3. Using eq. (14.8),

$$q = \frac{(1)(0.8)(6.69)}{43{,}200}(17.7) = 0.0022 \text{ cfs/ft}$$

4. $Q = 0.0022(150) = 0.33$ cfs.

14.7 CROSS-DRAINAGE SYSTEMS: CULVERTS

Bridges and culverts are two cross-drainage works that pass stream channels under roadways. The hydraulics of bridge openings have been discussed in Section 10.8.2. The distinction between a bridge and a culvert on the basis of size is arbitrary, with a structure whose span is in excess of 20 ft being classed as a bridge. However, a distinctive feature is that culverts can be designed to flow with a submerged inlet. A culvert acts as a control structure. In the hydraulic sense, a device is said to control flow if it limits the flow of water which would otherwise be exceeded under existing upstream and downstream conditions. In a control device, the head adjustment across the control section takes place until a balance is achieved between the inflow and the discharge through the section. In the case of culverts, a difficulty arises because the control section can be at the inlet or at the outlet, depending on the type of flow. In supercritical flow the flow velocity is faster than the velocity of a wave, so that the water waves cannot travel upstream, and hence control cannot be exercised from downstream (i.e., there is inlet control). In the subcritical flow condition, control from downstream will back up water until an equilibrium profile is achieved upstream of the control (i.e., outlet control exists).

Inlet control means that conditions at the entrance—depth of headwater and entrance geometry—control the capacity of the culvert. An orifice type of flow takes place at the entrance. A culvert runs part full (atmospheric pressure). Thus the barrel size beyond the inlet can be reduced without affecting the discharge, or the capacity can be increased by improving the inlet conditions. The detailed design of improved inlets has been discussed by DOT (1972). These improvements comprise provision of wingwalls; beveling or rounding of culvert edges; tapering the sides of the inlet, including slope tapering; and providing a drop inlet. The geometry of the top and sides of the inlet is important, but not as important as that of the culvert floor. The inlet geometry and channel contraction affect the coefficient of discharge as discussed by Bodhaine (1982).

In outlet control, the culvert can flow full or part full, depending on headwater and tailwater levels. The friction head in the barrel of a culvert affects the headwater or the total energy to pass the discharge through the culvert.

Discharge through culverts depends not only on the type of control but on different types of flow under each control. A general classification of flow through culverts is shown in Table 14.5, separated into two groups: unsubmerged and submerged flow. For submerged flow, the headwater-to-barrel diameter ratio should exceed approximately 1.2. The features of each type of flow with respect to culvert slope, flow depth, and control section are indicated in the table. The first three types relate to unsubmerged flow, with the first one relating to inlet control conditions. The other three types in the table relate to submerged culvert flow. The discharge equations given in the table for each type disregard entrance losses.

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Table 14.5 Classification of Culvert Flow

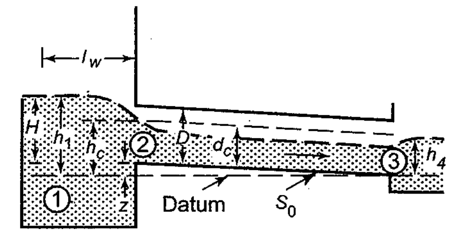
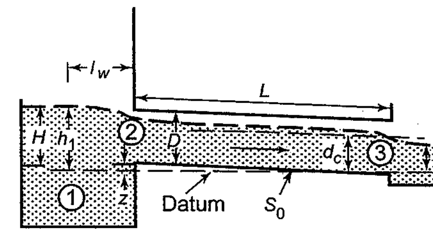
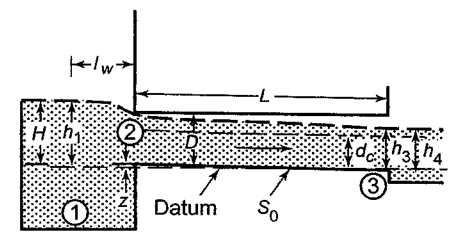
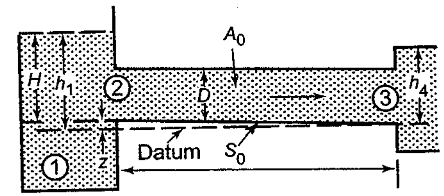
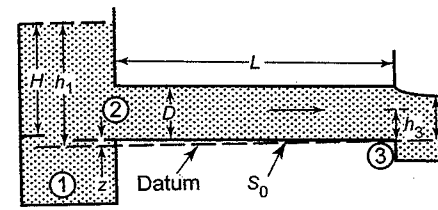
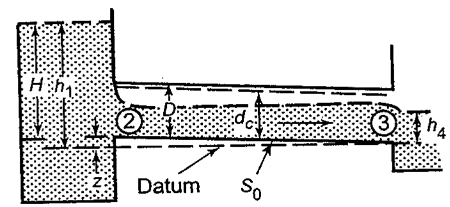
(1) Category	(2) Type	(3) Culvert Slope	(4) Flow	(5) Control Section	(6) Discharge, Q	(7) Eq. Number	(8) Illustration
Unsubmerged, H/D ≤ 1.2	1	Steep	Part full	Inlet	$C_d A_c \sqrt{2g\left(H + V_1^2/2g - dc - h_{1,2}\right)}$	(14.10)	
	2	Mild	Part full	Outlet	$C_d A_c \sqrt{2g\left(H + z + V_1^2/2g - dc - h_{1,2} - h_{2,3}\right)}$	(14.11)	
	3	Mild	Part full	Outlet	$C_d A_3 \sqrt{2g\left(H + z + V_1^2/2g - h_3 - h_{1,2} - h_{2,3}\right)}$	(14.12)	

