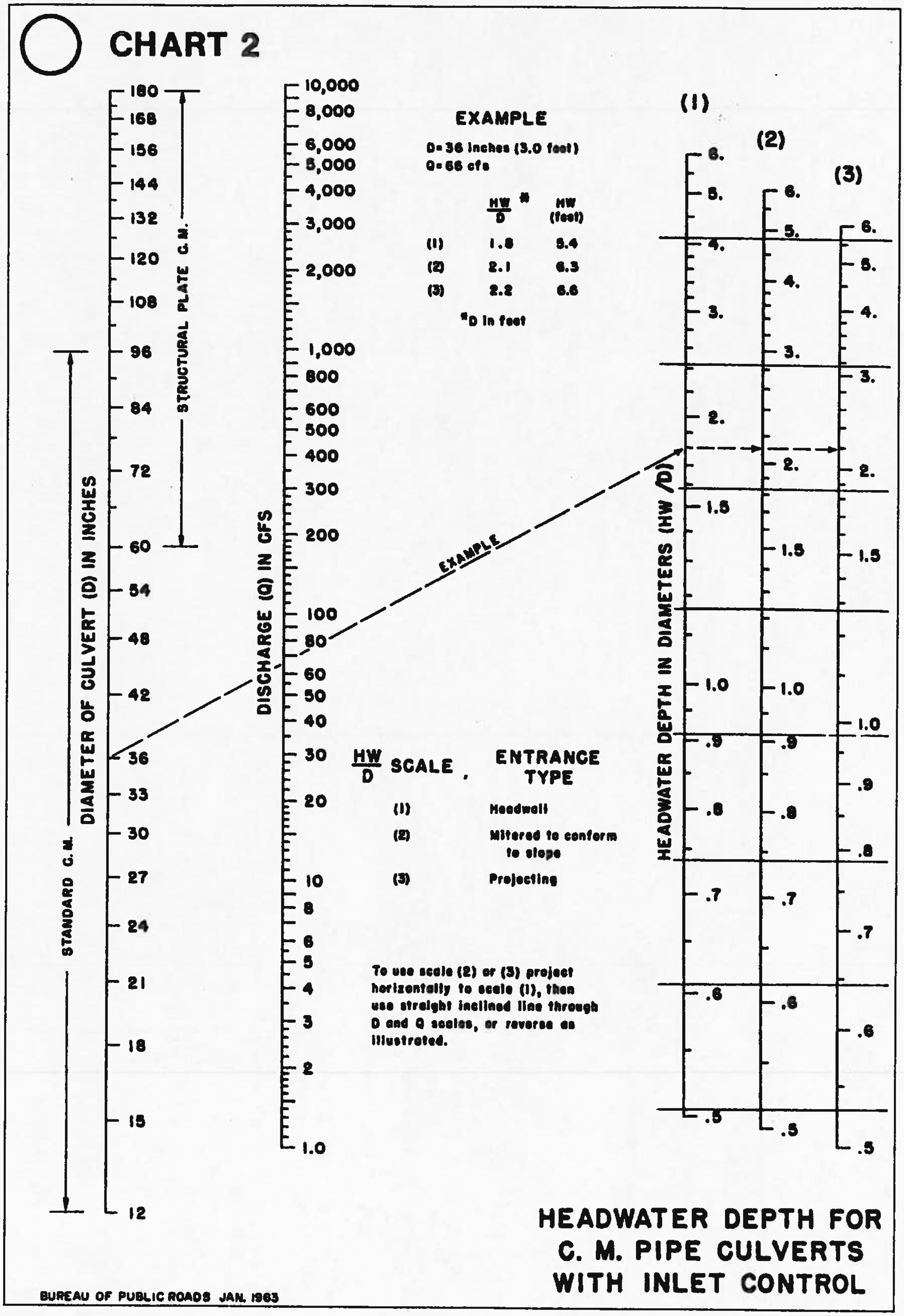


Figure 21.27 HDS-5 Chart 2, nomograph for headwater depth for corrugated metal pipe culvert with inlet control.

can flow full if the headwater depth is high enough to force enough water through the inlet. The headwater depth necessary to achieve full flow in the barrel of the culvert is very large and is usually not achievable, given practical site constraints. Figure 21.24 illustrates the various types of inlet control.

