CHAPTER 3

HYDROLOGY

INTRODUCTION

The hydrological cycle is a continuous process whereby water precipitates from the atmosphere and is transported from ocean and land surfaces back to the atmosphere from which it again precipitates. There are many inter-related phenomena involved in this process as conceptualized in Figure 3.1. Different specialist interests, such as meteorologists, oceanographers or agronomists, focus on different components of the cycle. From the point of view of the drainage engineer, the relevant part of the cycle can be represented in idealistic fashion by the block diagram of Figure 3.2.

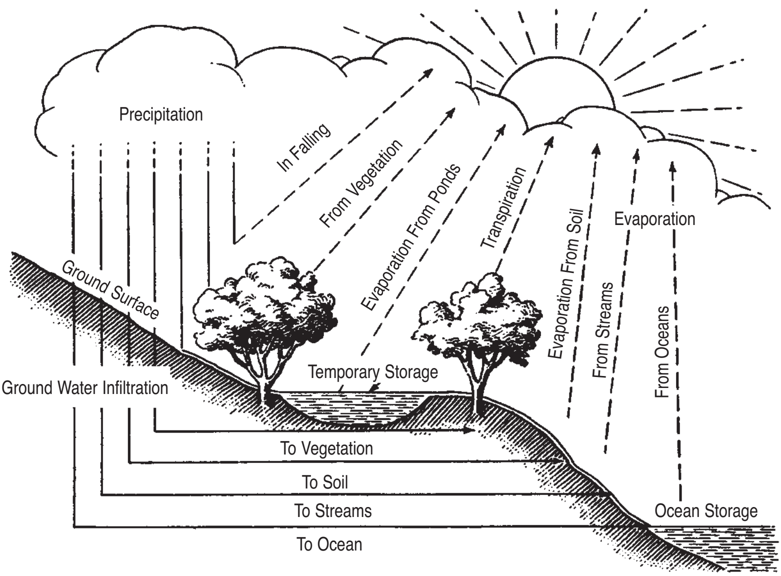


Figure 3.1 Hydrologic Cycle - where water comes from and where it goes. From M.G. Spangler's "Soil Engineering".

Urbanization complicates that part of the hydrologic cycle which is affected by the modifications of natural drainage paths, impounding of water, diversion of storm water and the implementation of storm water management techniques.

The objective of this chapter is to introduce the drainage engineer to the methods of estimating precipitation and runoff; those components of the hydrologic cycle which affect design decisions. Emphasis is placed on the description of alternative methods for analyzing or simulating the rainfall-runoff process, particularly where these apply to computer models. This should help the user of these models in determining appropriate data and in interpreting the results, thereby lessening the "black box" impression with which users are often faced.

It is often necessary to describe many of these processes in mathematical terms. Every effort has been made to keep the presentation simple but some fundamental knowledge of hydrology has been assumed.

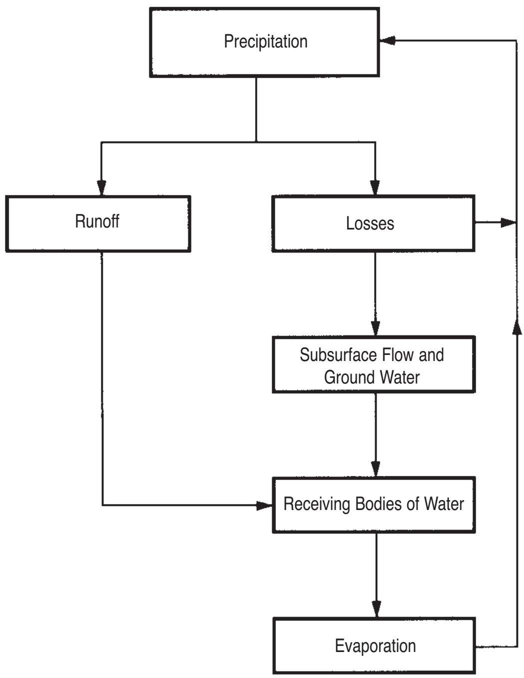


Figure 3.2 Block diagram of Hydrologic Cycle.

ESTIMATION OF RAINFALL

The initial data required for drainage design is a description of the rainfall. In most cases this will be a single event storm, i.e., a period of significant and continuous rainfall preceded and followed by a reasonable length of time during which no rainfall occurs. Continuous rainfall records extending many days or weeks may sometimes be used for the simulation of a series of storms, particularly where the quantity rather than the quality of runoff water is of concern.

The rainfall event may be either historic, taken from recorded events, or idealized. The main parameters of interest are the total amount (or depth) of precipitation (P_{tot}), the duration of the storm (t_d), and the distribution of the rainfall intensity (i) throughout the storm event. The frequency of occurrence (N) of a storm is usually expressed in years and is an estimate based on statistical records of the long-term average time interval which is expected to elapse between successive occurrences of two storms of a particular severity (for example, a storm of depth P_{tot} with a duration of t_d is expected to occur, on average, every N years). The word "expected" is emphasized because there is absolutely no certainty that after a 25-year storm has occurred, a storm of equal or greater severity will not occur for another 25 years. This fact, while statistically true, is often difficult to convey to concerned or affected citizens.

Rainfall Intensity-Duration-Frequency Curves

Rainfall intensity-duration-frequency (IDF) curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a certain return frequency, from a series of storms. These curves are then said to represent storms of a specific return frequency.

The intensity, or the rate of rainfall, is usually expressed in depth per unit time. The frequency of occurrence (N), in years, is a function of the storm intensity. Larger storm intensities occur less frequently. The highest intensity for a specific duration of N years of records is called the N year storm, with a frequency of once in N years.

The curves may be in graphical form as shown in Figure 3.3, or may be represented by individual equations that express the time-intensity relationships for specific frequencies. The formulae are in the form:

$$i = \frac{a}{(t + c)^b}$$

where: i = intensity (mm/hr)
t = time (minutes)
a,b,c = constants developed for each IDF curve

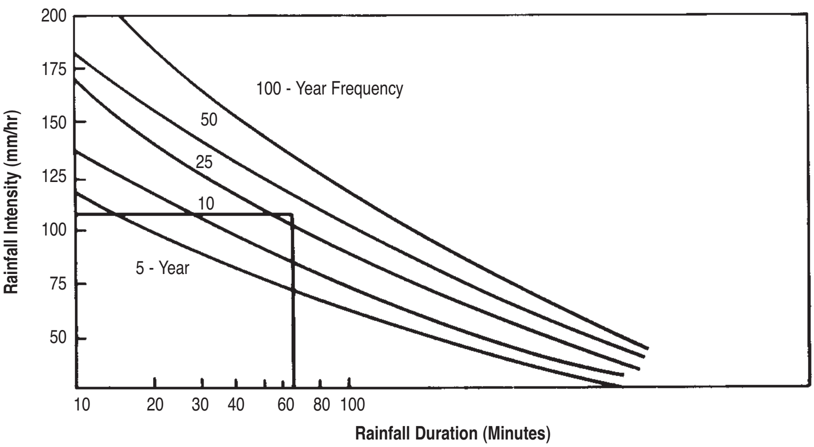


Figure 3.3 Rainfall intensities for various storm frequencies vs rainfall duration.

The fitting of rainfall data to the equation may be obtained by either graphical or least square methods.

It should be noted that the IDF curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of N frequency.

The rainfall intensity-duration-frequency curves are readily available from governmental agencies, local municipalities and towns, and are therefore widely used for designing drainage facilities and flood flow analysis.

Rainfall Hyetographs

The previous section discussed the dependence of the average rainfall intensity of a storm on various factors. It is also important to consider, from historical rainfall events, the way in which the precipitation is distributed in time over the duration of the storm. This can be described using a rainfall hyetograph which is a graphical representation of the variation of rainfall intensity with time. Rainfall hyetographs can be obtained (usually in tabular rather than graphical form) from weather stations which have suitable records of historical rainfall events. Figure 3.4 shows a typical example.

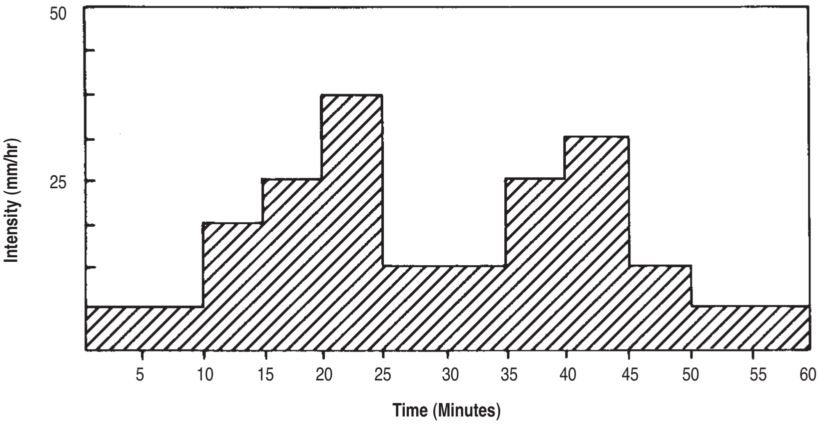
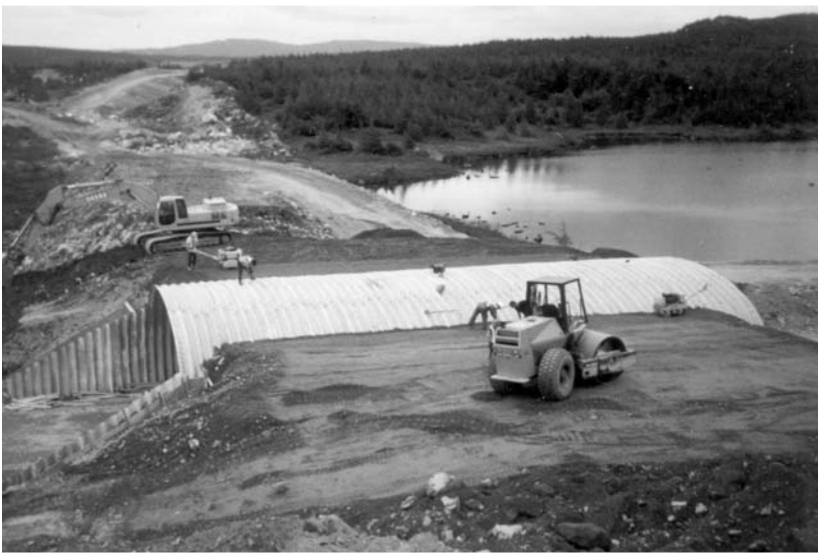


Figure 3.4 Rainfall hyetograph.



Large structure under construction.

Rainfall intensity is usually plotted in the form of a bar graph. It is therefore assumed that the rainfall intensity remains constant over the timestep used to describe the hyetograph. This approximation becomes a truer representation of reality as the timestep gets smaller. However, very small timesteps may require very large amounts of data to represent a storm. At the other extreme, it is essential that the timestep not be too large, especially for short duration events or for very small catchments. Otherwise, peak values of both rainfall and runoff can be "smeared" with consequent loss of accuracy. This point should be kept in mind, when using a computer model, since it is standard practice to employ the same timestep for the description of the rainfall hyetograph and for the computation of the runoff hyetograph. Choice of a timestep is therefore influenced by:

a) accuracy of rainfall-runoff representation,
b) the number of available data points, and
c) size of the watershed.

Synthetic Rainfall Hyetographs

An artificial or idealized hyetograph may be required for a number of reasons, two of which are:

a) The historic rainfall data may not be available for the location or the return frequency desired.
b) It may be desirable to standardize the design storm to be used within a region so that comparisons of results from various studies may be made.

Foundation prepared for large structure.

The time distribution of the selected design hyetograph will significantly affect the timing and magnitude of the peak runoff. Therefore, care should be taken in selecting a design storm to ensure that it is representative of the rainfall patterns in the area under study. In many cases, depending upon the size of the watershed and degree of urbanization, it may be necessary to use several different rainfall hyetographs to determine the sensitivity of the results to the different design storms. For example, when runoff from pervious areas is significant, it will be found that late peaking storms produce a higher peak runoff than early peaking storms of the same total depth. Early peaking storms are reduced in severity by the initially high infiltration capacity of the ground.

Selection of the storm duration will also influence the hyetograph characteristics. The handbook of the Natural Resource Conservation Service (formerly Soil Conservation Service) recommends that a six hour storm duration be used for watersheds with a time of concentration (which is discussed in detail later in this chapter) less than or equal to six hours. For watersheds where the time of concentration exceeds six hours, the storm duration should equal the time of concentration.

A number of different synthetic hyetographs are described in the following sections. These include:

a) uniform rainfall (as in the Rational Method),
b) the Chicago hyetograph,
c) the SCS design storms, and
d) Huff's storm distribution patterns.

Uniform Rainfall

The simplest possible design storm is to assume that the intensity is uniformly distributed throughout the storm duration. The intensity is then represented by the formula:

$$i = i_{ave} = \frac{P_{tot}}{t_d}$$

where: P_{tot} = total precipitation
t_d = storm duration

This simplified approximation is used in the Rational Method assuming that the storm duration, t_d, is equal to the time of concentration, t_c, of the catchment (see Figure 3.5). A rectangular rainfall distribution is only used for approximations or rough estimates. It can, however, have some use in explaining or visualizing rainfall runoff processes since any hyetograph may be considered as a series of such uniform, short duration pulses of rainfall.

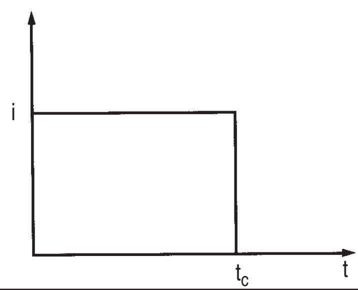


Figure 3.5 Uniform rainfall intensity.

The Chicago Hyetograph

The Chicago hyetograph is assumed to have a time distribution such that if a series of ever increasing "time-slices" were analyzed around the peak rainfall, the average intensity for each "slice" would lie on a single IDF curve. Therefore, the Chicago design storm displays statistical properties which are consistent with the statistics of the IDF curve. That being the case, the synthesis of the Chicago hyetograph starts with the parameters of an IDF curve together with a parameter (r) which defines the fraction of the storm duration which occurs before the peak rainfall intensity. The value of r is derived from the analysis of actual rainfall events and is generally in the range of 0.3 to 0.5.

The continuous curves of the hyetograph in Figure 3.6 can be computed in terms of the times before (t_b) and after (t_a) the peak intensity by the two equations below.

After the peak:

$$i_a = \frac{a\left[(1-b)\dfrac{t_a}{1-r} + c\right]}{\left(\dfrac{t_a}{1-r} + c\right)^{1+b}}$$

Before the peak:

$$i_b = \frac{a\left[(1-b)\dfrac{t_b}{r} + c\right]}{\left(\dfrac{t_b}{r} + c\right)^{1+b}}$$

where: t_a = time after peak
t_b = time before peak
r = ratio of time before the peak occurs to the total duration time (the value is derived from analysis of actual rainfall events)

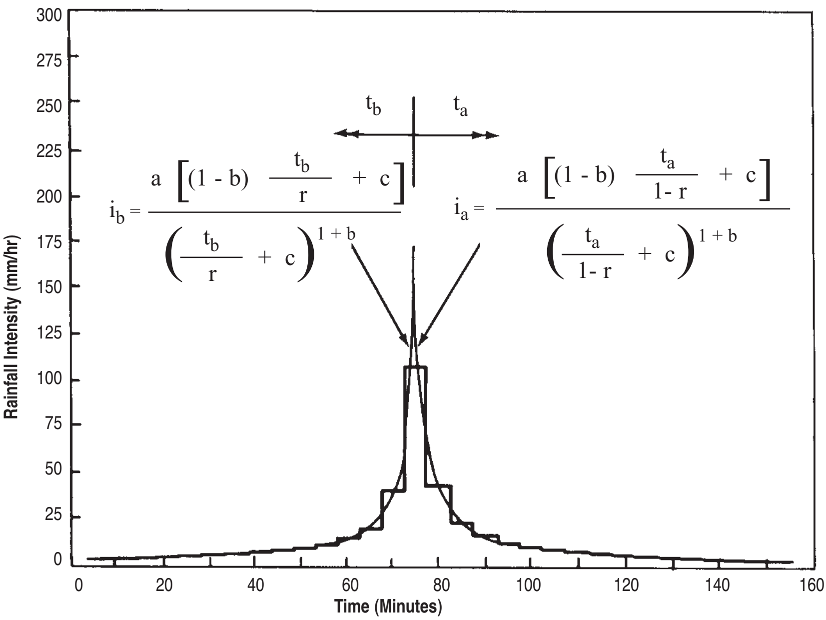


Figure 3.6 Chicago hyetograph.