Table 14.5 Classification of Culvert Flow (Continued)

(1) Category	(2) Type	(3) Culvert Slope	(4) Flow	(5) Control Section	(6) Discharge, Q	(7) Eq. Number	(8) Illustration
Submerged, $H/D > 1.2$	4	Any	Full	Outlet	$C_d A_0 \sqrt{\dfrac{2g(H+z-h_4)}{1+2\alpha C_d^2 n^2 L/R_0^{4/3}}}$	(14.13)	
	5	Any	Full	Outlet	$C_d A_0 \sqrt{\dfrac{2g(H+z-D)}{1+\alpha C_d^2 n^2 L/R_0^{4/3}}}$	(14.14)	
	6	Any	Full	Intlet	$C_d A_0 \sqrt{2gH}$	(14.15)	

α = 29 for FPS units and 19.6 for metric units.
A_c = area of section of flow at critical depth.
A_0 = area of culvert barrel.
A_3 = area of section of flow at exit end of culvert.
R_0 = hydraulic radius of culvert barrel.
V_1 = mean velocity in the approach section.
Other variables are shown on figures.

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14.7.1 Design of Culverts

Some box culverts are designed such that their top forms the base of the roadway. These are unsubmerged culverts that belong to types 1, 2, and 3. For a trial selected size, the type of flow can be determined as follows:

1. For the design flow, determine the critical depth, d_c (Section 11.5.2), and the normal depth, d_n (Section 11.6.2).
2. Compare the depths above with the tailwater, h_4. When:

$d_n < d_c$ and $h_4 < d_c$	type 1 flow	critical depth at inlet
$d_n > d_c$ and $h_4 < d_c$	type 2 flow	critical depth at outlet
$d_n > d_c$ and $h_4 > d_c$	type 3 flow	subcritical throughout

The appropriate discharge equation of Table 14.5 is used to confirm the size and type. If not adequate, then guided by the computed size, the trial may be repeated. In eqs. (14.10) through (14.15) in Table 14.5, the friction head loss between indicated sections is determined by Manning's equation, arranged as follows:

$$h_{ab} = \frac{n^2 L V^2}{2.22 R^{4/3}} \quad [L] \quad \text{(English units)} \tag{14.16a}$$

or

$$h_{ab} = \frac{n^2 L V^2}{R^{4/3}} \quad [L] \quad \text{(metric units)} \tag{14.16b}$$

where

h_{ab} = friction head loss between two points a and b
V = velocity of flow
L = length of section ab
R = hydraulic radius, A/P