For flows that do not submerge the inlet, as in Figure 21.24*a* and *b*, the flow into the culvert is modeled as weir flow. For submerged conditions, the flow into the culvert is modeled as orifice flow. Equations 21.47, 21.48, and 21.49 were determined from experimental data on culverts by the National Bureau of Standards (NBS). Equations 21.47 and 21.48 are for unsubmerged conditions, and Equation 21.49 is for submerged (orifice) conditions. (Equation 21.48 is another form of Equation 21.47.)

$$\frac{HW_i}{D} = H_c + K\left[\frac{Q}{AD^{0.5}}\right]^M + C_S \text{ (for } Q/AD^{0.5} \leq 3.5) \qquad (21.47)$$

$$\frac{HW_i}{D} = K\left[\frac{Q}{AD^{0.5}}\right]^M \text{ (for } Q/A\,D^{0.5} \leq 3.5) \qquad (21.48)$$

$$\frac{HW_i}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y + C_S \text{ (for } Q/AD^{0.5} \geq 4.0) \qquad (21.49)$$

where HW_i = headwater depth above inlet control section invert (feet), D = interior height of culvert barrel (feet), H_c = specific head at critical depth ($d_c + V^2/2g$) (feet), Q = discharge (cfs), A = full cross-sectional area of culvert barrel (square feet), C_S = slope correction factor (= $0.7S$ for mitered inlets and $C_S = -0.5S$ for all other inlets where S = culvert barrel slope [ft/ft]), and K, M, c, Y = constants from Table 21.7.

The Federal Highway Administration has developed numerous nomographs for various culvert shapes for inlet control. Figures 21.25, 21.26, and 21.27 illustrate several of these nomographs. Others are available in *Hydraulic Design of Highway Culverts* (1985) available through National Technical Information Service, Springfield, VA.

Outlet Control

When the control section is at the downstream side of the culvert, flow through the culvert is either subcritical or pressure flow. Additionally, the amount of flow through the culvert is governed by hydraulic characteristics of both the culvert and tailwater conditions. In culverts under outlet control, the barrel is not capable of carrying all of the water that passes through the inlet. Figure 21.28 shows the various types of outlet control conditions.

Flow through the culvert is a balance between the energy available to pass the water through and the energy consumed by friction and minor losses. The energy balance is given by:

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + \Sigma H_L \qquad (21.50)$$