CSP for storm drainage project.

Detention tank with internal baffle for sediment and debris control. (Ministry of Transportation, Ontario)

The Chicago storm is commonly used for small to medium watersheds (0.25 km^2 to 25 km^2) for both rural and urban conditions. Typical storm durations are in the range of 1.0 to 4.0 hours. It has been found that peak runoff flows computed using a Chicago design storm are higher than those obtained using other synthetic or historic storms. This is due to the fact that the Chicago storm attempts to model the statistics of a large collection of real storms and thus tends to present an unrealistically extreme distribution. Also, the resultant peak runoff may exhibit some sensitivity to the time step used; very small timesteps give rise to more peaked runoff hydrographs (which are discussed later in this chapter).

The Huff Rainfall Distribution Curves

Huff analyzed the significant storms in 11 years of rainfall data recorded by the State of Illinois. The data was represented in non-dimensional form by expressing the accumulated depth of precipitation, P_t, (that is, the accumulated depth at time t after the start of rainfall) as a fraction of the total storm depth, P_{tot}, and plotting this ratio as a function of a non-dimensional time, t/t_d, where t_d is time of duration.

The storms were grouped into four categories depending on whether the peak rainfall intensity fell in the 1st, 2nd, 3rd or 4th quartile of the storm duration. In each category, a family of curves was developed representing values exceeded in 90%, 80%, 70%, etc., of the storm events. Thus the average of all the storm events in a particular category is represented by the 50% curve. Table 3.1 shows the dimensionless coefficients for each quartile expressed at intervals of 5% of t_d.

Table 3.1 Dimensionless Huff storm coefficients

t/t_d	P_t/P_{tot} for Quartile			
	1	2	3	4
0.00	0.000	0.000	0.000	0.000
0.05	0.063	0.015	0.020	0.020
0.10	0.178	0.031	0.040	0.040
0.15	0.333	0.070	0.072	0.055
0.20	0.500	0.125	0.100	0.070
0.25	0.620	0.208	0.122	0.085
0.30	0.705	0.305	0.140	0.100
0.35	0.760	0.420	0.155	0.115
0.40	0.798	0.525	0.180	0.135
0.45	0.830	0.630	0.215	0.155
0.50	0.855	0.725	0.280	0.185
0.55	0.880	0.805	0.395	0.215
0.60	0.898	0.860	0.535	0.245
0.65	0.915	0.900	0.690	0.290
0.70	0.930	0.930	0.790	0.350
0.75	0.944	0.948	0.875	0.435
0.80	0.958	0.962	0.935	0.545
0.85	0.971	0.974	0.965	0.740
0.90	0.983	0.985	0.985	0.920
0.95	0.994	0.993	0.995	0.975
1.00	1.000	1.000	1.000	1.000

The first quartile curve is generally associated with relatively short duration storms in which 62% of the precipitation depth occurs in the first quarter of the storm duration. The fourth quartile curve is normally used for longer duration storms in which the rainfall is more evenly distributed over the duration t_d and is often dominated by a series of rain showers or steady rain or a combination of both. The third quartile has been found to be suitable for storms on the Pacific seaboard.

The study area and storm duration for which the distributions were developed vary considerably, with t_d varying from 3 to 48 hours and the drainage basin area ranging from 25 to 1000 km^2. The distributions are most applicable to Midwestern regions of North America and regions of similar rainfall climatology and physiography.

To use the Huff distribution the user need only specify the total depth of rainfall (P_{tot}), the duration (t_d) and the desired quartile. The curve can then be scaled up to a dimensional mass curve and the intensities are obtained from the mass curve for the specified timestep (t).

SCS Storm Distributions

The U.S. Soil Conservation Service (SCS) design storm was developed for various storm types, storm durations and regions of the United States. The storm duration was initially selected to be 6 hours. Durations of 3 hours and up to 48 hours have, however, been developed. The rainfall distribution varies depending on duration and location. The 3, 6, 12 and 24 hour distributions for the SCS Type II storm are given in Table 3.2. These distributions are used in all regions of the United States and Canada with the exception of the Pacific coast.

Table 3.2 **SCS Type II rainfall distribution for 3h,6h,12h and 24h durations**

3 Hour			6 Hour			12 Hour			24 Hour		
Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)
						0.5	1	1			
			0.5	2	2	1.0	1	2	2	2	2
						1.5	1	3			
0.5	4	4	1.0	2	4.0	2.0	1	4	4	2	4
						2.5	2	6			
			1.5	4	8	3.0	2	8	6	4	8
						3.5	2	10			
1.0	8	12	2.0	4	12	4.0	2	12	8	4	12
						4.5	3	15			
			2.5	7	19	5.0	4	19	10	7	19
						5.5	6	25			
1.5	58	70	3.0	51	70	6.0	45	70	12	51	70
						6.5	9	79			
			3.5	13	83	7.0	4	83	14	13	83
						7.5	3	86			
2.0	19	89	4.0	6	89	8.0	3	89	16	6	89
						8.5	2	91			
			4.5	4	93	9.0	2	93	18	4	93
						9.5	2	95			
2.5	7	96	5.0	3	96	10.0	1	96	20	3	96
						10.5	1	97			
			5.5	2	98	11.0	1	98	22	2	98
						11.5	1	99			
3.0	4	100	6.0	2	100	12.0	1	100	24	2	100

The design storms were initially developed for large (25 km^2) rural basins. However, the longer duration (6 to 48 hour) distributions and a shorter 1 hour duration thunderstorm distribution have been used in urban and smaller rural areas.

The longer duration storms tend to be used for sizing detention facilities while at the same time providing a reasonable peak flow for sizing the conveyance system.

ESTIMATION OF EFFECTIVE RAINFALL

Only a fraction of the precipitation which falls during a storm contributes to the overland flow or runoff from the catchment. The balance is diverted in various ways.

Evaporation — In certain climates, some fraction of the rainfall evaporates before reaching the ground. Since rainfall is measured by gauges on the earth's surface, this subtraction is automatically taken into account in recorded storms and may be ignored by the drainage engineer.

Interception — This fraction is trapped in vegetation or roof depressions and never reaches the catchment surface. It eventually dissipates by evaporation.

Infiltration — Rainfall which reaches a pervious area of the ground surface will initially be used to satisfy the capacity for infiltration into the upper layer of the soil. After even quite a short dry period the infiltration capacity can be quite large (for example, 100 mm/hr) but this gradually diminishes after the start of rainfall as the storage capacity of the ground is saturated. The infiltrated water will:

a) evaporate directly by capillary rise,
b) rise through the root system and be transpired from vegetal cover, where it then evaporates,
c) move laterally through the soil in the form of ground water flow toward a lake or a stream, and/or
d) penetrate to deeper levels to recharge the ground water.

Surface — If the intensity of the rainfall reaching the ground exceeds the infiltration capacity of the ground, the excess will begin to fill the small depressions on the ground surface. Clearly this will begin to happen almost immediately on impervious surfaces. Only after these tiny reservoirs have been filled will overland flow commence and contribute to the runoff from the catchment. Since these surface depressions are not uniformly distributed, it is quite possible that runoff will commence from some fraction of the catchment area before the depression storage on another fraction is completely filled. Typical recommended values for surface depression storage are given in Table 3.3.

Table 3.3 Typical recommended values for depth of surface depression storage

Land Cover	Recommended Value (mm)
Large Paved Areas	2.5
Roofs, Flat	2.5
Fallow Land Field without Crops	5.0
Fields with Crops (grain, root crops)	7.5
Grass Areas in Parks, Lawns	7.5
Wooded Areas and Open Fields	10.0

The effective rainfall is thus that portion of the storm rainfall which contributes directly to the surface runoff hydrograph. This can be expressed as follows:

Runoff = Precipitation - Interception - Infiltration - Surface Depression Storage

All of the terms are expressed in units of depth.

A number of methods are available to estimate the effective rainfall and thus the amount of runoff for any particular storm event. These range from the runoff coefficient (C) of the Rational Method to relatively sophisticated computer implementations of semi-empirical methods representing the physical processes. The method selected should be based on the size of the drainage area, the data available, and the degree of sophistication warranted for the design. Three methods for estimating effective rainfall are:

1) the Rational Method,
2) the Soil Conservation Service (SCS) Method, and
3) the Horton Method.

The Rational Method

If an impervious area (A) is subjected to continuous and long lasting rainfall of a specific intensity (i), then after a time (time of concentration, T_c) the runoff rate will be given by the equation:

$$Q = k \cdot C \cdot i \cdot A$$

where: Q = peak runoff rate (m^3/s)
k = constant = 0.00278
C = runoff coefficient
i = rainfall intensity (mm/hr)
A = drainage area (hectares)

When using the Rational Method, the following assumptions are considered:

a) The rainfall intensity is uniform over the entire watershed during the entire storm duration.
b) The maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration.
c) The time of concentration is the time required for the runoff from the most remote part of the watershed to reach the point under design.

The variable C is the component of the Rational Method formula that requires the most judgement, and the runoff is directly proportional to the value assigned to C. Care should be exercised in selecting the value as it incorporates all of the hydrologic abstractions, soil types and antecedent conditions. Table 3.4 lists typical values for C, as a function of land use, for storms that have (approximately) a 5 to 10 year return period. It is important to note that the appropriate value of C depends on the magnitude of the storm and significantly higher values of C may be necessary for more extreme storm events. This is perhaps one of the most serious deficiencies associated with this method.

Table 3.4 Recommended runoff coefficients based on description of area

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighbourhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

High profile arch completed assembly.

It often is desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area. Coefficients, with respect to surface type, are shown in Table 3.5.

Table 3.5 Recommended runoff coefficients based on character of surface

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.75 to 0.95
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tables are applicable for storms of 5- to 10-year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

Pipe-arch with manhole riser, inlet pipe and reinforced bulkhead.

SCS Method

Referred to here as the SCS Method, the Natural Resource Conservation Service (formerly Soil Conservation Service) developed a relationship between rainfall (P), retention (S), and effective rainfall or runoff[5] (Q). The retention, or potential storage in the soil, is established by selecting a curve number (CN). The curve number is a function of soil type, ground cover and Antecedent Moisture Condition (AMC).

The hydrological soil groups, as defined by SCS soil scientists, are:

A. (Low runoff potential) Soils having a high infiltration rate, even when thoroughly wetted, consisting chiefly of deep, well to excessively well drained sands or gravel.
B. Soils having a moderate infiltration rate when thoroughly wetted, consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse texture.
C. Soils having a slow infiltration rate when thoroughly wetted, consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture.
D. (High runoff potential) Soils having a very slow infiltration rate when thoroughly wetted, consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Knowing the hydrological soil group and the corresponding land use, the runoff potential or CN value of a site may be determined. Table 3.6 lists typical CN values.

Table 3.6 **Runoff curve number for selected agricultural, suburban and urban land use (Antecedent Moisture Condition II and I_a = 0.2 S)**

LAND USE DESCRIPTION			HYDROLOGIC SOIL GROUP A	B	C	D
Cultivated land[1]:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow:	good condition		30	58	71	78
Wood or forest land:	thin stand, poor cover, no mulch		45	66	77	83
	good cover[2]		25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.						
good condition:	grass cover on 75% or more of the area		39	61	74	80
fair condition:	grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)			89	92	94	95
Industrial districts (72% impervious)			81	88	91	93
Residential[3]:						
	Average lot size	Average % Impervious[4]				
	0.05 hectare or less	65	77	85	90	92
	0.10 hectare	38	61	75	83	87
	0.15 hectare	30	57	72	81	86
	0.20 hectare	25	54	70	80	85
	0.40 hectare	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]			98	98	98	98
Streets and roads:	paved with curbs and storm sewers[5]		98	98	98	98
	gravel		76	85	89	91
	dirt		72	82	87	89

1. For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug 1972.
2. Good cover is protected from grazing and litter and brush cover soil.
3. Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
4. The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
5. In some warmer climates of the country a curve number of 95 may be used.

Three levels of Antecedent Moisture Condition are considered in the SCS Method. The Antecedent Moisture Condition (AMC) is defined as the amount of rainfall in a period of five to thirty days preceding the design storm. In general, the heavier the antecedent rainfall, the greater the runoff potential. AMC definitions are as follows:

- AMC I - Soils are dry but not to the wilting point. This is the lowest runoff potential.
- AMC II - This is the average case, where the soil moisture condition is considered to be average.
- AMC III - Heavy or light rainfall and low temperatures having occurred during the previous five days. This is the highest runoff potential.

The CN values in Table 3.6 are based on Antecedent Moisture Condition II. Thus, if moisture conditions I or III are chosen, then a corresponding CN value is determined as provided in Table 3.7.

Table 3.7 **Curve number relationship for different antecedent moisture conditions**

CN for	CN for		CN for	CN for	
Condition II	Condition I	Condition III	Condition II	Condition I	Condition III
100	100	100	60	40	78
99	97	100	59	39	77
98	94	99	58	38	76
97	91	99	57	37	75
96	89	99	56	56	75
95	87	98	55	35	74
94	85	98	54	34	73
93	83	98	53	33	72
92	81	97	52	32	71
91	80	97	51	31	70
90	78	96	50	31	70
89	76	96	49	30	69
88	75	95	48	29	68
87	73	95	47	28	67
86	72	94	46	27	66
85	70	94	45	26	65
84	68	93	44	25	64
83	67	93	43	25	63
82	66	92	42	24	62
81	64	92	41	23	61
80	63	91	40	22	60
79	62	91	39	21	59
78	60	90	38	21	58
77	59	89	37	20	57
76	58	89	36	19	56
75	57	88	35	18	55
74	55	88	34	18	54
73	54	87	33	17	53
72	53	86	32	16	52
71	52	86	31	16	51
70	51	85	30	15	50
69	50	84			
68	48	84	25	12	43
67	47	83	20	9	37
66	46	82	15	6	30
65	45	82	10	4	22
64	44	81	5	2	13
63	43	80	0	0	0
62	42	79			
61	41	78			

The potential storage in the soils is based on an initial abstraction (I_a) which is the interception, infiltration and depression storage prior to runoff, and infiltration after runoff.

The effective rainfall is defined by the relationship:

$$Q = \frac{(P - I_a)^2}{P + S - I_a}$$

where: $S = [(100/CN) - 10] \cdot 25.4$

The original SCS Method assumed the value of I_a to be equal to 0.2S. However, many engineers have found that this may be overly conservative, especially for moderate rainfall events and low CN values. Under these conditions, the I_a value may be reduced to be a lesser percentage of S or may be estimated and input directly into the above equation.

The Horton Method

The Horton infiltration equation, which defines the infiltration capacity of the soil, changes the initial rate (f_o) to a lower rate (f_c). The infiltration capacity is an upper bound and is realized only when the available rainfall equals or exceeds the infiltration capacity. Therefore, if the infiltration capacity is given by:

$$f_{cap} = f_c + (f_o - f_c)\, e^{-t \cdot k}$$

then the actual infiltration (f), will be defined by one of the following two equations:

$$f = f_{cap} \quad \text{for } i \geq f_{cap}$$

$$f = i \quad \text{for } i \leq f_{cap}$$

where:
- f = actual infiltration rate into the soil
- f_{cap} = maximum infiltration capacity of the soil
- f_o = initial infiltration capacity
- f_c = final infiltration capacity
- i = rainfall intensity
- k = exponential decay constant (1/hours)
- t = elapsed time from start of rainfall (hours)

Figure 3.7 shows a typical rainfall distribution and infiltration curve.

For the initial timesteps the infiltration rate exceeds the rainfall rate. The reduction in infiltration capacity is dependent more on the reduction in storage capacity in the soil rather than the elapsed time from the start of rainfall. To account for this the infiltration curve should, therefore, be shifted (dashed line for first timestep, Δt) by an elapsed time that would equate the infiltration volume to the volume of runoff.