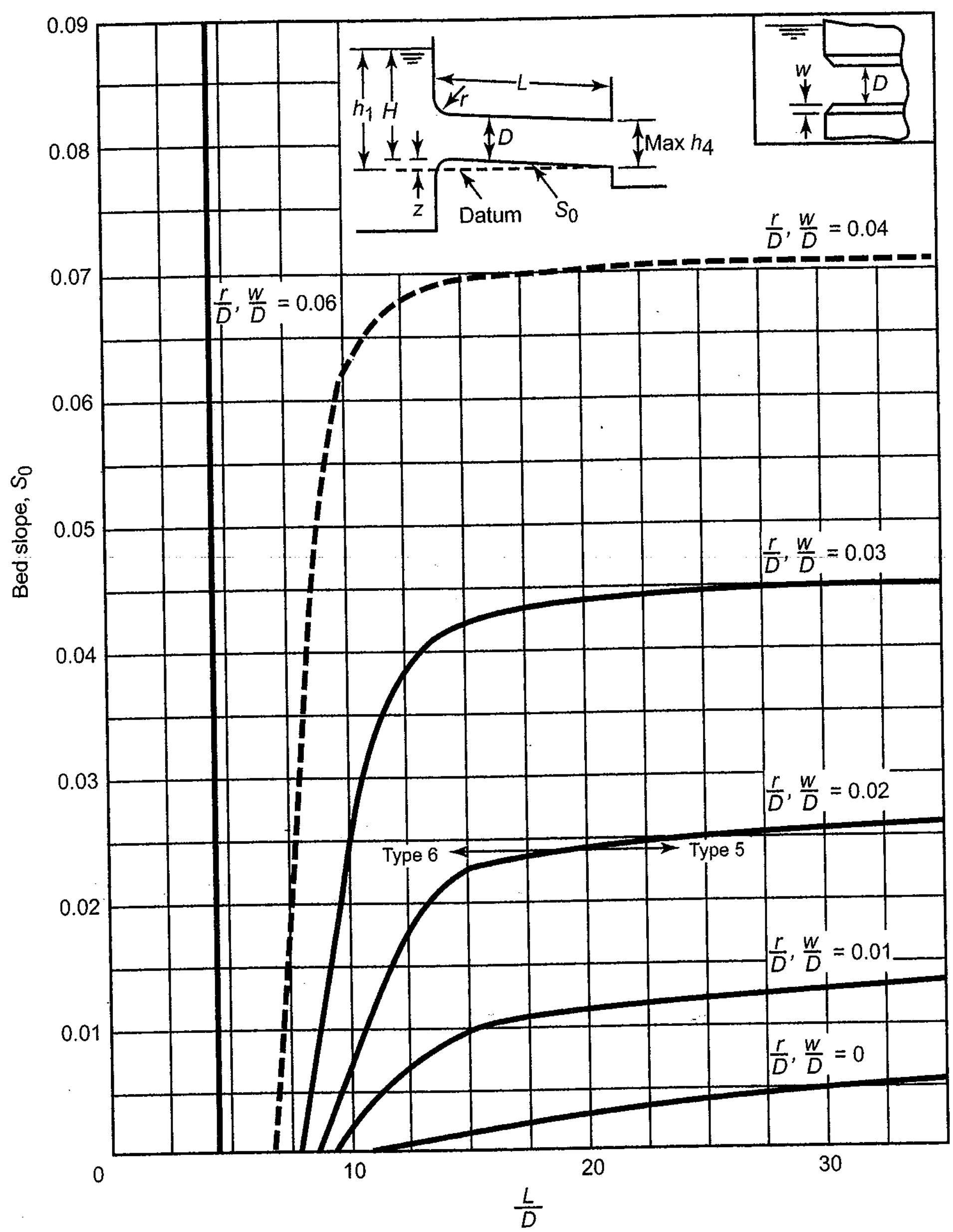
The majority of culverts are designed for submerged conditions (types 4, 5, and 6), since the entrance is submerged at least with the peak rate of flow.

When a culvert is submerged by both headwaters and tailwaters, it is a type 4 condition in which eq. (14.13) is applicable. However, the distinction between types 5 and 6 when the tailwater is low is not as obvious.

To classify type 5 or 6 flow, the curves of Figures 14.6 and 14.7, which are adapted from Bodhaine (1982), are used. Figure 14.6 is applicable to a concrete barrel box or pipe culverts of square, rounded, or beveled entrances with or without wingwalls. Figure 14.7 is for rough (corrugated) pipes of circular or arch sections mounted flush in a vertical headwall with or without wingwalls. The procedure to classify type 5 or 6 flow is as follows:

1. Compute the ratios L/D, r/D or w/D, S_0, and (for rough pipes), $\alpha n^2 H / R_0^{4/3}$ where r is the radius of rounding, w is the effective bevel, and $\alpha = 29$ for FPS units and 19.6 for metric units, shown in Figures 14.6 and 14.7.
2. For concrete pipes, select the curve of Figure 14.6 corresponding to r/D or w/D for the culvert.

Figure 14.6 Criterion for classifying type 5 and type 6 flow in box or pipe culverts with concrete barrels and square, rounded, or beveled entrances, either with or without wingwalls (from Bodhaine, 1982).



3. For rough pipes, select from Figure 14.7 the graph corresponding to the value of r/D for the culvert and then select the curve corresponding to the $\alpha n^2 H / R_0^{4/3}$ computed for the culvert.
4. Plot the point defined by the computed values of S_0 and L/D for the culvert.