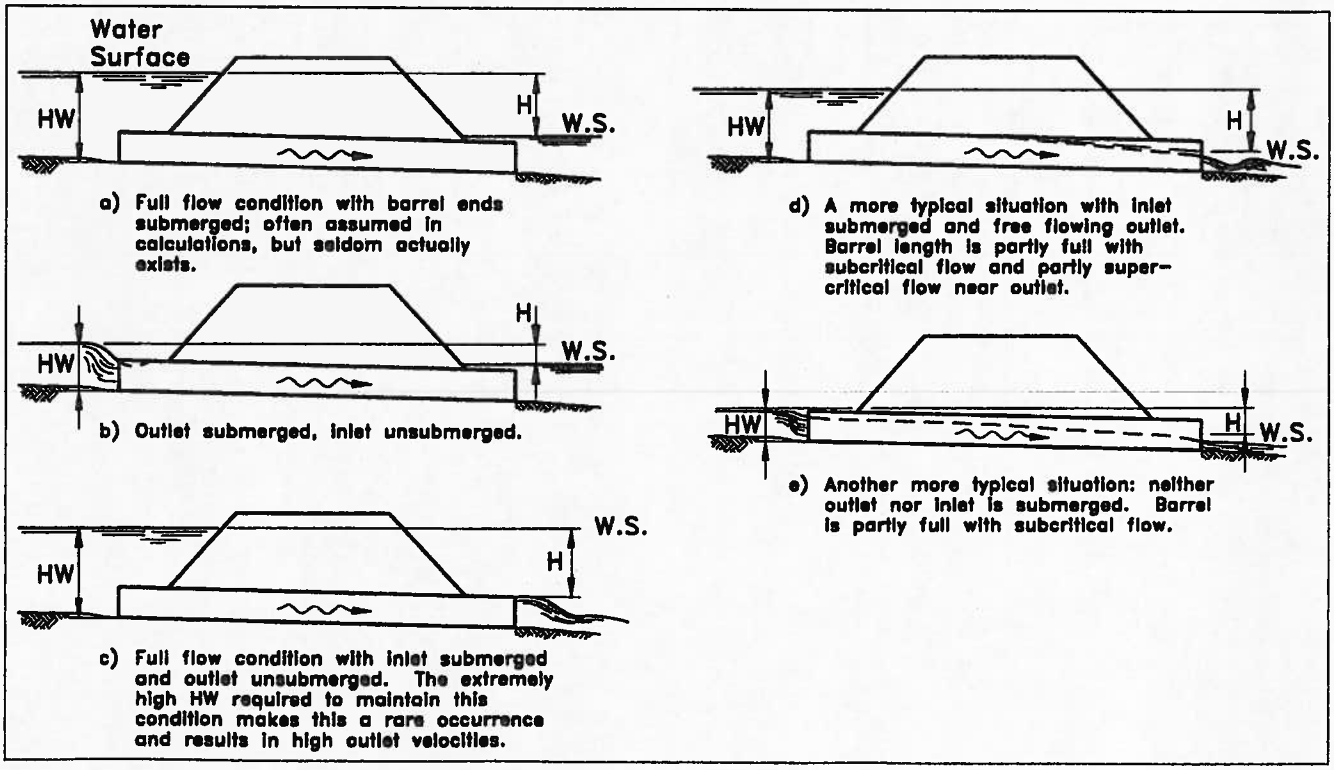


FIGURE 21.28 Types of outlet control.

where HW_o = headwater depth above the outlet invert, V_u = approach velocity of the water at the inlet, TW = tailwater depth above the outlet invert, V_d = discharge velocity of the water at the outlet, and ΣH_L represents the summation of all energy losses through the barrel.

As with pipe flow, energy losses for culverts are attributed to barrel roughness (friction) and minor losses at the inlet and outlet, as well as bends and other appurtenances. Similarly, minor losses are computed as a fraction of the velocity head (kinetic energy) as given in the following equation:

$$h = k\left[\frac{V^2}{2g}\right] \tag{21.51}$$

where the factor k is empirically determined and depends on geometry and hydraulic characteristics. Table 21.8 provides values of k for various entrance conditions. In most cases the k value for exit loss is 1.0.

Friction loss H_f in the barrel of the culvert depends on culvert material and geometry. Using the Manning equation, the friction loss is given as:

TABLE 21.8 Entrance Loss Coefficients

Outlet Control, Full or Partly Full Entrance Head Loss

$$H_e = k_e\left(\frac{V^2}{2g}\right)$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius = 1⁄12 D)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope*	0.5
Beveled edges, 33.7 or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End section conforming to fill slope*	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1⁄12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30°–75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1⁄12 barrel dimension, or beveled top edge	0.2
Wingwall at 10°–25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

**Note:* "End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both *inlet* and *outlet* control. Some end sections, incorporating a *closed* taper in their design have a superior hydraulic performance.

Reference: FHWA, *Hydraulic Design of Highway Culverts*, HDS-5, 1985.

$$H_f = \left[\frac{29\,n^2 L}{R^{1.33}}\right]\frac{V^2}{2g} \qquad (21.52)$$

where n = Manning's roughness coefficient, L = barrel length, and V = average discharge velocity in the barrel. The variable R is the hydraulic radius = A/WP, where A = cross-sectional area of the culvert and WP = wetted perimeter.

The head loss through a straight culvert of uniform cross section is:

$$H = \left(1 + k_e + \frac{29\,n^2 L}{R^{4/3}}\right)\frac{V^2}{2g} \qquad (21.53)$$

where k_e is the entrance loss coefficient. For culverts with bends, sudden expansions, and other features that create energy losses, additional terms are added to Equation 21.53.

PROCEDURE FOR CULVERT DESIGN

1. Determine the culvert alignment. The most desirable alignment follows the natural streambed, although factors such as economics, environmental issues, maintenance considerations, and other site constraints contribute to the alignment selection. Setting the alignment also determines the length and slope of the culvert.

2. Select the design storm, which is frequently dictated by state or local criteria. For culverts crossing roadways, the type of roadway affects the design storm. For secondary roads, the recurrence interval may range from 10 to 25 years. On major highways and interstates, the recurrence interval may be 50 years or greater. Other criteria affecting design may be the freeboard distance or depth of flow allowed on the roadway for a storm other than the design storm.

3. Utilizing land use maps, topographic maps, and whatever hydrologic technique is best suited for the area (e.g., rational method, TR-55 methodology, and regional equations), determine the design discharge for the culvert.