A further modification is necessary if surface depression is to be accounted for. Since the storage depth must be satisfied before overland flow can occur, the initial finite values of the effective rainfall hyetograph must be reduced to zero until a depth equivalent to the surface depression storage has been accumulated. The final hyetograph is the true effective rainfall which will generate runoff from the catchment surface.

CSP with rodent grate.

Joints wrapped with geotextile to prevent migration of fines into the pipes.

The selection of the parameters for the Horton equation depends on soil type, vegetal cover and antecedent moisture conditions. Table 3.8 shows typical values for f_o and f_c (mm/hour) for a variety of soil types under different crop conditions. The value of the lag constant should typically be between 0.04 and 0.08.

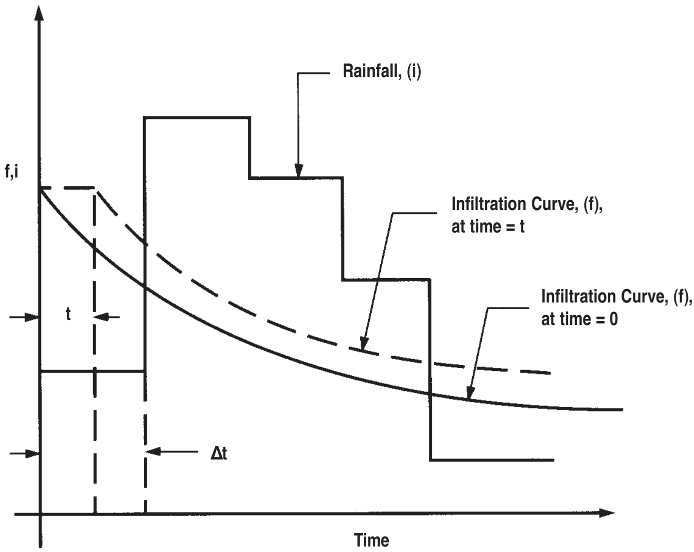


Figure 3.7 Representation of the Horton equation.

Table 3.8 Typical values for the Horton equation parameters (mm/hr)

	Loam, Clay K = 0.08		Clayey Sand K = 0.06		Sand, Loess, Gravel K = 0.04	
Land Surface Types	f_o	f_c	f_o	f_c	f_o	f_c
Fallow land field without crops	15	8	33	10	43	15
Fields with crops (grain, root crops, vines)	36	3	43	8	64	10
Grassed verges, playground, ski slopes	20	3	20	3	20	3
Noncompacted grassy surface, grass areas in parks, lawns	43	8	64	10	89	18
Gardens, meadows, pastures	64	10	71	15	89	18
Coniferous woods	53*	53*	71*	71*	89*	89*
City parks, woodland, orchards	89	53	89	71	89*	89*

*K=0

Comparison of the SCS and Horton Methods

Figure 3.8 illustrates the various components of the rainfall runoff process for the SCS and Horton Methods. The following example serves to demonstrate the difference between the SCS Method, in which the initial abstraction is used, and the moving curve Horton Method, in which surface depression storage is significant. The incident storm is assumed to be represented by a second quartile Huff curve with a total depth of 50 mm and a duration of 120 minutes. In one case the SCS Method is used with the initial abstraction set at an absolute value of I_a = 6.1 mm. The curve number used is 87.6. Figure 3.9 shows that no runoff occurs until approximately 30 minutes have elapsed at which time the rainfall has satisfied the initial abstraction. From that point, however, the runoff, although small, is finite and continues to be so right to the end of the storm.

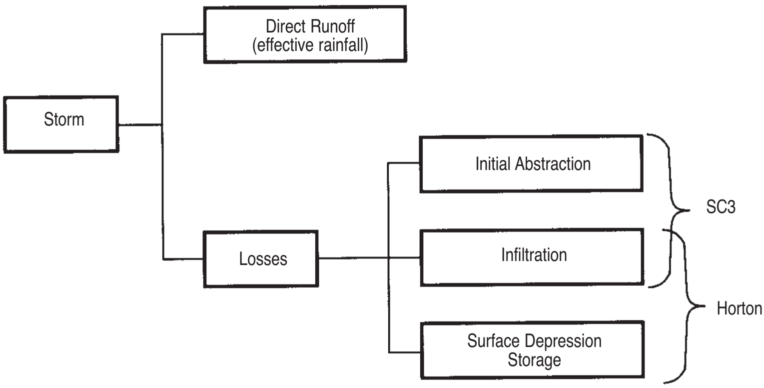


Figure 3.8 Conceptual components of rainfall.

The Horton case is tested using values of f_o = 30 mm/hr, f_c = 10 mm/hr, K = 0.25 hour, and a surface depression storage depth of 5 mm.

These values have been found to give the same volumetric runoff coefficient as the SCS parameters. Figure 3.10 shows that infiltration commences immediately and absorbs all of the rainfall until approximately 30 minutes have elapsed. The initial excess surface water has to fill the surface depression storage which delays the commencement of runoff for a further 13 minutes. After 72 minutes the rainfall intensity is less than f_c and runoff is effectively stopped at that time.

It will be found that the effective rainfall hyetograph generated using the Horton Method has more leading and trailing "zero" elements so that the effective hyetograph is shorter but more intense than that produced using the SCS Method.

ESTABLISHING THE TIME OF CONCENTRATION

Apart from the area and the percentage of impervious surface, one of the most important characteristics of a catchment is the time which must elapse until the entire area is contributing to runoff at the outflow point. This is generally called the Time of Concentration (T_c). This time is comprised of two components:

1) The time for overland flow to occur from a point on the perimeter of the catchment to a natural or artificial drainage conduit or channel.
2) The travel time in the conduit or channel to the outflow point of the catchment.

In storm sewer design, the time of concentration may be defined as the inlet time plus travel time. Inlet times used in sewer design generally vary from 5 to 20 minutes, with the channel flow time being determined from pipe flow equations.

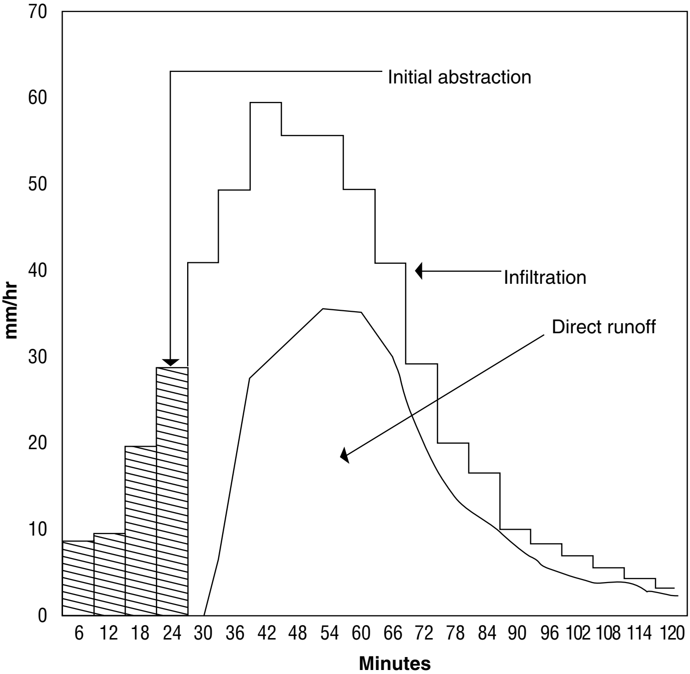


Figure 3.9 SCS Method with I_a = 6.1 mm and CN = 87.6

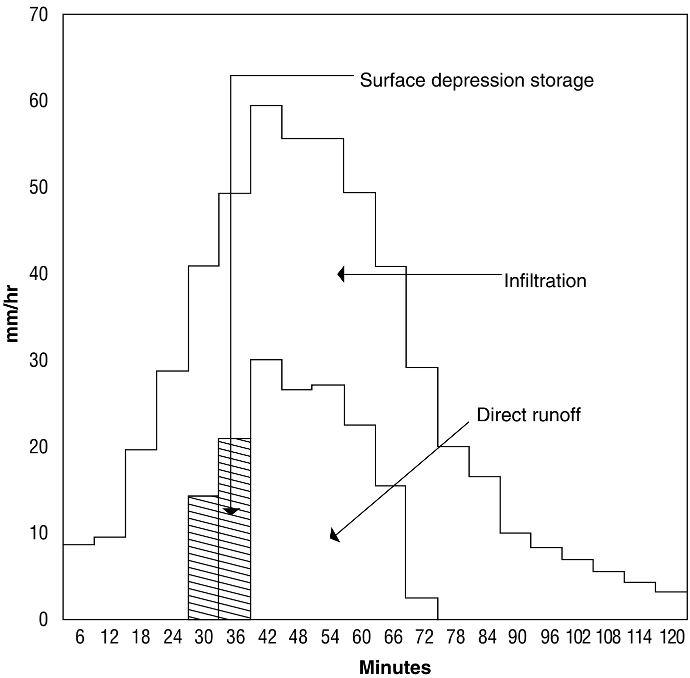


Figure 3.10 Horton equation with f_o = 30 mm, f_c = 10 mm, K = 0.25, and surface depression storage = 5 mm

Factors Affecting Time of Concentration

The time taken for overland flow to reach a conduit or channel depends on a number of factors:

a) Overland flow length (L). This should be measured along the line of longest slope from the extremity of the catchment to a drainage conduit or channel. Long lengths result in long travel times.
b) Average surface slope (S). Since T_c is inversely proportional to S, care must be exercised in estimating an average value for the surface slope.
c) Surface roughness. In general, rough surfaces result in longer travel times and smooth surfaces result in shorter travel times. Therefore, if a Manning equation is used to estimate the velocity of overland flow, T_c will be proportional to the Manning roughness factor (n).
d) Depth of overland flow (y). Very shallow surface flows move more slowly than deeper flows. However, the depth of flow is not a characteristic of the catchment alone but depends on the intensity of the effective rainfall and surface moisture excess.

Several methods of estimating the Time of Concentration are described below. Since it is clear that this parameter has a strong influence on the shape of the runoff hydrograph, it is desirable to compare the value to that obtained from observation, if possible. In situations where sufficient historical data is not available, it may help to compare the results obtained by two or more methods. The impact on the resultant hydrograph, due to using different methods for establishing the time of concentration, should then be assessed.

The Kirpich Formula

This empirical formula relates T_c to the length and average slope of the basin by the equation:

$$T_c = 0.00032\ L^{0.77}\ S^{-0.385} \text{ (See Figure 3.11)}$$

where: T_c = time of concentration (hours)
L = maximum length of water travel (m)
S = surface slope, given by H/L (m/m)
H = difference in elevation between the most remote point on the basin and the outlet (m)

From the definition of L and S it is clear that the Kirpich equation combines both the overland flow, or entry time, and the travel time in the channel or conduit. It is, therefore, particularly important that in estimating the drop (H), the slope (S) and ultimately the time of concentration (T_c), an allowance, if applicable, be made for the inlet travel time.

The Kirpich equation is normally used for natural basins with well defined routes for overland flow along bare earth or mowed grass roadside channels. For overland flow on grassed surfaces, the value of T_c obtained should be doubled. For overland flow in concrete channels, a multiplier of 0.2 should be used.

For large watersheds, where the storage capacity of the basin is significant, the Kirpich formula tends to significantly underestimate T_c.

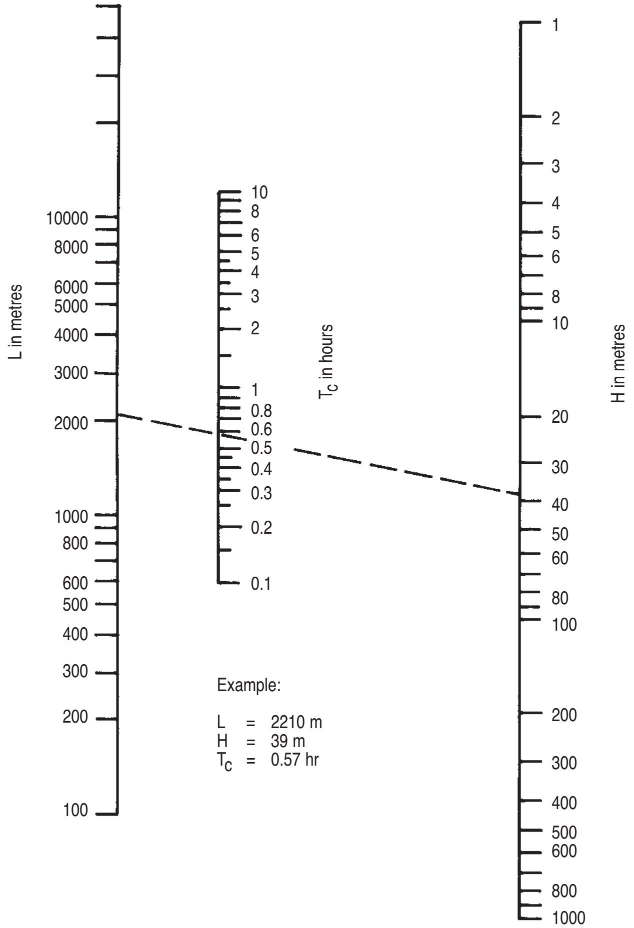


Figure 3.11 T_C nomograph using the Kirpich formula.

The Uplands Method

When calculating travel times for overland flow in watersheds with a variety of land covers, the Uplands Method may be used. This method relates the time of concentration to the basin slope, basin length and type of ground cover. Times are calculated for individual areas, with their summation giving the total travel time.

A velocity is derived using the $V/S^{0.5}$ values from Table 3.9 and a known slope. The time of concentration is obtained by dividing the length by the velocity.

A graphical solution can be obtained from Figure 3.12. However, it should be noted that the graph is simply a log-log plot of the values of $V/S^{0.5}$ given in Table 3.9.

Table 3.9 $V/S^{0.5}$ relationship for various land covers

Land Cover	$V/S^{0.5}$ (m/s)
Forest with heavy ground litter, hay meadow (overland flow)	0.6
Trash fallow or minimum tillage cultivation, contour, strip cropped woodland (overland flow)	1.5
Short grass pasture (overland flow)	2.3
Cultivated, straight row (overland flow)	2.7
Nearly bare and untilled (overland flow) or alluvial fans in Western mountain regions	3.0
Grassed waterway	4.6
Paved areas (sheet flow); small upland gullies	6.1

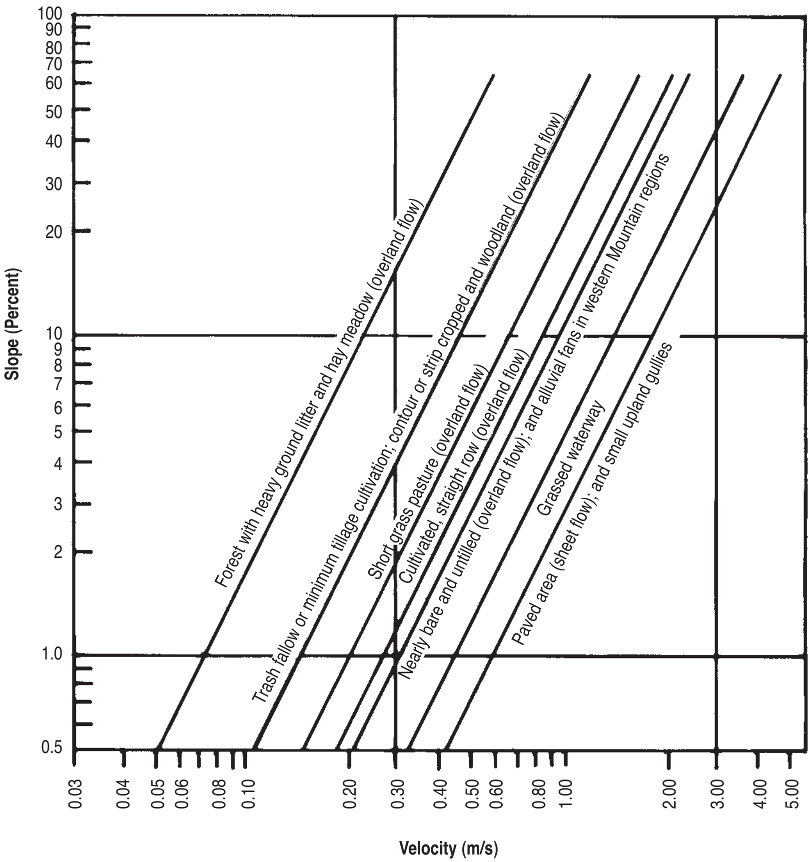


Figure 3.12 Velocities for Upland method for estimating travel time for overland flow.