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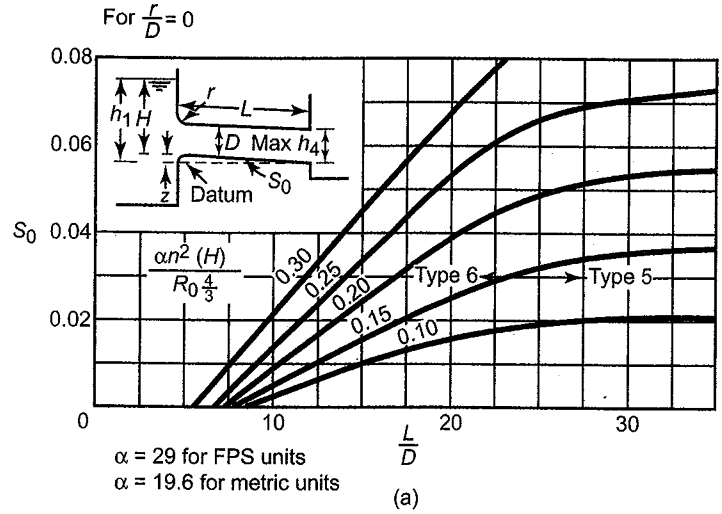
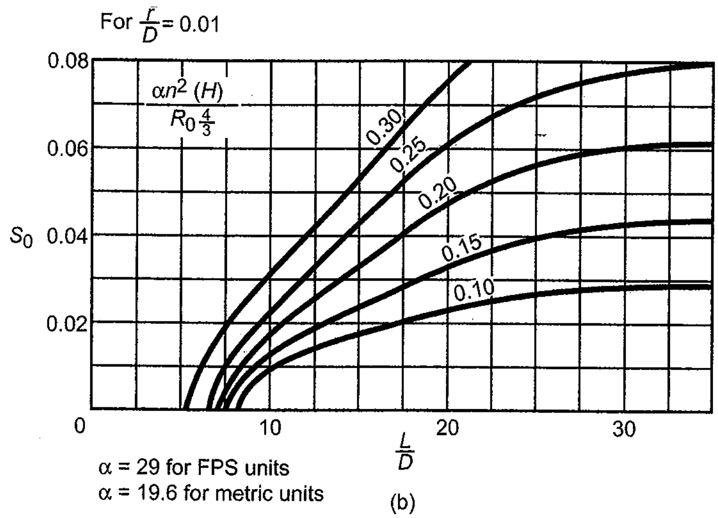
Figure 14.7 Criterion for classifying type 5 and type 6 flow in pipe culverts with rough barrels (from Bodhaine, 1982).

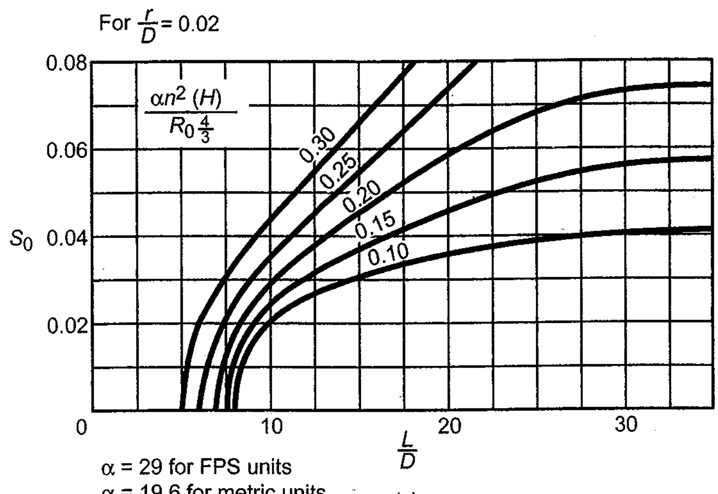




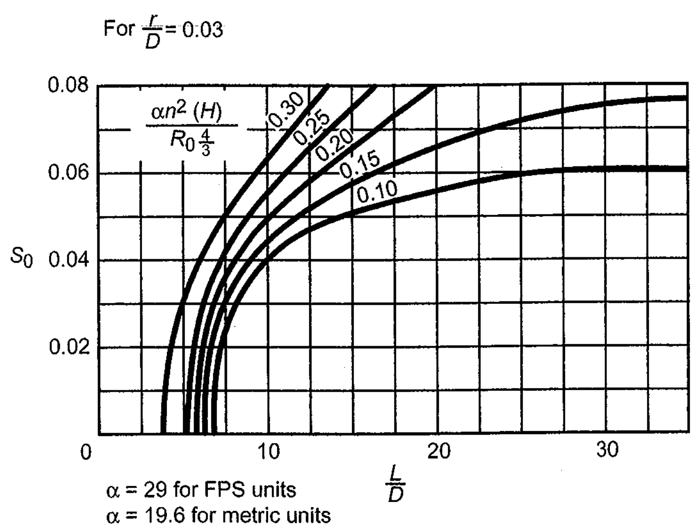
5. If the point plots to the right of the curve in step 2 or 3, the flow is type 5. If it plots to the left, the flow is type 6.

As in the case of bridge openings, the coefficient of discharge, C_d, is a function of many variables relating to type of flow, degree of channel contraction, and the geometry of the culvert entrance. The coefficient varies from 0.4 to 0.98. A systematic presentation has been made by Bodhaine (1982).

Figure 14.7 (Continued) Criterion for classifying type 5 and type 6 flow in pipe culverts with rough barrels (from Bodhaine, 1982).