4. Select the culvert material and shape that best conform to the constraints of the site. Note that certain materials and shapes may not be readily available from suppliers. For example, low embankment height may warrant the use of elliptical pipe, or high embankments may warrant structurally enhanced material. Aesthetics may also affect the selection of material and shape.

   Size the culvert using either the equations for inlet and outlet control or the culvert nomographs, whichever gives the greatest controlling elevation. The FHWA culvert design form of Figure 21.29 provides a guide to the selection of a culvert. For the design discharge, the headwater depth is computed assuming inlet control and outlet control. The type of control that produces the highest headwater depth is the governing control for the design.

6. After sizing the culvert for the design discharge, check to see that other requirements, as dictated by applicable design standards, are met. Such requirements might include freeboard height or roadway overtopping for lower-recurrence interval storms, effects and impacts of upstream ponding, the need for erosion control for excessive outlet velocities, structural stability, and so on.

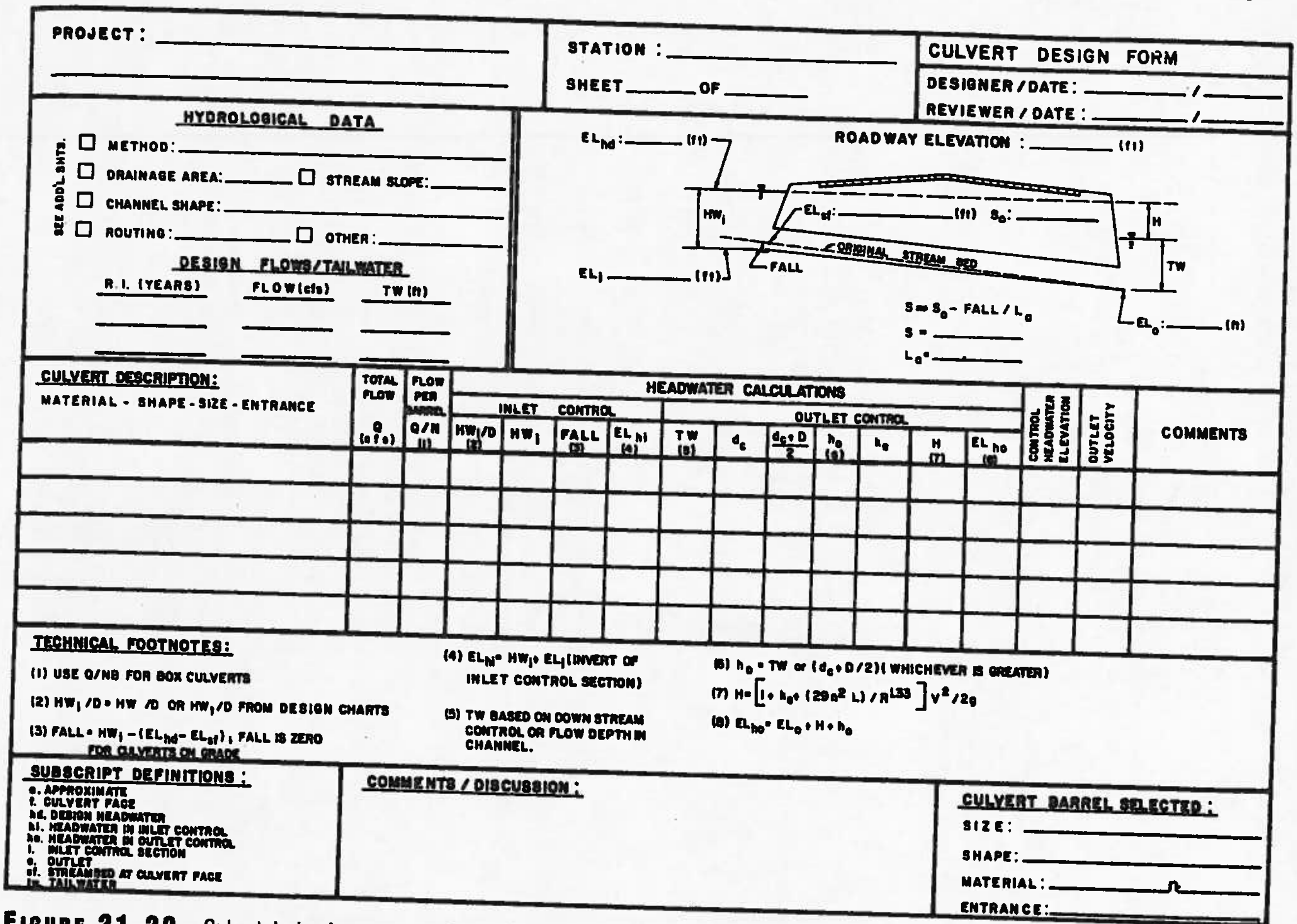
Culvert Design Example

Size a concrete (n = 0.013, k_e = 0.5) box culvert for the following constraints: 25-year design discharge = 461 cfs with tailwater depth = 2.6 feet; 100-year design discharge = 690 cfs with tailwater depth = 4.3 feet. At least 1.5 feet of freeboard is required for the 25-year design discharge, and the maximum allowable flow depth over the embankment for the 100-year design discharge is 12 inches. The upstream invert elevation = 314.2 feet, the downstream invert elevation 313.8 feet, and the top of the roadway elevation = 324.5 feet. The culvert is 140 feet long. Assume flared wingwalls with angle of flare between 30° and 75° and square-edge conditions. The minimum allowable cover over the culvert is 3.0 feet. From the given data, the constraints for the 25-year discharge are:

$$\begin{aligned}
&\text{Maximum headwater depth} = 324.5 - 1.5 - 314.2 = 8.8\text{ ft}\\
&\text{Maximum culvert height} \quad = 324.5 - 3 - 314.2 \qquad (21.54)\\
&\qquad\qquad\qquad\qquad\qquad\;\; = 7.3\text{ ft}\\
&\text{Slope} \qquad\qquad\qquad\quad\;\; = \frac{314.2 - 313.8}{140} = 0.003\text{ ft/ft}
\end{aligned}$$