The Kinematic Wave Method

The two methods described above have the advantage of being quite straightforward and may be used for either simple or more complex methods of determining the runoff. Apart from the empirical nature of the equations, the methods assume that the time of concentration is independent of the depth of overland flow, or more generally, the magnitude of the input. A method in common use, which is more physically based and which also reflects the dependence of T_c on the intensity of the effective rainfall, is the Kinematic Wave Method.

The method was proposed by Henderson to analyze the kinematic wave resulting from rainfall of uniform intensity on an impermeable plane surface or rectangular area. The resulting equation is as follows:

$$T_C = 0.116\,(L \cdot n/S)^{0.6}\, i_{eff}^{-0.4}$$

Where: T_c = time of concentration (hr)
L = length of overland flow (m)
n = Manning's roughness coefficient
S = average slope of overland flow (m/m)
i_{eff} = effective rainfall intensity (mm/hr)

Other Methods

Other methods have been developed which determine T_c for specific geographic regions or basin types. These methods are often incorporated into an overall procedure for determining the runoff hydrograph. Before using any method the user should ensure that the basis on which the time of concentration is determined is appropriate for the area under consideration.

DETERMINATION OF THE RUNOFF HYDROGRAPH

The following sections outline alternative methods for generating the runoff hydrograph, which is the relationship of discharge over time. Emphasis will be given to establishing the hydrograph for single storm events. Methods for estimating flow for urban and rural conditions are given.

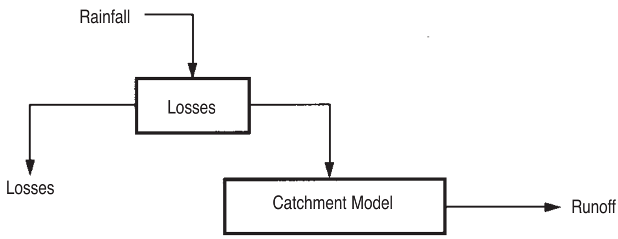
Irrespective of the method used, the results should be compared to historical values wherever possible. In many cases, a calibration/validation exercise will aid in the selection of the most appropriate method.

All of the methods described could be carried out using hand calculations. However, for all but the simplest cases the exercise would be very laborious. Furthermore, access to computers and computer models is very economical. For these reasons emphasis will be placed on describing the basis of each method and the relevant parameters. A subsequent section will relate the methods to several computer models.

Rainfall runoff models may be grouped into two general classifications, which are illustrated in Figure 3.13.

One approach uses the concept of effective rainfall, in which a loss model is assumed which divides the rainfall intensity into losses (initial infiltration and depression storage) and effective rainfall. The effective rainfall hyetograph is then used as input to a catchment model to produce a runoff hydrograph. It follows from this approach that infiltration must stop at the end of the storm.

The alternative approach employs a surface water budget in which the infiltration or loss mechanism is incorporated into the catchment model. In this method, the storm rainfall is used as input and the estimation of infiltration and other losses is an integral part of the runoff calculation. This approach implies that infiltration will continue as long as there is excess water on the surface. Clearly, this may continue after the rainfall ends.



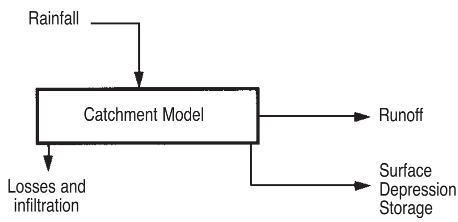


Figure 3.13 Classification of rainfall-runoff models: Effective Rainfall (top) and Surface Water Budget (bottom).

SCS Unit Hydrograph Method

A unit hydrograph represents the runoff distribution over time for one unit of rainfall excess over a drainage area for a specified period of time. This method assumes that the ordinates of flow are proportional to the volume of runoff from any storm of the same duration. Therefore, it is possible to derive unit hydrographs for various rainfall blocks by convoluting the unit hydrograph with the effective rainfall distribution. The unit hydrograph theory is based on the following assumptions:

1 For a given watershed, runoff-producing storms of equal duration will produce surface runoff hydrographs with approximately equivalent time bases, regardless of the intensity of the rain.
2 For a given watershed, the magnitude of the ordinates representing the instantaneous discharge from an area will be proportional to the volumes of surface runoff produced by storms of equal duration.
3 For a given watershed, the time distribution of runoff from a given storm period is independent of precipitation from antecedent or subsequent storm periods.

The U.S. Natural Resource Conservation Service (formerly Soil Conservation Service), based on the analysis of a large number of hydrographs, proposed a unit hydrograph which only requires an estimate of the time to peak (t_p). Two versions of this unit hydrograph were suggested; one being curvilinear in shape, while the other is a simple asymmetric triangle as shown in Figure 3.14. The SCS has indicated that the two hydrographs give very similar results as long as the time increment is not greater than 0.20 $\cdot T_c$.

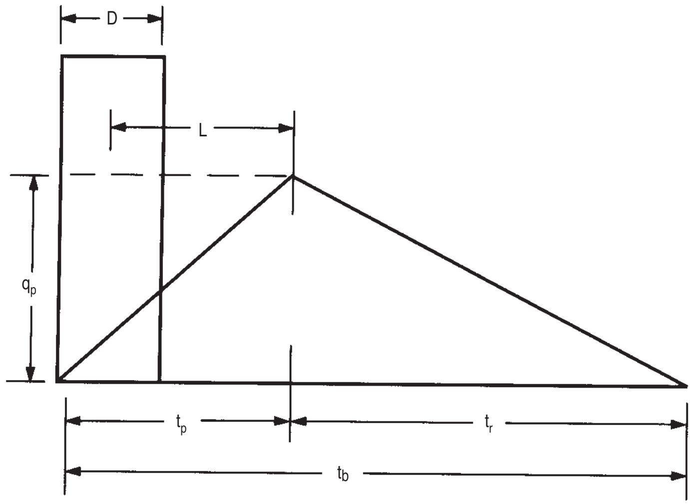


Figure 3.14 SCS triangular unit hydrograph.

The following parameters must be determined to define the triangular unit hydrograph; the time to peak (t_p), the peak discharge corresponding to 1 mm of runoff (q_p), and the base time of the hydrograph (t_b).

Once these parameters are determined, the unit hydrograph can be applied to a runoff depth or to a series of runoff depths. When applied to a series of runoff depths, sub-hydrographs are developed for each and summed to provide an overall hydrograph. A series of runoff depths, for instance, may be a sequence of runoff depths such as those values obtained from a hyetograph where excess rainfall is that portion of the rainfall that is runoff, calculated as the rainfall adjusted to account for retention losses.

The lag time (L) is the delay between the centre of the excess rainfall period (D) and the peak of the runoff (t_p). The SCS has suggested that the lag time, for an average watershed and fairly uniform runoff, can be approximated by:

$$L \approx 0.6\ T_c$$

The estimate of the time to peak (t_p) is therefore affected by the time of concentration (T_c) and the excess rainfall period (D). It is calculated using the relationship:

$$t_p = 0.5\ D + 0.6\ T_c$$

where T_c may be determined by and acceptable method such as those described in the previous section. For a series of runoff depths, where the timestep used is Δt, the parameter D should be replaced by Δt in the above equation, so that it becomes:

$$t_p = 0.5\ \Delta t + 0.6\ T_c$$

The duration of the recession limb of the hydrograph is assumed to be $t_r = (^5/_3)\ t_p$ so that the time base given by $t_b = (^8/_3)\ t_p$.

The ordinates of the unit hydrograph are expressed in units of discharge per unit depth of runoff. In terms of the notation used in Figure 3.14:

$$q_p = 0.208\ A/t_p$$

where: q_p = peak discharge, m^3/s per mm of runoff
A = catchment area, km^2
t_p = time to peak, hours

The numerical constant in the above equation is a measure of the watershed characteristics. This value varies between about 0.129 for flat marshy catchments and 0.258 for steep flashy catchments. A value of 0.208 is recommended by the SCS for average watersheds.

From the above equation it can be seen that the time to peak (t_p), and therefore the peak discharge of the unit hydrograph (q_p), is affected by the value of the excess rainfall period (D) and, in the case of a series of runoff depths, the timestep used (Δt). Values of D or Δt in excess of 0.25 t_p should not be used as this can lead to the underestimation of the peak runoff.

Rectangular Unit Hydrograph Method

An alternative option to the triangular distribution used in the SCS Method is the rectangular unit hydrograph. Figure 3.15 illustrates the concept of convoluting the effective rainfall with a rectangular unit hydrograph. The ordinate of the unit hydrograph is defined as the area of the unit hydrograph divided by the time of concentration (T_c).

The Rational Method is often used for a rough estimate of the peak flow. This method, which assumes the peak flow occurs when the entire catchment surface is contributing to runoff, may be simulated using a rectangular unit hydrograph. The effective rainfall hydrograph is reduced to a simple rectangular function and $i_{eff} = k \cdot C \cdot i$. The effective rainfall, with duration t_d, is convoluted with a rectangular unit hydrograph which has a base equal to the time of concentration (T_c). If t_d is made equal to T_c, the resultant runoff hydrograph will be symmetrical and triangular in shape with a peak flow given by $Q = k{\cdot}C{\cdot}i{\cdot}A$ and a time base of $t_b = 2\ T_c$. If the rainfall duration (t_d) is not equal to T_c, then the resultant runoff hydrograph is trapezoidal in shape with a time base of $t_b = t_d = T_c$ and a peak flow given by the following equation:

$$Q = k \cdot C \cdot i \cdot A\ (t_d / T_c) \qquad \text{for } t_d \leq T_c$$

$$\text{and } Q = k \cdot C \cdot i \cdot A \qquad \text{for } t_d > T_c$$

This approach makes no allowance for the storage effect due to the depth of overland flow and results in an "instantaneous" runoff hydrograph. This may be appropriate for impervious surfaces in which surface depression storage is negligible, but for pervious or more irregular surfaces it may be necessary to route the instantaneous hydrograph through a hypothetical reservoir in order to more closely represent the runoff hydrograph.

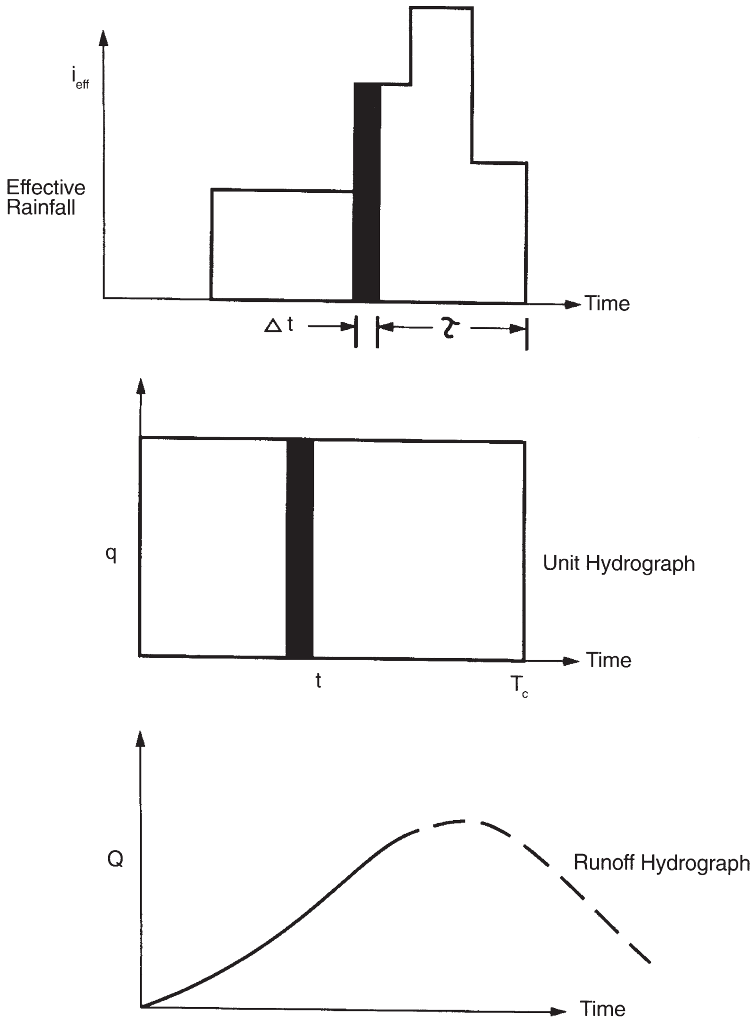


Figure 3.15 Convolution process using a rectangular unit hydrograph.

Linear Reservoir Method

Pederson suggested a more complex response function in which the shape of the unit hydrograph is assumed to be the same as the response of a single linear reservoir to an inflow having a rectangular shape and duration Δt. A linear reservoir is one in which the storage (S) is linearly related to the outflow (Q) by the formula:

$$S = K \cdot Q$$

where: K = the reservoir lag or storage coefficient (hours)

In Pederson's method, the value of K is taken to be 0.5 T_c where T_c is computed from the kinematic wave equation in which the rainfall intensity used is the maximum for the storm being modeled. The use of i_{max} is justified since this intensity tends to dominate the subsequent runoff hydrograph. The resulting unit hydrograph is illustrated in Figure 3.16 and comprises a steeply rising limb, which reaches a maximum at time t = Δt, followed by an exponential recession limb. The two curves can be described by the following equations:

$$q_p = \frac{(1 - e^{-\Delta t/K})}{\Delta t} \quad \text{at } t = \Delta t$$

and,

$$q = q_p \bullet e^{-(t-\Delta t)/K} \quad \text{for } t > \Delta t$$

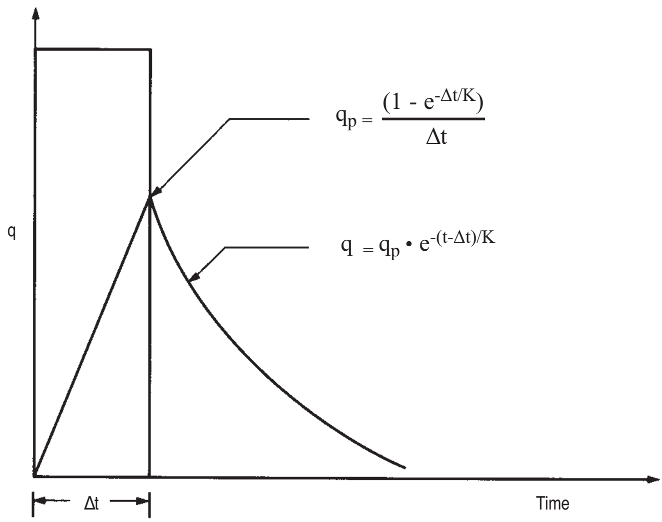


Figure 3.16 The single linear reservoir.

An important feature of the method is that the unit hydrograph always has a time to peak of Δt and is incapable of reflecting different response times as a function of catchment length, slope or roughness. It follows that the peak of the runoff hydrograph will usually be close to the time of peak rainfall intensity irrespective of the catchment characteristics.

SWMM Runoff Algorithm

The Storm Water Management Model was originally developed for the U.S. Environmental Protection Agency in 1971. Since then it has been expanded and improved by the EPA and many other agencies and companies. In particular, the capability for continuous simulation has been included (in addition to the original ability to handle single event simulation), quality as well as quantity is simulated, and snow-melt routines are included in some versions.

The model is intended for use in urban or partly urban catchments. It comprises five main "blocks" of code in addition to an Executive Block or supervisory calling program. Following is a description of the basic algorithm of the Runoff Block, which is used to generate the runoff hydrograph in the drainage system based on a rainfall hyetograph, antecedent moisture conditions, land use and topography.

The method differs from those described above in that it does not use the concept of effective rainfall, but employs a surface water budget approach in which rainfall, infiltration, depression storage and runoff are all considered as processes occurring simultaneously at the land surface. The interaction of these inputs and outputs may be visualized with reference to Figure 3.17.