(c)



(d)

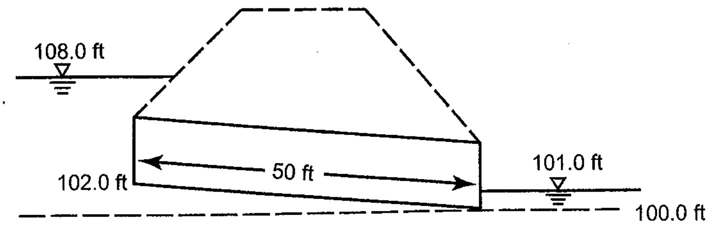
EXAMPLE 14.5

A culvert section is shown in Figure 14.8 with upstream and downstream water levels. Design the culvert for a peak discharge of 120 cfs. For the culvert section, corrugated metal pipe ($n = 0.024$) is to be used without rounding. $C_d = 0.5$.

SOLUTION

1. Consider a pipe section 4 ft in diameter.
2. $H/D = 6/4 = 1.5 > 1.2$; submerged flow

Figure 14.8 Submerged culvert section of Example 14.5.



3. Since not submerged by tailwaters, it is either type 5 or 6.
4. Thus,

$$r/D = 0,\ S_0 = \left(\frac{102.0 - 100.0}{50}\right) = 0.04,\ L/D = 50/4 = 12.5$$

$$R_0 = D/4 = \frac{4}{4} = 1,\ \frac{29n^2H}{R_0^{4/3}} = \frac{29(0.024)^2(6)}{(1)^{4/3}} = 0.10$$

From Figure 14.7(a), flow is type 6, as the point plots to the left of curve of 0.10.

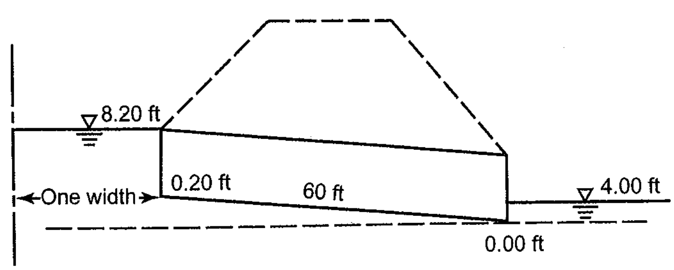
5. Apply eq. (14.15):

$$A_0 = \frac{\pi}{4}(4)^2 = 12.56 \text{ ft}^2$$
$$H = 108 - 102 = 6 \text{ ft}$$
$$Q = 0.5(12.56)\sqrt{2(32.2)(6)}$$
$$= 123 \text{ cfs} \approx 120 \text{ cfs ok}$$

Example 14.6

Design a box culvert of concrete section ($n = 0.015$) to carry a discharge of 520 cfs for the condition shown in Figure 14.9. It has a square-edged entrance. The approach stream has a rectangular section of width 40 ft. $C_d = 0.93$.

Figure 14.9 Unsubmerged box culvert section.



SOLUTION

1. Assume an 8-ft square section. The approach section will be one width (8 ft) upstream.
2. $H/D = 8/8 = 1.0 < 1.2$; unsubmerged case.
3. Determine the critical depth. From eq. (11.7),

$$Z_c = \frac{Q}{\sqrt{g}} = 520/\sqrt{32.3} = 91.64 \text{ ft}$$

Since

$$A_c = 8d_c$$
$$Z_c = A_c^{3/2}/T^{1/2} = (8d_c)^{3/2}/(8)^{1/2} = 8.0d_c^{3/2}$$
$$8.0d_c^{3/2} = 91.64 \text{ or } d_c = 5.09 \text{ ft}$$

4. Determine the normal depth. From eq. (11.10a),

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}} = \frac{520(0.015)}{1.49(0.0033)^{1/2}} = 91.13$$

But

$$AR^{2/3} = (8d_n)^{5/3}/(8+2d_n)^{2/3}$$

Hence

$$\frac{(8d_n)^{5/3}}{(8+2d_n)^{2/3}} = 91.13$$

Solving either by trial and error or by plotting d versus $AR^{2/3}$, $d_n = 6.3$ ft.

5. Since $d_n > d_c$ but $h_4 < d_c$ it is type 2 flow.
6. For velocity of approach,

$$V_1 = \frac{Q}{A} = \frac{520}{8.2(40)} = 1.59 \text{ ft/sec}$$
$$\frac{V_1^2}{2g} = \frac{(1.59)^2}{2(32.2)} = 0.04 \text{ ft}$$

7. For head losses between 1 and 2,

$$h_{1,2} = \frac{n^2 L V^2}{2.22 R_1^{4/3}}$$
$$R_1 = \frac{40(8.2)}{40+2(8.2)} = 5.82 \text{ ft}$$
$$h_{1,2} = \frac{(0.013)^2(8)(1.59)^2}{2.22(5.82)^{4/3}} = \text{negligible}$$