Sizing a culvert is a trial-and-error procedure. Estimating a first-try size becomes more intuitive with experience. In this example, select a trial HW/D based on the maximum headwater depth and culvert height, HW/D = 8.8/7.3 = 1.2, to obtain a first approximation. (Note: When finalizing the culvert design, the engineer should consider the physical characteristics of an installation and determine that the culvert will fit safely underneath the roadway pavement box. This should be done by drawing a profile of the culvert with relation to the proposed ground above the culvert. The thickness of the culvert should also be considered when profiling the system.) For inlet control conditions, use the nomograph in Figure 21.25. On the scale at the right, for HW/D = 1.2, extend a straight line to the left scale (height of box) value of 7 and read Q/B = 68 cfs/ft on the middle scale. Therefore, a discharge of 461 cfs requires a 461/68 = 6.8-foot-wide box culvert. The first trial size is 7 feet by 7 feet. The corresponding headwater depth is 8.3 feet.

Now check the HW depth for outlet control conditions. From Table 21.8 for the given headwall/wingwall configuration, the entrance loss coefficient k_e is 0.5. Substituting the values into Equation 21.53 gives:


PROJECT:

STATION:

SHEET ___ OF ___

CULVERT DESIGN FORM

DESIGNER / DATE: ___ / ___

REVIEWER / DATE: ___ / ___

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD:

☐ DRAINAGE AREA: ___ ☐ STREAM SLOPE:

☐ CHANNEL SHAPE:

☐ ROUTING: ___ ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW(cfs)	TW (ft)

EL_{hd}: ___ (ft)

ROADWAY ELEVATION: ___ (ft)

HW_i

EL_{sf}: ___ (ft) S_o: ___

ORIGINAL STREAM BED

FALL

EL_i ___ (ft)

H

TW

$S = S_o - FALL / L_a$

S =

L_a =

EL_o: ___ (ft)

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE

CULVERT DESCRIPTION	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

HEADWATER CALCULATIONS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

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

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE:

SHAPE:

MATERIAL: ___ n

ENTRANCE:

FIGURE 21.29 Culvert design form.

$$H = \left(1 + 0.5 + \frac{(29)(0.013)^2(140)}{(49/28)^{4/3}}\right)\frac{(461/49)^2}{2g} \quad (21.55)$$

$$= 2.5 \text{ ft}$$

The critical depth d_c for the box culvert using the given data is:

$$d_c = 0.315^3\sqrt{\left(\frac{Q}{B}\right)^2} \quad (21.56)$$

$$= 0.315^3\sqrt{\left(\frac{461}{7}\right)^2}$$

$$= 5.1 \text{ ft}$$

and $(d_c + D)/2 = (5.1 + 7)/2 = 6.1$ feet. The variable h_o is the greater of either the tailwater elevation or $(d_c + D)/2$. Here $(d_c + D)/2 = 6.1$ feet > TW = 2.6 feet.