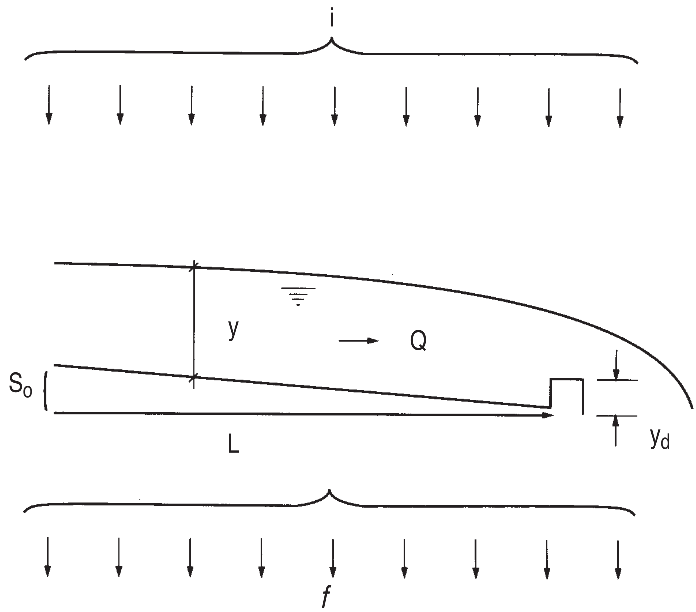


Figure 3.17 Representation of the SWMM/Runoff algorithm.

Treating each sub-catchment as an idealized, rectangular plane surface having a breadth (B) and length (L), the continuity or mass balance equation at the land surface is given by:

Inflow = (Infiltration + Outflow) + Rate of Surface Ponding

That is:

$$i \bullet L \bullet B = (f \bullet L \bullet B + Q) + L \bullet B \bullet (\Delta y/\Delta t)$$

where: i = rainfall intensity
f = infiltration rate
Q = outflow
y = depth of flow over the entire surface

The depth of flow (y) is computed using the Manning equation, taking into account the depth of surface depression storage (y_d) which is also assumed to be uniform over the entire surface. The dynamic equation is given by:

$$Q = B\,(1/n)\,(y-y_d)^{5/3}\,S^{1/2}$$

where: n = Manning's roughness coefficient for overland flow
S = average slope of the overland flow surface

The infiltration rate (f) must be computed using a method such as the 'moving curve' Horton equation or the Green-Ampt model. Infiltration is assumed to occur as long as excess surface moisture is available from rainfall, depression storage or finite overland flow.

It is important to note that the value of Manning's "n" used for overland flow is somewhat artificial (for example, in the range of 0.1 to 0.4) and does not represent a value which might be used for channel flow calculation.

Various methods can be used for the simultaneous solution of the continuity and dynamic equations. One method is to combine the equations into one nondifferential equation in which the depth (y) is the unknown. Once the depth is determined (for instance, by an interactive scheme such as the Newton-Raphson Method) the outflow (Q) follows.

COMPUTER MODELS

Many computer models have been developed for the simulation of the rainfall/runoff process. Table 3.10 lists several of these models and their capabilities.

Table 3.10 Hydrologic computer models

Model Characteristics	HEC-1	HYMO	HSPF	ILLUDAS	MIDUSS	OTTHYMO	QUALHYMO	SCS TR-20	SCS TR-55	SSARR	STANFORD	STORM	SWMM	USDAHL-74
Model Type:														
Single Event	●	●		●	●	●		●	●				●	
Continuous			●				●			●	●	●	●	●
Model Components:														
Infiltration	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Evapotranspiration			●							●	●	●	●	●
Snowmelt	●		●							●	●	●	●	●
Surface Runoff	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Subsurface Flow	●		●				●			●	●			●
Reservoir Routing	●	●	●		●	●	●	●	●	●			●	
Channel Routing	●	●	●	●	●	●	●	●	●	●	●		●	
Water Quality			●				●					●	●	
Application:														
Urban Land Use			●	●	●	●	●		●			●	●	
Rural Land Use	●	●	●		●	●	●	●		●	●	●		●
Ease of Use:														
High	●	●		●	●	●	●	●	●			●		●
Low			●							●	●		●	

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CHAPTER 4

HYDRAULICS

INTRODUCTION

Many millions of dollars are spent annually on culverts, storm drains and subdrains, all vital to the protection of streets, highways and railroads. If inadequately sized, they can jeopardize the roadway and cause excessive property damage and loss of life. Over design means extravagance. Engineering can find an economical solution.

Topography, soil and climate are extremely variable, so drainage sites should be designed individually from reasonably adequate data for each particular site. In addition, the designer is advised to consult with those responsible for maintaining drainage structures in the area. One highway engineer comments:

"With the exception of the riding qualities of the traveled way, no other single item requires as much attention on the part of maintenance personnel as highway culverts. Many of the problems of culvert maintenance stem from the fact that designers in all too many instances consider that culverts will be required to transport only clear water. This is a condition hardly ever realized in practice, and in many instances storm waters may be carrying as much as 50 percent detrimental material. A rapid change in grade line at the culvert entrance can cause complete blockage of the culvert. This results in overflow across the highway and in some cases, especially where high fills are involved, the intense static pressure results in loss of the embankment."

HYDRAULICS OF OPEN DRAINAGE CHANNELS

General

Before designing culverts, storm sewers and other drainage structures, one should consider the design of ditches, gutters, chutes, median swales, and other channels leading to these structures. (See Figure 4.1).

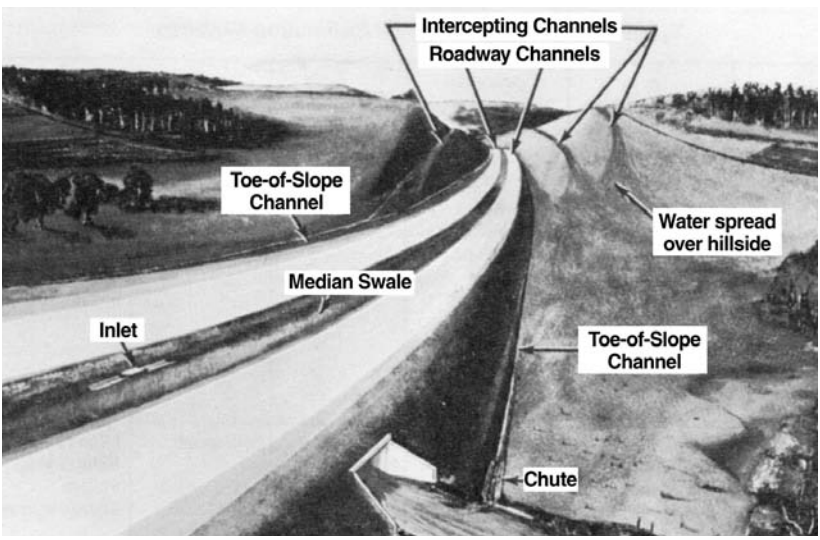


Figure 4.1 Types of roadside drainage channels.

The design engineer with needs beyond the scope of this handbook may refer to the CSPI publication, “Modern Sewer Design” and AISI “Design Charts for Open Channel Flow”. These include numerous examples of calculations and references on all aspects of the subject.

Rainfall and runoff, once calculated, are followed by the design of suitable channels to handle the peak discharge with minimum erosion, maintenance and hazard to traffic.

The AASHTO publication "A Policy on Geometric Design of Highway and Streets" states: "The depth of channels should be sufficient to remove the water without saturation of the pavement subgrade. The depth of water that can be tolerated, particularly on flat channel slopes, depends upon the soil characteristics. In open country, channel side slopes of 5:1 or 6:1 are preferable in order to reduce snow drifts."

Systematic maintenance is recognized as essential to any drainage channel and therefore should be considered in the design of those channels.

Chezy Equation

Chezy developed a basic hydraulic formula for determining the flow of water, particularly in open channels. It is as follows:

$$Q = AV$$

if: $$V = c\sqrt{RS}$$

then: $$Q = Ac\sqrt{RS}$$

where:
Q = discharge, m^3/s
A = cross-sectional area of flow, m^2
V = mean velocity of flow, m/s
c = coefficient of roughness, depending upon the surface over which water is flowing, $m^{1/2}/s$

R = hydraulic radius, m
$= \frac{A}{WP}$
WP = wetted perimeter (length of wetted contact between water and its containing channel), m
S = slope, or grade, m/m

This fundamental formula is the basis of most capacity formulas.

Manning’s Equation

Manning’s equation, published in 1890, gives the value of c in the Chezy formula as:

$$c = \frac{R^{1/6}}{n}$$

where: n = coefficient of roughness (see Tables 4.1 and 4.2)

Table 4.1 Manning's *n* for constructed channels

Type of channel and description	*n*
1. LINED OR BUILT-UP	
A. Concrete - Trowel Finish	0.013
B. Concrete - Float Finish	0.015
C. Concrete - Unfinished	0.017
D. Gunite - Good Section	0.019
E. Gravel Bottom with sides of:	
1) Formed Concrete	0.020
2) Random Stone in Mortar	0.023
3) Dry Rubble or Rip Rap	0.033
2. EXCAVATED OR DREDGED - EARTH	
A. Straight and Uniform	
1) Clean, Recently Completed	0.018
2) Clean, After Weathering	0.022
3) Gravel, Uniform Section, Clean	0.025
4) With Short Grass, Few Weeds	0.027
B. Winding and Sluggish	
1) No Vegetation	0.025
2) Grass, Some Weeds	0.030
3) Dense Weeds, Deep Channels	0.035
4) Earth Bottom and Rubble Sides	0.030
5) Stony Bottom and Weedy Banks	0.035
6) Cobble Bottom and Clean Sides	0.040
3. CHANNELS NOT MAINTAINED, WEEDS & BRUSH UNCUT	
A. Dense Weeds, High as Flow Depth	0.080
B. Clean Bottom, Brush on Sides	0.050
C. Same, Highest Stage of Flow	0.070
D. Dense Brush, High Stage	0.100

Table 4.2 Manning's *n* for natural stream channels
Surface width at flood stage less than 30 m

1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01–0.02
2. Irregular sections, with pools, slight channel meander; increase values given above about	0.01–0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07

The complete Manning equation is:

$$V = \frac{R^{2/3}S^{1/2}}{n}$$

Combining this with the Chezy Equation results in the equation:

$$Q = \frac{AR^{2/3}S^{1/2}}{n}$$

In many calculations, it is convenient to group the channel cross section properties in one term called conveyance, K, so that:

$$K = \frac{AR^{2/3}}{n}$$

Then:

$$Q = KS^{1/2}$$

Uniform flow of clean water in a straight unobstructed channel would be a simple problem but is rarely attained. Manning's formula gives reliable results if the channel cross section, roughness, and slope are fairly constant over a sufficient distance to establish uniform flow.

The Use of Charts and Tables

While design charts for open-channel flow reduce computational effort, they cannot replace engineering judgment and a knowledge of the hydraulics of open-channel flow and flow through conduits with a free water surface.

Design charts contain the channel properties (area and hydraulic radius) of many channel sections and tables of velocity for various combinations of slope and hydraulic radius. Their use is explained in the following examples.

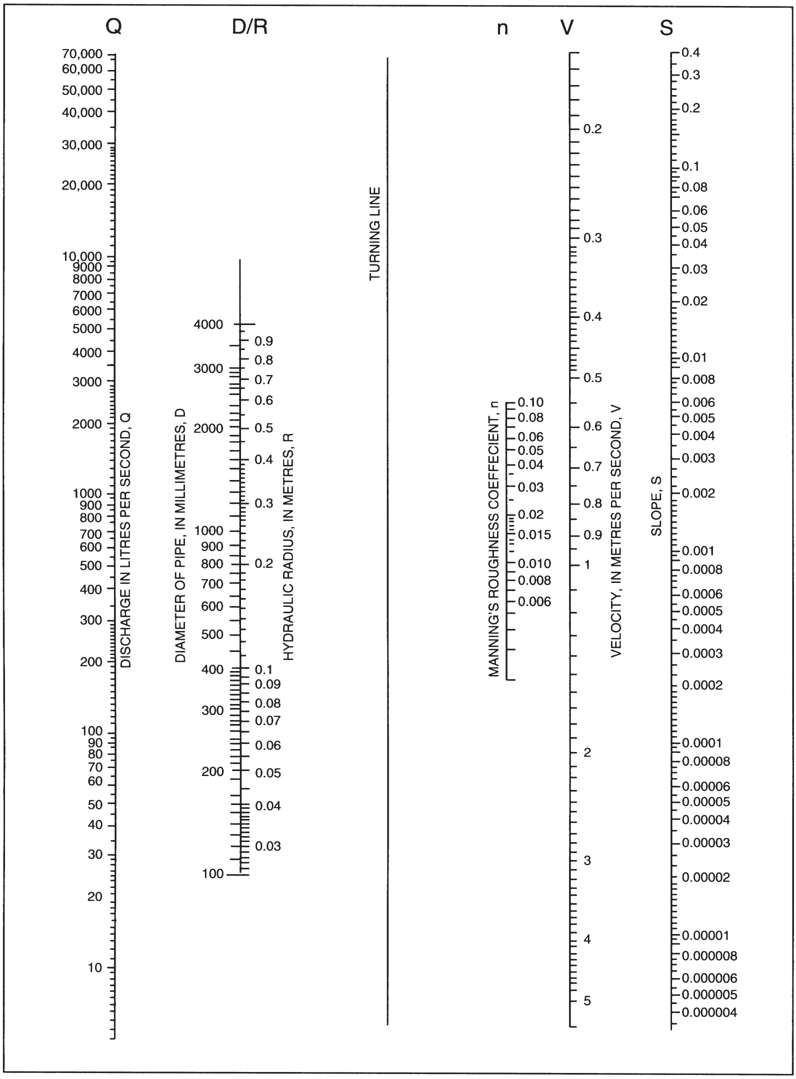
Example 1

Given: A trapezoidal channel of straight alignment and uniform cross section in earth with a bottom width of 0.6 m, side slopes at 1:1, a channel slope of 0.003 m/m, and a normal depth of water of 0.3 m.

Find: Velocity and discharge.

Solution:

1. Based on Table 4.1, for an excavated channel in ordinary earth, n is taken as 0.022.
2. Cross-sectional area, A, is 0.27 m^2 [0.3 * (0.6 + 1 * 0.3)].
3. Wetted perimeter, WP, is 1.449 m [0.6 + 2 * 0.3 * $(1^2+1)^{1/2}$].
4. Hydraulic radius, R, is 0.186 m [0.27 / 1.449].
5. Using the nomograph in Figure 4.2, lay a straight edge between the outer scales at the values of S = 0.003 and n = 0.02. Mark where the straight edge intersects the turning line.
6. Place the straight edge to line up the point on the turning line and the hydraulic radius of 0.186 m.
7. Read the velocity, V, of 0.80 m/s on the velocity scale.
8. Discharge, Q, is 0.216 m^3/s [0.27 * 0.89].



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Figure 4.2 Nomograph for solution of Manning's equation.

Figure 4.3 provides the means to calculate a trapezoidal channel capacity for a specific bottom width, channel slope, side slope, n value and a variety of flow depths. For a given drainage project, these variables are known or determined using known site parameters through trial and error. The flow rate, Q, can then be calculated.