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8. For head losses between 2 and 3,

$$A_2 = 8(6.3) = 50.4 \text{ ft}^2$$

$$R_2 = \frac{50.4}{8+2(6.3)} = 2.45 \text{ ft}$$

$$V_2 = \frac{Q}{A_2} = \frac{520}{50.4} = 10.32 \text{ ft/sec}$$

$$h_{2-3} = \frac{(0.015)^2(60)(10.32)^2}{2.22(2.45)^{4/3}} = 0.20 \text{ ft}$$

9. From eq. (14.11), $A_c = 8(5.09) = 40.72 \text{ ft}^2$

$$Q = C_d A_c \sqrt{2g\left(H + z + \frac{V_1^2}{2g} - d_c - h_{1-2} - h_{2-3}\right)}$$

$$= 0.93(40.72)\sqrt{2(32.2)(8+0.2+0.04-5.09-0-0.20)}$$

$$= 522 \approx 520 \text{ cfs ok}$$

14.8 AIRPORT DRAINAGE SYSTEMS

The objectives of airport drainage systems are (1) to collect and drain surface water runoff, (2) to remove excess groundwater and lower the water table where it is too high, and (3) to protect all slopes from erosion.

The first objective is met by (1) properly grading the airport area so that all shoulders and slopes drain away from runways, taxiways, and all paved areas; (2) providing a field storm drainage system serving all the depressed areas; and (3) constructing peripheral and other ditches to convey the outfall from the drainage system, to collect surface flows from the airport and adjoining sites, and to intercept groundwater flow from higher adjacent areas. Proper coordination of grading and draining is most desirable since a drainage system cannot function effectively unless the area is graded correctly to divert the surface flow into the drainage system. Similarly, ditches form an integral part of the drainage system.

Subsurface drainage is provided to take care of the second objective of diverting subterranean flows, lowering the water table, and controlling the moisture in the base and subbase of the pavements. Intercepting ditches or intercepting drainlines are provided to collect flows through the porous water-bearing stratum. For draining off the moisture pocketed in pervious soils over an impervious stratum or in the low-lying areas of an undulating impervious stratum, the subsurface drains are placed within wet masses of soil. It is desirable to place the best drainable soils adjacent to and beneath the paved areas to provide drainage away from the pavement. Less-drainable soils are placed in nontraffic areas. The draining of large areas through subsurface drainage systems is usually not required on airports since it can be done more efficiently by grading properly and installing surface drainage (Federal Aviation Agency, 1965).

Cut-and-fill slopes address the problem of erosion. As a first step of protection, these slopes are made as flat as possible. Deep-cut slopes of over 10 ft, with higher ground above them, are provided with a cutoff ditch running back to the top-of-cut line and set back a few feet from the top of the bank to intercept the water flowing down from the higher ground. A ditch is constructed at the base of the bank to collect runoff. The cut slopes are protected by riprap, sod, grass, or vegetation. The fill slopes above 5 ft high are protected by constructing beams and gutters along the top of the slope to prevent water from running down the slope.

Only the surface storm drainage is discussed here. The design starts with a comprehensive study of the topography of the site and surrounding areas to identify surface and subsurface direction of flow, natural water courses, and outfalls. The topography affects the layout of the runways, taxiways, aprons, and buildings. The outline of the boundary of the airport is superimposed on the map. A plan is prepared from the topographic map, showing the contours of the finished grade and the location of such features as runways, taxiways, aprons, buildings, and roads. This is known as the *drainage working drawing.*

On the plan, the entire surface drainage system is sketched, showing all laterals, submains, and main storm sewers; direction of flow; gradients; and identifying each subarea, catch basin, inlet, gutter, shallow channel, manhole, and peripheral and outfall ditches.

The layout should cover all depressed areas in which overland flow will accumulate. Inlet structures are located at the lowest points within each field area. Each inlet is connected to the drainage line. The pipelines lead to the major outfalls.

Once a layout of inlets, manholes, and storm pipes has been made, determination of the area contributing to each inlet, tabulation of data, and computations of peak flows and drain capacities proceed in exactly the manner described in Section 13.13 for urban storm sewer design. This is illustrated in Example 14.7. Several different drainage layouts are necessary to select the most economic and effective system.

The rate of outflow from a drainage area is controlled by the capacity of the drainpipe. Whenever the rate of runoff to an inlet exceeds the drain capacity, ponding or temporary storage occurs. Where considerable low-lying flat field areas exist away from the pavements, the desirability of using a ponding facility should be considered. This will reduce the size and/or number of drains. Also, this will act as a safety factor in the case of heavier-than-design storms. The volume of storage in ponding is determined by the method of Section 13.14.

EXAMPLE 14.7

A surface (storm) drainage system for a part of an airport is shown in Figure 14.10. The finished contours, drainage layout, and length and slope of drains are marked on the figure. The computed tributary area, the composite runoff coefficient, and the time of overland flow to each intercept are given in Table 14.6. The 5-year rainfall intensity in in./hr is given by $190/(t + 25)$, where t is in minutes. Manning's coefficient $n = 0.015$. Design the drainage system.