Finally, the headwater depth for outlet control is:

$$HW = H + h_o - LS_o \quad (21.57)$$

$$= 2.5 + 6.1 - 140\,(.003) = 8.1 \text{ ft}$$

Since the headwater depths for both inlet control (= 8.3 feet) and outlet control (= 8.1 feet) are less than or equal to the maximum allowable headwater depth (= 8.8 feet), a 7-foot by 7-foot box culvert is an acceptable size. Also, since the headwater depth for inlet control is greater than the headwater depth for outlet control, the culvert operates as inlet control.

The second part of the design is to check whether the flow depth over the road embankment for the 100-year discharge is less than 12 inches. From the nomograph in Figure 21.25, extend a straight line from the left scale (height of box = 7 feet) through the $Q/B = 690/7 = 98$ cfs/ft and read $HW/D = 1.8$ on scale 1 at the left. The resulting headwater depth = $(7 \times 1.8) = 12.6$ feet.

Now determine the headwater depth for outlet control conditions. From Equation 21.53:

$$H = \left(1 + 0.5 + \frac{(29)(0.013)^2(140)}{(49/28)^{4/3}}\right)\frac{(690/49)^2}{2g} \quad (21.58)$$

$$= 5.6 \text{ ft}$$

This same value can be obtained using the outlet control nomograph for the concrete box culvert of Figure 21.30. First, draw a straight line from the dimension of square box scale (7 feet by 7 feet) to the length scale value of 140 feet for $k_e = 0.5$. Next, draw a straight line from the discharge scale