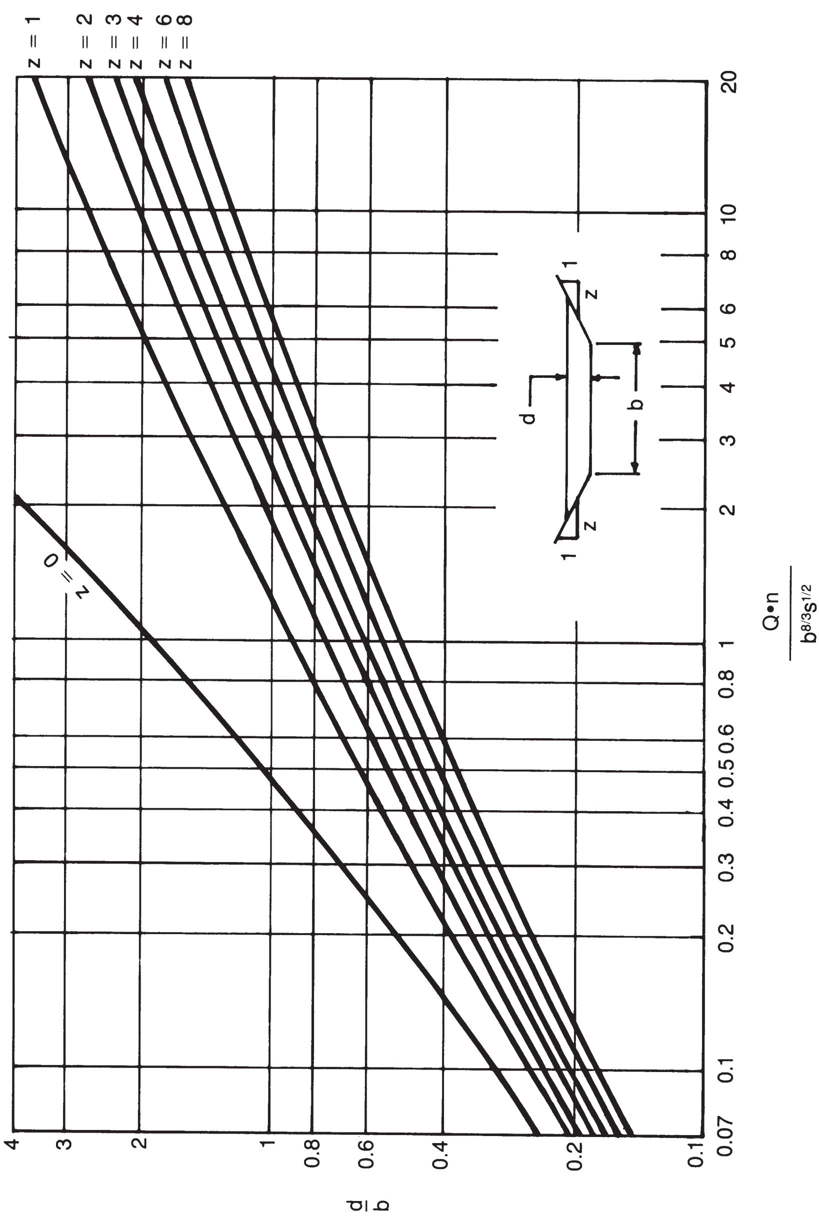


Figure 4.3 Capacity of trapezoidal channel.

Example 2

Given: Bottom width, b = 6.1 m
Side slopes @ 2:1, so z = 2
Roughness coefficient, n = 0.030 (from Table 4.2 for grass and weeds, no brush)
Channel slope, S = 0.002 m/m
Depth to width ratio, $\frac{d}{b} = 0.6$ (flood stage depth)

Find: Depth of flow, d, and flow rate, Q.

Solution:

Depth, d = 0.6 (6.1) = 3.66 m

From Figure 4.3: $$\frac{Q \bullet n}{b^{8/3}\, S^{1/2}} = 0.62$$

So: $$\frac{Q\,(0.030)}{(6.1)^{8/3}\,(0.002)^{1/2}} = 0.62$$

And: $$Q = 114.8 \text{ m}^3/\text{s}$$

If the resulting design is not satisfactory, the channel parameters are adjusted and the design calculations are repeated.

Safe Velocities

The ideal situation is one where the velocity will cause neither silt deposition nor erosion. For the design of a channel, the approximate grade can be determined from a topographic map, from the plan profiles, or from both.

To prevent the deposition of sediment, the minimum gradient for earth and grass-lined channels should be about 0.5 percent and that for smooth paved channels about 0.35 percent.

Convenient guidelines for permissible velocities are provided in Tables 4.3 and 4.4. More comprehensive design data may be found in the U.S. FHWA's HEC 15 (Design of Stable Channels with Flexible Linings).

Channel Protection

Corrugated steel flumes or chutes (and pipe spillways) are favored solutions for channel protection especially in wet, unstable or frost susceptible soils. They should be anchored to prevent undue shifting. This will also protect against buoyancy and uplift, which can occur especially when empty. Cutoff walls or collars are used to prevent undermining.

If the mean velocity exceeds the permissible velocity for the particular type of soil, the channel should be protected from erosion. Grass linings are valuable where grass growth can be supported. Ditch bottoms may be sodded or seeded with the aid of temporary quick growing grasses, mulches, or erosion control blankets. Grass may also be used in combination with other, more rigid types of linings, where the grass is on the upper bank slopes and the rigid lining is on the channel bottom. Linings may consist of stone which is dumped, hand placed or grouted, preferably laid on a filter blanket of gravel or crushed stone and a geotextile.

Table 4.3 Comparison of limiting water velocities and tractive force values for the design of stable channels (straight channels after aging; channel depth = 1m)

Material	n	For Clear Water		Water Transporting Colloidal Silts	
		Velocity, m/s	Tractive Force, Pa	Velocity, m/s	Tractive Force, Pa
Fine sand colloidal	0.020	0.46	1.29	0.76	3.59
Sandy loam noncolloidal	0.020	0.53	1.77	0.78	3.59
Silt loam noncolloidal	0.020	0.61	2.30	0.91	5.27
Alluvial silts noncolloidal	0.020	0.61	2.30	1.07	7.18
Ordinary firm loam	0.020	0.76	3.59	1.07	7.18
Volcanic ash	0.020	0.76	3.59	1.07	7.18
Stiff clay very colloidal	0.025	1.14	12.45	1.52	22.02
Alluvial silts colloidal	0.025	1.14	12.45	1.52	22.02
Shales and hardpans	0.025	1.83	32.08	1.83	32.08
Fine gravel	0.020	0.76	3.59	1.52	15.32
Graded loam to cobbles when non-colloidal	0.030	1.14	18.19	1.52	31.60
Graded silts to cobbles when colloidal	0.030	1.22	20.59	1.68	38.30
Coarse gravel non-colloidal	0.025	1.22	14.36	1.83	32.08
Cobbles and shingles	0.035	1.52	43.57	1.68	52.67

Table 4.4 Maximum permissible velocities in vegetal-lined channels[d]

Cover Average, Uniform Stand, Well Maintained	Slope Range	Permissible Velocity[a]	
		Erosion Resistant Soils	Easily Eroded Soils
	Percent	m/s	m/s
Bermudagrass	0 -5 5-10 over 10	2.44 2.13 1.83	1.83 1.52 1.22
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0-5 5-10 over 10	2.13 1.83 1.52	1.52 1.22 0.91
Grass mixture[b]	0 -5 5 -10	1.52 1.22	1.22 0.91
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0 -5	1.07	0.76
Common lespedeza[b] Sudangrass[b]	0-5[c]	1.07	0.76

[a] From "Handbook of Channel Design for Soil and Water Conservation:' Soil Conservation Service SCS-TP-61, Revised June 1954

[b] Annuals-used on mild slopes or as temporary protection until permanent covers are established.

[c] Use on slopes steeper than 5 percent is not recommended.

[d] Data for this table is a composite of data from several reference sources.

Asphalt and concrete lined channels are used for steep erodible channels.

Ditch checks are an effective means of decreasing the velocity and thereby the erodability of the soil.

High velocities, where water discharges from a channel, must be considered and provisions must be made to dissipate the excess energy.

HYDRAULICS OF CULVERTS

Introduction

Culvert design has not yet reached the stage where two or more individuals will always arrive at the same answer, or where actual service performance matches the designer's expectation. The engineer's interpretation of field data and hydrology is often influenced by personal judgement, based on experience in a given locality. However, hydrology and hydraulic research are closing the gap to move the art of designing a culvert closer to becoming a science.

Up to this point, the design procedure has consisted of (1) collecting field data, (2) compiling facts about the roadway, and (3) making a reasonable estimate of flood discharge. The next step is to design an economical corrugated steel structure to handle the flow (including debris) with minimum damage to the slope or culvert barrel. Treatment of the inlet and outlet ends of the structure must also be considered.

Improving hydraulic capacity (inlet control) with special features.

What Makes a Good Culvert?

An ASCE Task Force on Hydraulics of Culverts offers the following recommendations for "Attributes of a Good Highway Culvert":

1. The culvert, appurtenant entrance and outlet structures should properly take care of water, bed-load, and floating debris at all stages of flow.
2. It should cause no unnecessary or excessive property damage.
3. Normally, it should provide for transportation of material without detrimental change in flow pattern above and below the structure.

4. It should be designed so that future channel and highway improvement can be made without too much loss or difficulty.
5. It should be designed to function properly after fill has caused settlement.
6. It should not cause objectionable stagnant pools in which mosquitoes may breed.
7. It should be designed to accommodate increased runoff occasioned by anticipated land development.
8. It should be economical to build, hydraulically adequate to handle design discharge, structurally durable and easy to maintain.
9. It should be designed to avoid excessive ponding at the entrance which may cause property damage, accumulation of drift, culvert clogging, saturation of fills, or detrimental upstream deposits of debris.
10. Entrance structures should be designed to screen out material which will not pass through the culvert, reduce entrance losses to a minimum, make use of the velocity of approach in so far as practicable, and by use of transitions and increased slopes, as necessary, facilitate channel flow entering the culvert.
11. The design of the culvert outlet should be effective in re-establishing tolerable non-erosive channel flow within the right-of-way or within a reasonably short distance below the culvert.

CSP structure ready for backfill placement and headwalls.

12. The outlet should be designed to resist undermining and washout.
13. Energy dissipaters, if used, should be simple, easy to build, economical and reasonably self-cleaning during periods of easy flow.

Design Method

The culvert design process should strive for a balanced result. Pure fluid mechanics should be combined with practical considerations to help assure satisfactory performance under actual field conditions. This includes due consideration of prospective maintenance and the handling of debris.

The California Division of Highways uses an excellent method of accomplishing this; one that has worked well for many years. Other jurisdictions have used similar approaches. California culvert design practice establishes the following:

Criteria for Balanced Design:
The culvert shall be designed to discharge

a) a 10 year flood without static head at the entrance, and
b) a 100 year flood utilizing the available head at the entrance.

This approach lends itself well to most modern design processes and computer programs. It provides a usable rationale for determining a minimum required waterway area.

The permissible height of water at the inlet controls hydraulic design. This should be determined and specified for each site based on the following considerations:

1. Risk of overtopping the embankment and the resulting risk to human life.
2. Potential damage to the roadway, due to saturation of the embankment, and pavement disruption due to freeze-thaw.
3. Traffic interruptions.
4. Damage to adjacent or upstream property, or to the channel or flood plain environment.
5. Intolerable discharge velocities, which can result in scour and erosion.
6. Deposition of bed load and/or clogging by debris on recession of flow.

Flow Conditions and Definitions

Culverts considered here are circular pipes and pipe-arches with a uniform barrel cross-section throughout.

There are two major types of culvert flow conditions:

Inlet Control – A culvert flowing in inlet control is characterized by shallow, high velocity flow categorized as supercritical. Inlet control flow occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section is near the inlet, and the downstream pipe and flow have no impact on the amount of flow through the pipe. Under inlet control, the factors of primary importance are (1) the cross-sectional area of the barrel, (2) the inlet configuration or geometry, and (3) the headwater elevation or the amount of ponding upstream of the inlet (see Figure 4.4). The barrel slope also influences the flow under inlet control, but the effect is small and it can be ignored.

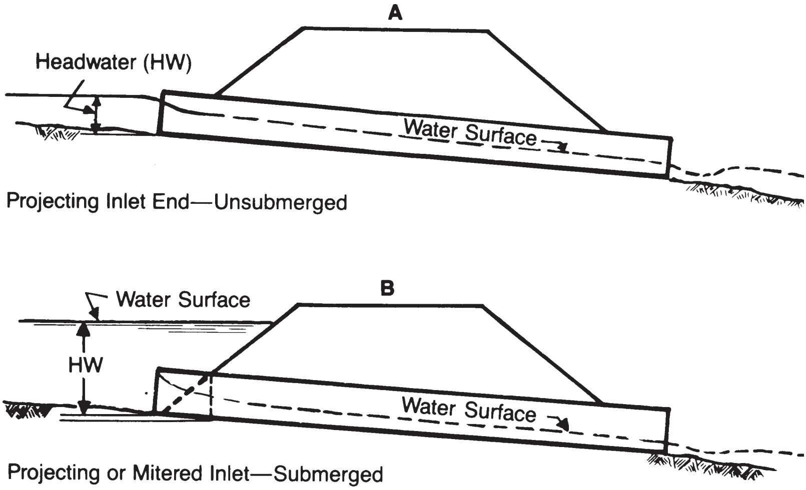


Figure 4.4 Inlet control flow regimes.

Outlet Control – A culvert flowing in outlet control is characterized by relatively deep, lower velocity flow categorized as subcritical. Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section is at the outlet of the culvert. In addition to the factors considered for inlet control, factors that must be considered for outlet control include (1) the tailwater elevation in the outlet channel, (2) the barrel slope, (3) the barrel roughness, and (4) the length of the barrel (see Figure 4.5).

Hydraulics of Culverts in Inlet Control

Inlet control means that the discharge capacity is controlled at the entrance by the headwater depth, cross-sectional area and type of inlet edge. The roughness, length, and outlet conditions are not factors in determining the culvert capacity.

Sketches A and B in Figure 4.4 show unsubmerged and submerged projecting inlets. Inlet control performance is classified by these two regions (unsubmerged flow and submerged flow) as well as a transition region between them.

Entrance loss depends upon the geometry of the inlet edge and is expressed as a fraction of the velocity head. Research with models and prototype testing have resulted in coefficients for various types of inlets, as shown in Table 4.5 and Figure 4.6.

Table 4.5 **Entrance loss coefficients for corrugated steel pipe or pipe-arch**

Inlet End of Culvert	Entrance Type	Coefficient, k_e
Projecting from fill (no headwall)	1	0.9
Mitered (bevelled) to conform to fill slope	2	0.7
Headwall or headwall and wingwalls square-edge	3	0.5
End-Section conforming to fill slope	4	0.5
Headwall rounded edge	5	0.2
Bevelled Ring	6	0.25