SOLUTION

1. The design is performed in exactly the same manner as for the storm design in Section 13.13.
2. Computations for peak flows by the rational method and size of drains are arranged in Tables 14.7 and 14.8, respectively.

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Figure 14.10 Section of an airport.

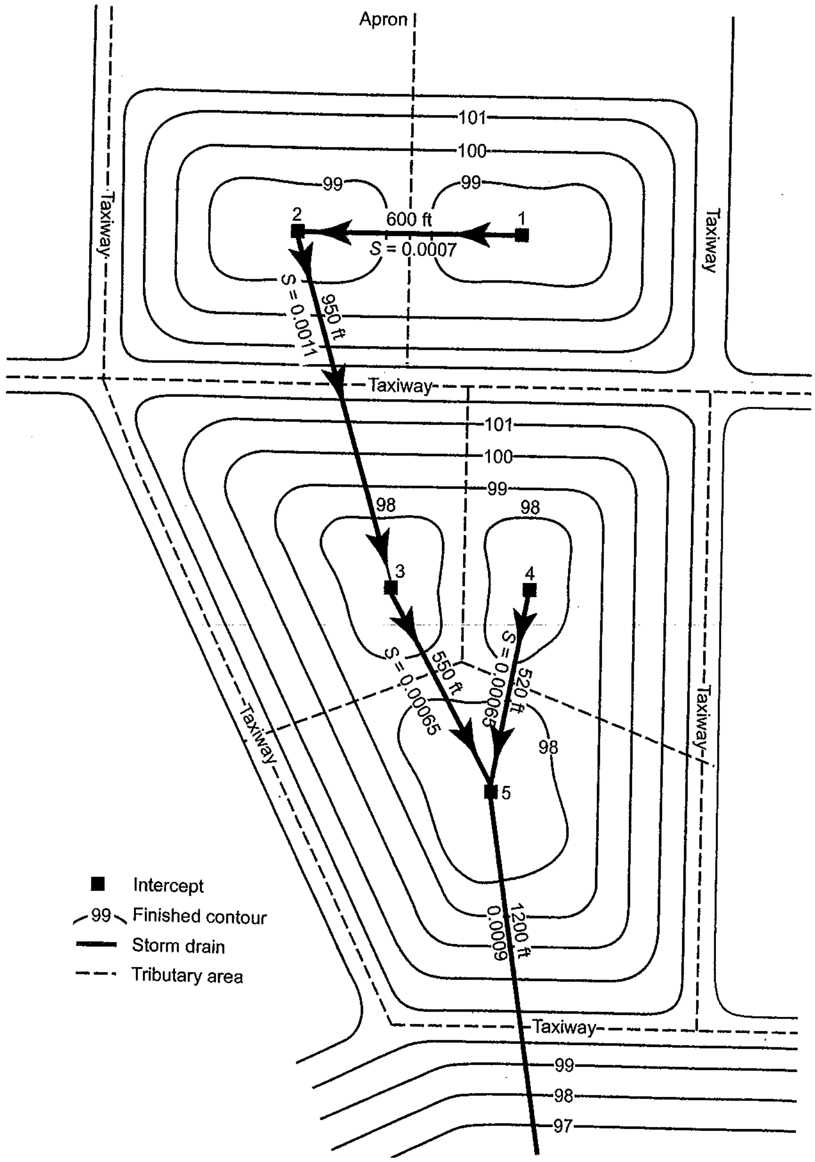

Table 14.6 Drainage Data for Example 14.7

Intercept	Tributary Area (acres)	Weighted Runoff Coefficient	Time of Overland Flow (min)
1	14.2	0.65	25.0
2	16.3	0.65	28.0
3	20.7	0.35	35.0
4	13.5	0.35	35.0
5	25.0	0.35	40.0

Table 14.7 Computation of Peak Discharge

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
						Travel Time (min)					
Intercept	Location	Tributary Area, a (acres)	Coefficient, C	aC (acres)	ΣaC (acres)	Route	Overland Flow	In Sewer[a]	Total	Intensity (in./hr)	Q (cfs)
1		14.2	0.65	9.23	9.23	TA–1	25	0	25	3.8	35.1
2		16.3	0.65	10.60	19.83[b]	TA–2	28	0	28		
						1–2	25	3.18	<u>28.18</u>	3.57	70.8
3		20.7	0.35	7.25	27.08	TA–3	35	0	<u>35</u>	3.17	85.8
						2–3	28.18	4.14	32.32		
4		13.5	0.35	4.73	4.73	TA–4	35	0	35	3.17	15.0
5		25.0	0.35	8.75	40.56[c]	TA–5	40	0	<u>40</u>	2.92	118.4
						4–5	35	2.82	37.82		
						3–5	35	2.85	37.85		

[a] From column 14 of Table 14.8.