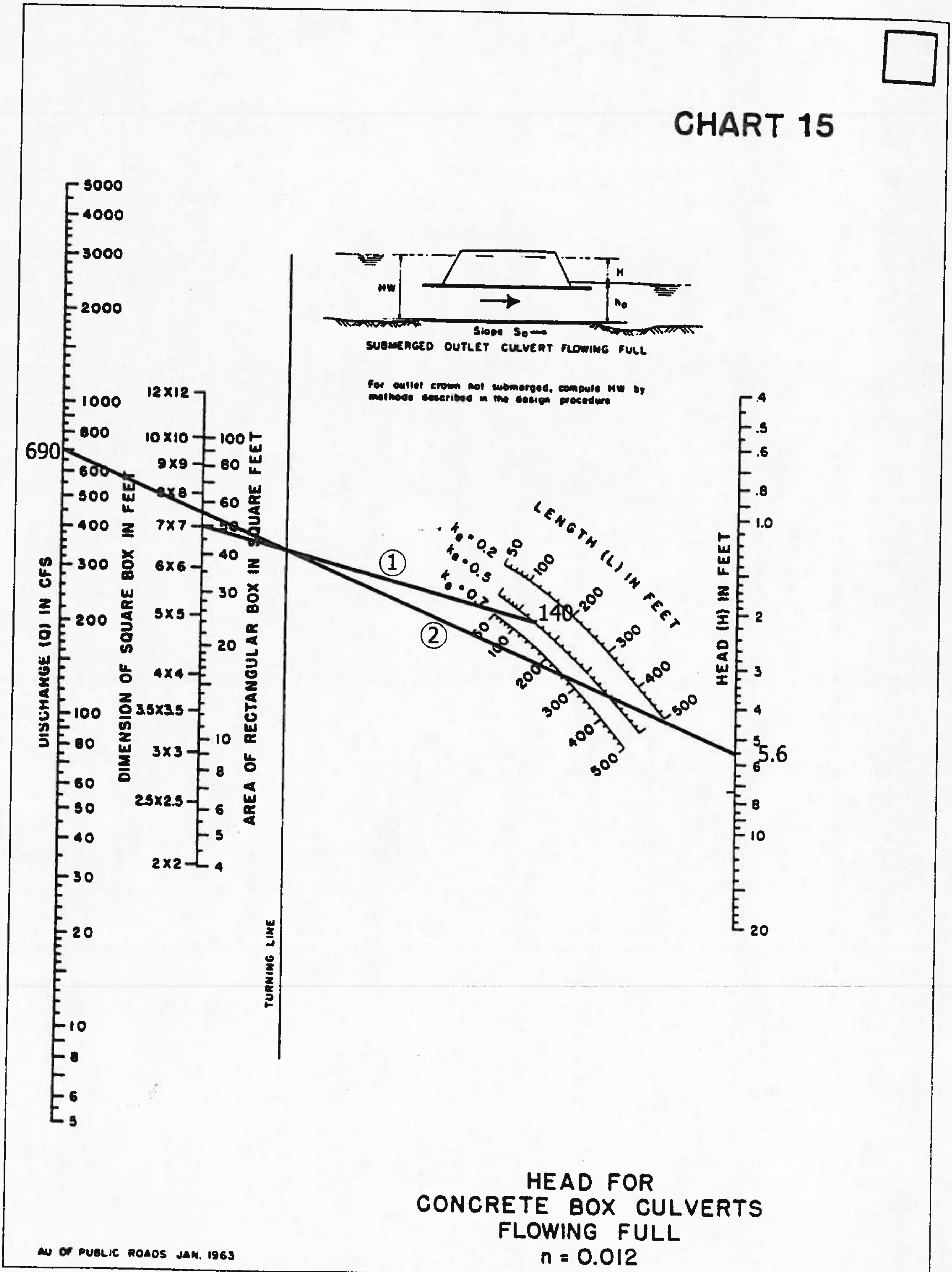


FIGURE 21.30 Outlet control nomograph for example problem.

(= 690 cfs) through the point where the previously drawn line intersects the turn line to the head (H) scale on the left. Read a value of H = 5.6 feet.

The critical depth d_c is:

$$d_c = 0.315^3\sqrt{\left(\frac{Q}{B}\right)^2} \qquad (21.59)$$

$$= 0.315^3\sqrt{\left(\frac{690}{7}\right)^2}$$

$$= 6.5 \text{ ft}$$

where $(d_c + D)/2 = (6.5 + 7)/2 = 6.8$ feet. Hence, the variable h_o = 6.8 feet. The headwater depth for outlet control is:

$$HW = H + h_o - LS_o \qquad (21.60)$$

$$= 5.6 + 6.8 - 140\,(.003) = 12.0 \text{ ft}$$

For the 100-year discharge, the culvert operates under inlet control (i.e., 12.6 > 12.0).

Based on the assumption of no roadway overtopping, the headwater elevation for the 100-year discharge is 326.8, which is 2.3 feet above the road. Finding the actual depth over the road is another trial-and-error procedure. For a given headwater (above elevation 324.5 feet), the total flow is the amount that the given headwater depth pushes through the culvert plus the amount of flow going over the embankment. The objective is to find the headwater depth where the combined flows equal the specific design discharge.

Flow over the embankment is typically modeled as a weir. The length of the weir and depth of water over the weir are difficult to determine when the road profile is a sag vertical curve or when the embankment surface has a very irregular shape. There are various numerical methods to approximate each of these values. Sometimes a simple approximation of a broad crested rectangular weir can be used. For more complex situations, many culvert analysis programs allow for the input of the roadway profile above the culvert and calculate the weir flow incrementally over the road until the resulting discharge balances. Weir flow is:

$$Q_w = C_w L\, H^{3/2} \qquad (21.61)$$

where C_w is a weir coefficient that depends on the shape and depth of water above the weir relative to the depth of water below the weir, L is the length of the weir, and H is the depth of water above the weir. For a broad crested trapezoidal weir, C_w ranges between 2.5 and 3.1. Refer to hydraulic handbooks, such as Brater et al. (1996), for various other values for different types of weirs.

To determine whether the roadway overtops less than 1 foot, as was required in the example problem, we will assume that a 50-foot-long rectangular weir approximates the overtopping portion of the roadway. Using Equation 21.61, a C_w = 2.6, L = 50 feet, and H = 1 foot (allowable overtopping) yields a weir discharge of 130 cfs. Next, we determine whether the combination of weir flow and culvert flow, at the allowable elevation of 325.5 feet (1 foot above the road elevation), is sufficient to pass the 100-year discharge.