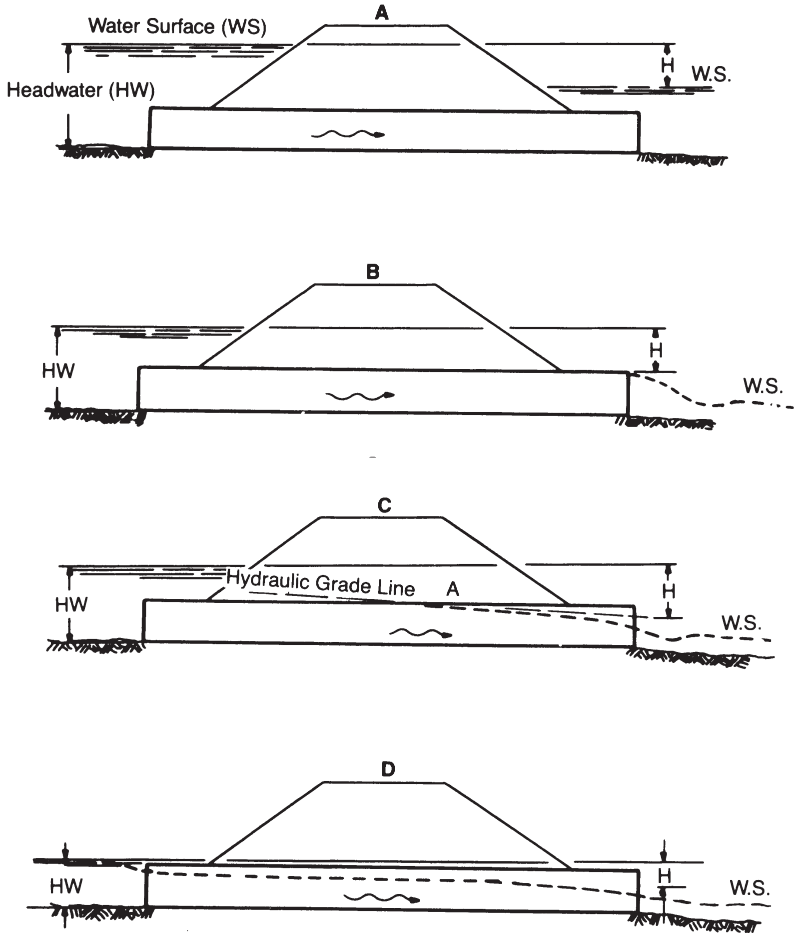
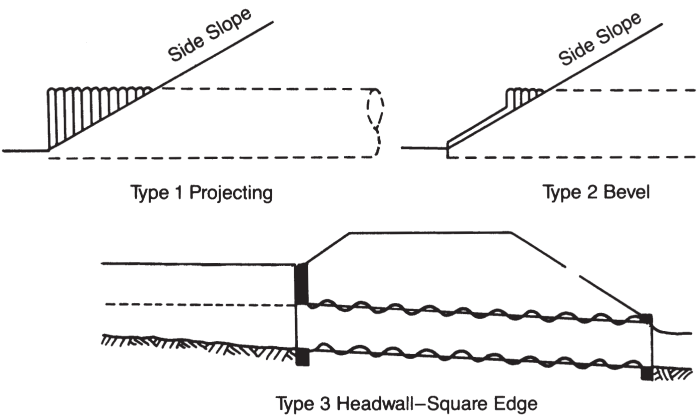
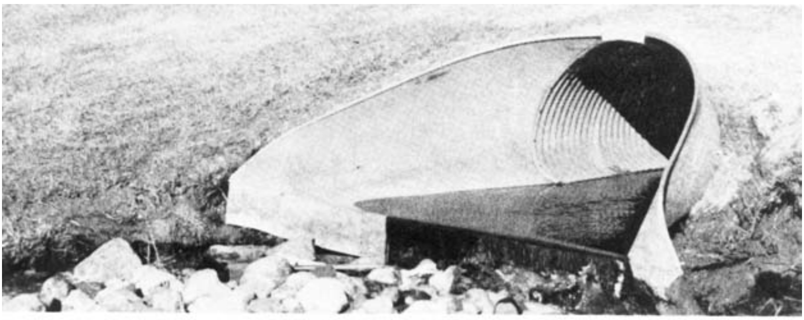
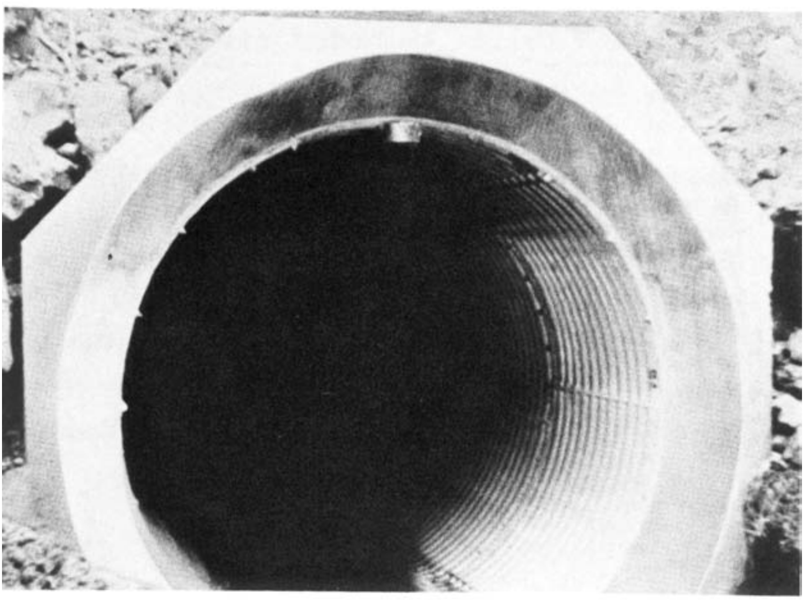


Figure 4.5 Outlet control flow regimes.





Type 4 End Section



Type 6 Bevelled Ring

Figure 4.6 Typical inlet entrances.

The model testing and prototype measurements also provide information used to develop equations for unsubmerged and submerged inlet control flow. The transition zone is poorly defined, but it is approximated by plotting the two flow equations and connecting them with a line which is tangent to both curves. These plots, done for a variety of structure sizes, are the basis for constructing the design nomographs included in this handbook.

In the nomographs, the headwater depth, HW, is the vertical distance from the culvert invert (bottom) at the entrance to the energy grade line of the headwater pool. It, therefore, includes the approach velocity head. The velocity head tends to be relatively small and is often neglected. The resulting headwater depth is therefore conservative and the actual headwater depth would be slightly less than the calculated value. If a more accurate headwater depth is required, the approach velocity head should be subtracted from the headwater depth determined using the nomographs.

Hydraulics of Culverts in Outlet Control

Outlet control means that the discharge capacity is controlled at the outlet by the tailwater depth or critical depth, and it is influenced by such factors as the slope, wall roughness and length of the culvert. The following energy balance equation contains the variables that influence the flow through culverts flowing under outlet control:

$$L \cdot S_o + HW + \frac{V_1^2}{2g} = h_o + H + \frac{V_2^2}{2g}$$

where: L = length of culvert, m
S_o = slope of barrel, m/m
HW = headwater depth, m
V_1 = approach velocity, m/s
g = gravitational constant = 9.806 m/s^2
h_o = outlet datum, m
H = head, m
V_2 = downstream velocity, m/s

The headwater depth, HW, is the vertical distance from the culvert invert at the entrance (where the entrance is that point in the pipe where there is the first full cross-section) to the surface of the headwater pool.

As discussed under inlet control hydraulics, the water surface and energy grade line are usually assumed to coincide at the entrance (the approach velocity head is ignored). The same can be said for the downstream velocity head. That being the case, the approach velocity head and downstream velocity head terms in the above equation would be dropped and the equation would take the form below. Note that this equation has been organized to provide the resulting headwater depth.

$$HW = h_o + H - L \cdot S_o$$

The head, or energy (Figures 4.5 through 4.9), required to pass a given quantity of water through a culvert flowing in outlet control, is made up of a (1) entrance loss, (2) friction loss, and (3) exit loss.

This energy is expressed in equation form as:

$$H = H_e + H_f + H_o$$

where: H_e = entrance loss, m
H_f = friction loss, m
H_o = exit loss, m

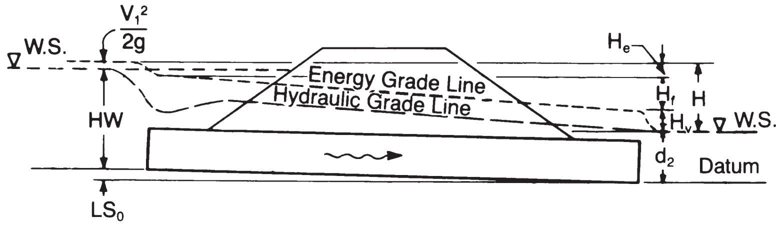


Figure 4.7 Difference between energy grade line and hydraulic grade line.

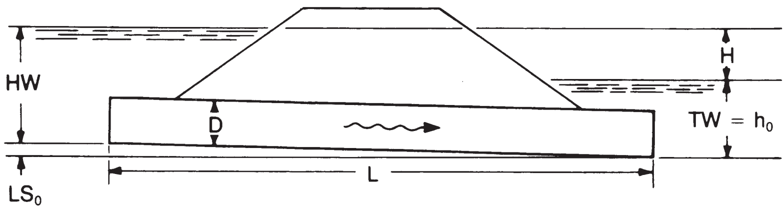


Figure 4.8 Relationship of headwater to high tailwater.

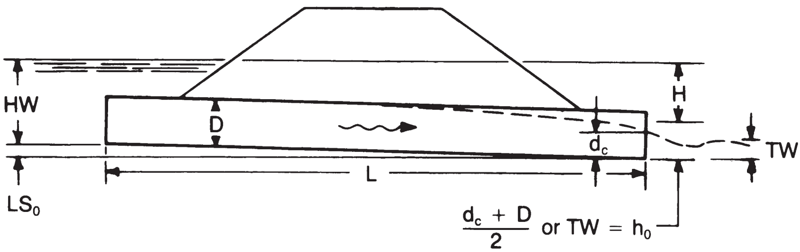


Figure 4.9 Relationship of headwater to low tailwater.

The hydraulic slope, or hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length (see Figure 4.7). For full flow, the energy grade line and hydraulic grade line are parallel over the length of the barrel except in the vicinity of the inlet where the flow contracts and re-expands. The difference between the energy grade line and hydraulic grade line is the velocity head. It turns out that the velocity head is a common variable in the expressions for entrance, friction and exit loss.

The velocity head is expressed by the following equation:

$$H_v = \frac{V^2}{2g}$$

where: H_v = velocity head, m
V = mean velocity of flow in the barrel, m/s = $\frac{Q}{A}$
Q = design discharge, m^3/s
A = cross sectional area of the culvert, m^2

The entrance loss depends upon the geometry of the inlet. This loss is expressed as an entrance loss coefficient multiplied by the velocity head, or:

$$H_e = k_e \frac{V^2}{2g}$$

where: k_e = entrance loss coefficient (Table 4.5)

The friction loss is the energy required to overcome the roughness of the culvert barrel and is expressed by the following equation:

$$H_f = \left\{ \frac{2gn^2L}{R^{1.33}} \right\} \frac{V^2}{2g}$$

where: n = Manning's friction factor (see Tables 4.6 and 4.7)

R = hydraulic radius, m = $\frac{A}{WP}$

WP = wetted perimeter, m

Table 4.6 **Values of Manning's *n* for corrugated and spiral rib steel pipe**

	Annular 68 x 13 mm	Helical 38 x 6.5 mm		Helical 68 x 13 mm						
	All Diameters	200	250	300	400	500	600	900	1200	1400 & Larger
Unpaved	0.024	0.012	0.011	0.013	0.014	0.015	0.016	0.018	0.020	0.021
25 % Paved	0.021						0.014	0.017	0.020	0.019
Fully Paved	0.012						0.012	0.012	0.012	0.012
	Annular 76 x 25 mm All Diameters	Helical-76 x 25 mm								
		1200	1400	1600	1800	2000	2200 & Larger			
Unpaved	0.027	0.023	0.023	0.024	0.025	0.026	0.027			
25 % Paved	0.023	0.020	0.020	0.021	0.022	0.022	0.023			
Fully Paved	0.012	0.012	0.012	0.012	0.012	0.012	0.012			
	Annular 125 x 25 mm All Diameters	Helical-125 x 25 mm								
		1400	1600	1800	2000 & Larger					
Unpaved	0.025	0.022	0.023	0.024	0.025					
25 % Paved	0.022	0.019	0.020	0.021	0.022					
Fully Paved	0.012	0.012	0.012	0.012	0.012					

Spiral Rib Pipe - all diameters Manning's n = .013

Note: ** When helically corrugated steel pipe is used for air conduction, the Darcy-Weisbach Formula with other values of F (or n) is used.

Combination stream crossing and voids to reduce dead load on foundation soils. (Ontario Ministry of Transportation project.)

Table 4.7 Values of Manning's *n* for 152 mm x 51 mm corrugation structural plate pipe

	Diameters			
	1500 mm	2120 mm	3050 mm	4610 mm
Plain-unpaved	0.033	0.032	0.030	0.028
25 % Paved	0.028	0.027	0.026	0.024

The exit loss depends on the change in velocity at the outlet of the culvert. For a sudden expansion, the exit loss is expressed as:

$$H_o = 1.0 \left[\frac{V^2}{2g} - \frac{V_2{}^2}{2g} \right]$$

As discussed previously, the downstream velocity head is usually neglected, in which case the above equation becomes the equation for the velocity head:

$$H_o = H_v = \frac{V^2}{2g}$$

Substituting in the equation for head we get (for full flow):

$$H = \left\{ k_e + \frac{2gn^2L}{R^{1.33}} + 1 \right\} \frac{V^2}{2g}$$

Nomographs have been developed and can be used for solving this equation. Note that these nomographs provide the head, whereas the inlet control nomographs provide the headwater depth. The head is then used to calculate the headwater depth by solving the preceding equation for HW (including the terms of h_o and $L \cdot S_o$).

This equation was developed for the full flow condition, which is as shown in Figure 4.5 A. It is also applicable to the flow condition shown in Figure 4.5 B.

Backwater calculations are required for the partly full flow conditions shown in Figure 4.5 C and D. These calculations begin at the downstream water surface and proceed upstream to the entrance of the culvert and the headwater surface. The downstream water surface is based on the greater of the critical depth or the tailwater depth.

The backwater calculations can be tedious and time consuming. Approximate methods have therefore been developed for the analysis of partly full flow conditions. Backwater calculations have shown that a downstream extension of the full flow hydraulic grade line, for the flow condition shown in Figure 4.5 C, intersects the plane of the culvert outlet cross section at a point half way between the critical depth and the top of the culvert. This is more easily envisioned as shown in Figure 4.9. It is possible, then, to begin the hydraulic grade line at that datum point and extend the straight, full flow hydraulic grade line to the inlet of the culvert. The slope of the hydraulic grade line is the full flow friction slope:

$$S_n = \frac{H_f}{L} = \left\{ \frac{2gn^2}{R^{1.33}} \right\} \frac{V^2}{2g}$$

If the tailwater elevation exceeds the datum point described above, the tailwater depth is used instead as the downstream starting point for the full flow hydraulic grade line.

The headwater depth is calculated by adding the inlet losses to the elevation of the hydraulic grade line at the inlet.

This approximate method works best when the culvert is flowing full for at least part of its length, as shown in Figure 4.5 C. If the culvert is flowing partly full for its whole length, as shown in Figure 4.5 D, the results become increasingly inaccurate as the flow depth decreases. The results are usually acceptable down to a headwater depth of about three quarters of the structure rise. For lower headwater depths, backwater calculations are required.

The outlet control nomographs can by used with the approximate method. In this case, the head is added to the datum point elevation to obtain the headwater depth. This method also works best when the culvert is flowing full for part of its length, and the results are not as accurate for a culvert flowing partly full.

Research on Values of n for Helically Corrugated Steel Pipe

Tests conducted on helically corrugated steel pipe, both round and pipe arch flowing full and part full, demonstrate a lower coefficient of roughness compared to annularly corrugated steel pipe. The roughness coefficient is a function of the corrugation helix angle (angle subtended between corrugation direction and centerline of the corrugated steel pipe), which determines the helically corrugated pipe diameter. A small helix angle associated with small diameter pipe, correlates to a lower roughness coefficient. Similarly, as the helix angle increases with diameter, the roughness coefficient increases, approaching the value associated with annularly corrugated pipe.

Values for 125 x 25 mm corrugations have been based on tests conducted using 152 x 25 mm and subsequently modified for the shorter pitch. Most published values of the coefficient of roughness, n, are based on experimental work conducted under controlled laboratory conditions using clear or clean water. The test pipe lines are straight with smooth joints. However, design values should take into account the actual construction and service conditions which can vary greatly for different drainage materials. Also, as noted on preceding pages, culvert or storm drain capacity under inlet control flow conditions is not affected by the roughness of pipe material.

Field Studies on Structural Plate Pipe

Model studies by the U.S. Corps of Engineers, and analyses of the results by the U.S. Federal Highway Administration, have been the basis for friction factors of structural plate pipe for many years. These values, originally shown in the 1967 edition of this Handbook, ranged from 0.0328 for 1500 mm diameter pipe to 0.0302 for 4610 mm pipe.

In 1968, the first full-scale measurements were made on a 457 m long 4300 mm diameter structural plate pipe line in Lake Michigan. These measurements indicated a lower friction factor than those derived from the model studies. As a result, the recommended values of Manning’s n for structural plate pipe of 3050 mm diameter and larger have been modified as shown in Table 4.7. The values for the smaller diameters remain as they were.

HYDRAULIC COMPUTATIONS

A balanced design approach establishes a minimum opening required to pass a 10 year flood with no ponding.

The 10 year discharge is established from hydrology data. The pipe size required to carry this flow, with no head at the entrance (HW/D = 1.0), is determined from nomographs. The designer uses the 10 year discharge to determine the pipe size required for inlet control and for outlet control, and uses whichever is greater. This is typically the minimum required opening size for the culvert.

Inlet Control

The headwater, HW, for a given pipe flowing under inlet control can be determined from Figures 4.10 through 4.16. Note that these figures are for arches as well as round pipes and pipe-arches.