[b] Col. 5 for intercept 2 + col. 6 for intercept 1 via route 1–2.

[c] Col. 5 for intercept 5 (TA–5) + (col. 6 for intercept 3 via route 3–5) + (col. 6 for intercept 4 via route 4–5) = 8.75 + 27.08 + 4.73 = 40.56.

Table 14.8 Storm Sewer Design Computations

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	Drain Line				Surface Elevation					Design Parameters			
Line	From	To	Design Flow (cfs)	Length of Sewer (ft)	Upstream	Downstream	Street Slope	Maximum Diameter for Velocity of 3 ft/s[a] (in.)	Diameter for Street Grade[b] (in.)	Diameter (in.)	Sewer[c] Grade	Velocity at Full[d] (ft/sec)	Travel Time (min)[e]
1	1	2	35.1	600			0.0007	46	49	45	0.0011	3.14	3.18
2	2	3	70.8	950			0.0011	66	58	60	0.0011	3.82	4.14
3	3	5	85.8	550			0.00065	72	69	69	0.00065	3.22	2.85
4	4	5	15.0	520			0.00065	30	36	30	0.0018	3.07	2.82
5	5	outlet	118.4	1200			0.0009	85	74	75	0.0008	3.78	5.29

[a] $D=(1.274Q/v)^{1/2}\ \text{ft}\times 12\frac{\text{in.}}{\text{ft.}}$ (continuity eq.)

[b] $D=\left(\frac{2.155nQ}{S^{1/2}}\right)^{0.375}\ \text{ft}\times 12\frac{\text{in.}}{\text{ft.}}$ (Manning's eq.)

[c] Recompute S from Manning's eq. $S=\left[\frac{2.155nQ}{D^{8/3}}\right]^2$, D in ft

[d] $v=\frac{0.59}{n}D^{2/3}S^{1/2}$ (Manning's eq.), D in ft

[e] Col. 5/col. 13$\left(\frac{1}{60}\right)$

14.9 Computer Applications for Drainage

The Hydrology Web of the Pacific Northwest National Laboratory of the U. S. Department of Energy hosts a comprehensive list of links to hydrology and related hydrology resources. Under the computer applications category it maintains a detailed listing of hydrological software, including the programs for stormwater management. The site also provides the supporting resources for software downloading and documentation.

The Surface-Water Quality and Flow Modeling Interest Group (SMIG) of the U.S. Geological Survey maintains an archives of commercial water resources and storm water models. The site provides links to modeling software.

Dodson and Associates have compiled hydrology and hydraulic software in HydroCD, which is a source of 60 stormwater management programs.

The Natural Resources Conservation Service has formulated software based on its TR-55 document. The software has DOS as well as Windows versions. It has a menu-driven interface to calculate the peak flow by the graphical method or to generate a peak inflow hydrograph by the tabular method, as discussed in section 13.12, for small watersheds from 1 to 200 acres.

Haestad Methods offers three salient programs—SewerCAD, StormCAD, and WaterCAD—which they have now combined into a single unified platform. The program provides analysis of peak flows and allows users to size pipes, offset inverts, drop manholes, and design catch basins in addition to analysis and design of water distribution systems and sanitary sewer systems.

The Texas Department of Transportation has created a program, THYSYS, that can analyze storm drain layouts, compute discharges to inlets using the rational method, design inlets, and compute hydraulic grade lines for storm drain lines in a network with up to 100 junctions.

The University of Central Florida has developed the Stormwater Management and Design Aid (SMADA) that allows users to create runoff hydrographs by multiple methods and to perform routing of these hydrographs through ponds, canals, and pipes. The other routines within the program perform optimal sewer design and retention system design.

The EPA's Storm Water Management Model (SWMM) has a diverse array of routines. It can develop runoff hydrographs from rain or snowfall. These hydrographs are for analysis and design of a storm sewer network. SWMM can also estimate the rate of sewage flow from land-use and population statistics and analyze a sanitary sewer network. It can model runoff pollutants and the treatment of sewage through the system. SWMM5 is a completely revised release (November 2004) of SWMM with GIS and CAD interfacing, while SWMM 4.4H has been revised most recently in 2005.

Problems

14.1 The irrigable area covered up to a point by an open drain is 450 acres. The irrigation waste constitutes 25% of the water applied. For canal capacity, the curve of Figure 14.1 applies. The land topography is flat, having a slope of 0.008%. Determine the drain of a semicircular section. $n = 0.03$.

14.2 A subsurface flow of 0.6 cfs drains through a plastic tube drain ($n = 0.011$) into the open drain of Problem 14.1. Design the tube drain and redesign the open drain for a best trapezoidal section when the top width is twice that of the sloping sides. The side slope is 1:1.

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