Using an elevation of 325.5 feet, the corresponding HW/D ratio is (325.5 − 314.2)/7 = 1.61. From the box culvert with inlet control nomograph (Figure 21.24), construct a straight line from HW/D scale (1) at HW/D = 1.61 to the left scale for D = 7 feet. Read Q/B = 90 and find Q = 7(90) = 630 cfs. The total flow of culvert and weir flow is 630 + 270 = 760 cfs. Our 100-year design discharge is equal to 690 cfs. Therefore, there is sufficient capacity within the culvert and the 1 foot of allowable overtopping of the road to pass the design discharge. A trial-and-error process using the same procedures could calculate the actual 100-year water surface elevation.

This example uses the culvert nomograph for inlet control to calculate the headwater depth. Equations 21.47, 21.48, and 21.49 could also have been used for the same purpose. The nomograph is convenient and quick, however, and the equations can be incorporated into computer programs or hand-held calculators. A variety of computer programs are available to assist and automate the culvert analysis process. There are also numerous inlet and outlet nomographs for various culvert shapes that can be found in *Hydraulic Design of Highway Culverts* by the FHWA, available from National Technical Information Service (NTIS), Springfield, Virginia.

Major Culverts

The preceding example demonstrates how a 7-foot by 7-foot box culvert would meet the minimum design requirements for the given discharges. Major culverts usually have different definitions in different localities but are usually large single- and multiple-cell culverts with drainage areas of 200 acres or greater. These crossings may not fall under floodplain requirements mandated by localities or the federal government; therefore, careful consideration should be given to the analysis, design, and construction of these structures.

When analyzing larger culvert crossings, simple single-basin hydrology or rational method hydrology may not be appropriate. Developing models to take into account terrain, soil types, land covers, and various inflow points may be necessary; therefore, developing a rainfall runoff model such as HEC-HMS or TR-20 may be appropriate. Regression equations or other regionally adopted approaches may be suitable and even required by some localities to estimate the runoff to the culvert. The range of storms required for analysis may vary as well. Larger culverts may need to be designed for 25-, 50-, or even the 100-year storm events based on the classification of the roadway, proximity to existing or proposed development, or localized soil conditions.

Hydraulic analysis and design of these major culvert crossings may also require more complex approaches than previously described. The nomographs used in the previous example, although still effective in predicting the appropriate size of a culvert, may be overly conservative. With many major culvert crossings, a more appropriate hydraulic analysis may be more similar to a stormwater management or reservoir routing process than simple pipe flow. The nomograph approach does not consider storage on the upstream side of the culvert. Hand calculations (typical reservoir routing) or most simple hydrologic computer programs can handle these calculations, and the results give a more realistic answer to the headwater elevations and discharge through the culvert. The computer models can also give you an approximation of the amount of time the culvert embankments may be saturated during a particular storm event. Oversaturation of water sitting behind a culvert for long periods of time could cause eventual slope and/or erosion problems with certain soil types and embankments.

More detailed hydraulic programs (step backwater programs such as HECRAS or WSPRO) may also be helpful in analyzing the impacts major culverts have on the upstream property. Major culverts may also have bend losses or be located in series such that these models can better predict the water surface elevations for the different design storms. Upstream conditions may be more affected by the headwater conditions at the culvert, and as a result, you can more accurately determine the amount of impact to the upstream property and where that impact would dissipate or tie out. This type of analysis may be required (even if a floodplain analysis is not) so that the proper easements can be obtained upstream to encompass the predicted increase in water surface elevations.

There are other issues when dealing with major culverts that may factor into the overall size and hydraulic requirements. For instance, upstream flood stages may be increased above what might be acceptable. Besides the local design criteria, another consideration is the amount of upstream land inundated because of the increased flood elevation. In flat areas, with significant land value, a 0.5-foot rise in flood stage may mean acres of land impacted by flooding, resulting in reduced development potential or increased development costs for grading. Therefore, the cost to increase the culvert size in order to reduce upstream water surface elevations could be warranted.

Another consideration to culvert sizing includes providing for the increase in runoff from proposed upstream development. Oftentimes, communities require that the designer consider the potential for upstream development, usually based from an approved comprehensive or zoning plan. For many culvert installations, the required design storm could be the 10-, 25-, or even 50-year storm. Most localities do not regulate SWM controls to mitigate for the 25- or 50-year storm events. Many localities only consider the 10-year and smaller events. Therefore, a culvert located in the lower part of a watershed, especially where development potential is high, should be sized to accommodate the increase in peak discharges over time.

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