These figures are first used to determine the pipe size required so that there is no head at the entrance under a 10 year flood condition. Once a pipe size is chosen, the designer also checks that pipe to determine whether outlet control will govern (as described below), and makes pipe size adjustments accordingly.

The designer then uses the selected pipe size to determine the headwater (for specific entrance conditions) for the 100 year flood discharge under inlet control. If this amount of headwater is acceptable, the chosen size is satisfactory for the full 100 year design discharge under inlet control. If the resulting headwater is too high, a larger size must be selected based on the maximum permissible headwater.

The values from the nomographs give the headwater in terms of a number of pipe rises (HW/D). The following formula is used to calculate the headwater depth:

$$HW_i = \frac{HW}{D} \bullet D$$

where: HW_i = headwater depth under inlet control, m

$\frac{HW}{D}$ = headwater depth in number of pipe rises, from nomograph, m/m

D = diameter of pipe, or rise of arch or pipe-arch, m

Outlet Control

Figures 4.17 through 4.24 are used, with the pipe size selected for inlet control, to determine the head loss, H. The head loss is then used in the following equation to determine the headwater depth under outlet control. If the depth computed for outlet control is greater than the depth determined for inlet control, then outlet conditions govern the flow conditions of the culvert and the higher headwater depth applies.

$$HW_o = h_o + H - L \bullet S_o$$

where: HW_o = headwater depth under outlet control, m

h_o = outlet datum, m; the greater of the tailwater depth, TW, or $\frac{(d_c + D)}{2}$

H = head, from nomograph, m

L = length of culvert barrel, m

S_o = slope of culvert barrel, m/m

TW = depth of flow in channel at culvert outlet, m

d_c = critical depth, from Figures 4.25 through 4.28, m

D = diameter of pipe, or rise of arch or pipe-arch, m

Wall roughness factors (Manning's n), on which the nomographs are based, are stated on each figure. In order to use the nomographs for other values of n, an adjusted value for length, L', is calculated using the formula below. This value is then used on the length scale of the nomograph, rather than the actual culvert length.

$$L' = L \bullet \left(\frac{n'}{n}\right)^2$$

where L' = adjusted length for use in nomographs, m
L = actual length, m
n' = actual value of Manning's n
n = value of Manning's n on which nomograph is based

Values of Manning's n for standard corrugated steel pipe, which were reported in Table 4.6, are shown for convenience in Table 4.8, together with the corresponding length adjustment factors, $\left(\frac{n'}{n}\right)^2$.

Table 4.8 **Length adjustment factors for corrugated steel pipe**

Pipe Diameter or span, mm	Roughness Factor n for Helical Corr.*	Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
300	0.013	0.29
600	0.016	0.44
900	0.018	0.56
1200	0.020	0.70
1400 & Larger	0.021	0.77

* Other values of roughness, n, are applicable to paved pipe, lined pipe, pipe with 76 x 25 and 125 x 25 corrugations, and spiral rib pipe. See Table 4.6.

Values of Manning's n for structural plate corrugated steel pipe, which were determined in the 1968 full-scale field measurements and which were reported in Table 4.7, are shown for convenience in Table 4.9, together with the corresponding length adjustment factors, $\left(\frac{n'}{n}\right)^2$.

Table 4.9 **Length adjustment factors for 152 mm x 51 mm corrugation structural plate pipe**

Pipe Diameter, mm	Roughness Factor		Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
	Curves based on $n =$	Actual $n' =$	
1500	0.0328	0.033	1.0
2120	0.0320	0.032	1.0
3050	0.0311	0.030	0.93
4920	0.0302	0.028	0.86
Pipe-arch Size, mm	**Roughness Factor**		**Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$**
	Curves based on $n =$	Actual $n' =$	
2060 x 1520	0.0327	0.033	1.0
2590 x 1880	0.0321	0.032	1.0
3400 x 2010	0.0315	0.030	0.91
5050 x 3330	0.0306	0.028	0.84

An appropriate entrance loss curve is used based on the desired entrance condition. Typical values of the entrance loss coefficient, k_e, for a variety of inlet configurations, are in Table 4.5.

If outlet control governs the capacity of the culvert and the headwater exceeds the maximum allowable value, a larger size pipe can be selected so that an acceptable headwater depth results. In such a case, corrugated steel structures with lower roughness coefficients should be considered. See Table 4.6 for alternatives. A smaller size of paved pipe, a helical pipe or a spiral rib pipe may be satisfactory.

Entrance conditions should also be considered. It may be economical to use a more efficient entrance than originally considered if a pipe size difference results. This can be easily investigated by checking the pipe capacity using other entrance loss coefficient curves.

Improved Inlets

Culvert capacity may be increased through the use of special inlet designs. The U.S. Federal Highway Administration (FHWA) has developed design methods for these types of structures. While these designs increase the flow, their use has been limited as a result of their cost and the level of knowledge of designers.

Hydraulic Nomographs

The inlet and outlet control design nomographs which appear in this handbook (Figures 4.10 through 4.24) were reproduced from nomographs developed and published by the FHWA. A certain degree of error is introduced into the design process due to the fact that the construction of nomographs involves graphical fitting techniques resulting in scales which do not exactly match equation results. All of the nomographs used in this handbook have a precision which is better that ±10 percent of the equation value in terms of headwater depth (inlet control) or head loss (outlet control). This degree of precision is usually acceptable, especially when considering the degree of accuracy of the hydrologic data. If a structure size is not shown on a particular nomograph, accuracy is not drastically affected when a user interpolates between known points.

Hydraulic Programs

Numerous computer programs now exist to aid in the design and analysis of highway culverts. These programs possess distinct advantages over traditional hand calculation methods. The increased accuracy of programmed solutions represents a major benefit over the inaccuracies inherent in the construction and use of tables and nomographs. In addition, programmed solutions are less time consuming. This feature allows the designer to compare alternative sizes and inlet configurations very rapidly so that the final culvert selection can be based on economics. Interactive capabilities in some programs can be utilized to change certain input parameters or constraints and analyze their effects on the final design. Familiarity with culvert hydraulics and the traditional analytical methods provides a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities available in culvert hydraulics programs.

Most programs analyze the performance of a given culvert, although some are capable of design. Generally, the desired result of either type of program is to obtain a culvert design which satisfies hydrologic needs and site conditions by considering both inlet and outlet control. Results usually include the barrel size, inlet dimensions, headwater depth, outlet velocity, and other hydraulic data. Some programs are capable of analyzing side-tapered and slope-tapered inlets. The analysis or design of the barrel size can be for one barrel only or for multiple barrels.

Some programs may contain features such as backwater calculations, performance curves, hydrologic routines, and capabilities for routing based on upstream storage considerations.

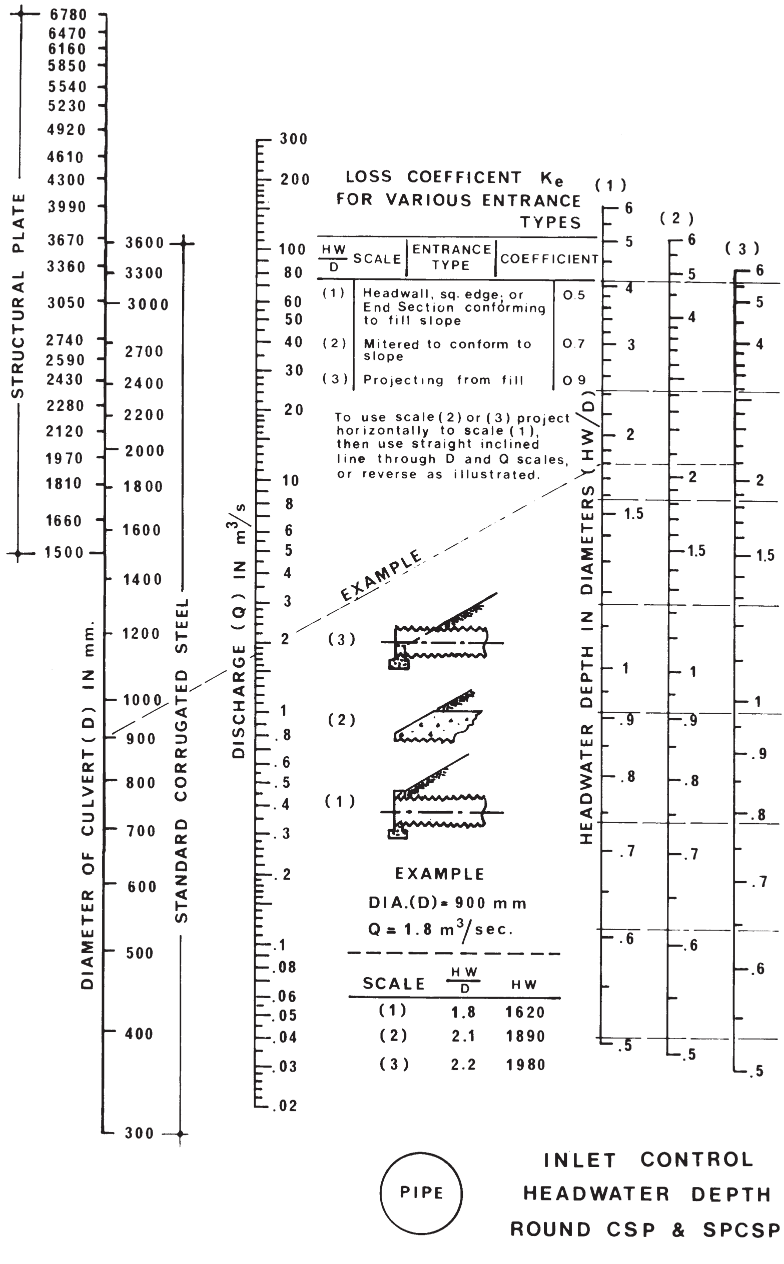


Figure 4.10 Headwater depth for round corrugated steel pipe and structural plate corrugated steel pipe under inlet control.

$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B

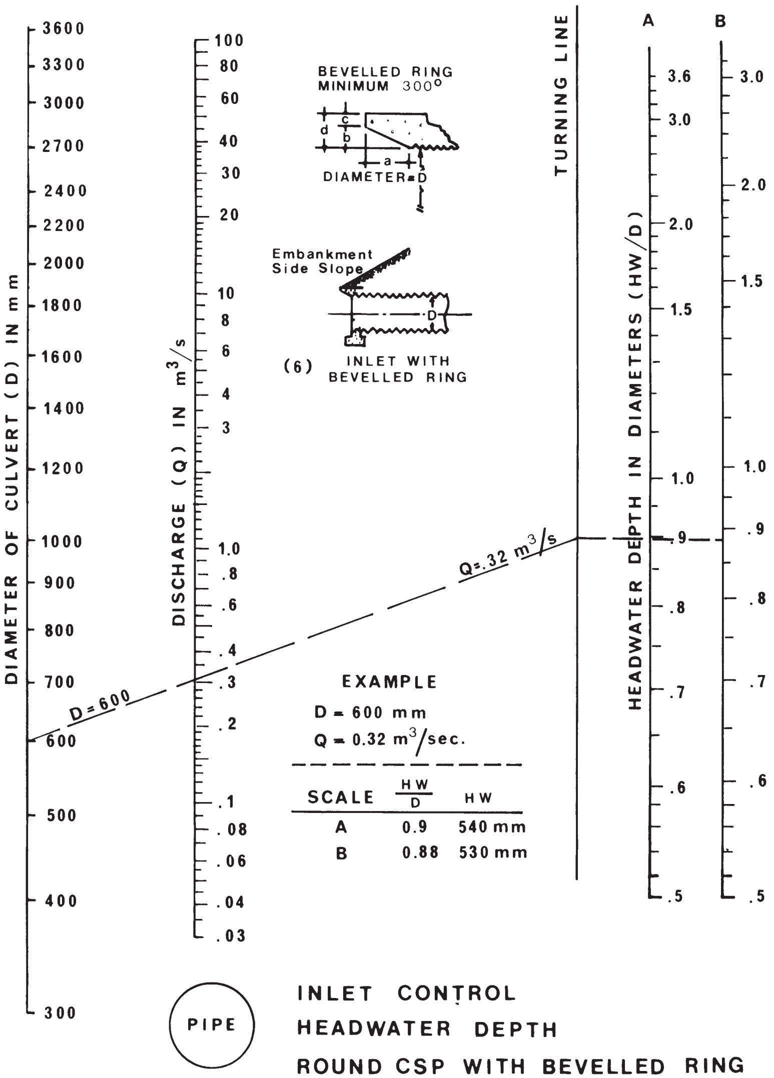


Figure 4.11 Headwater depth for round corrugated steel pipe with bevelled ring headwall under inlet control.

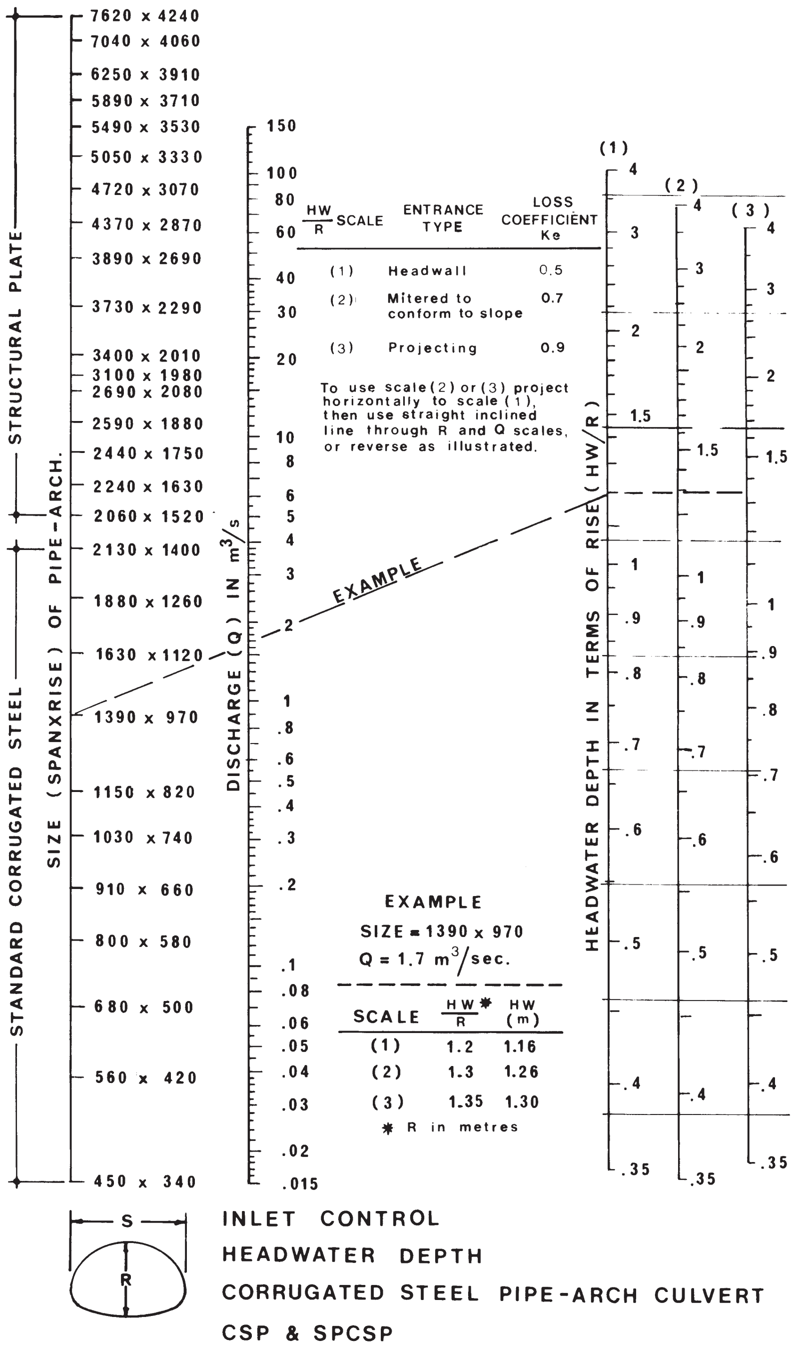


Figure 4.12 Headwater depth for corrugated steel and structural plate corrugated steel pipe-arch under inlet control.

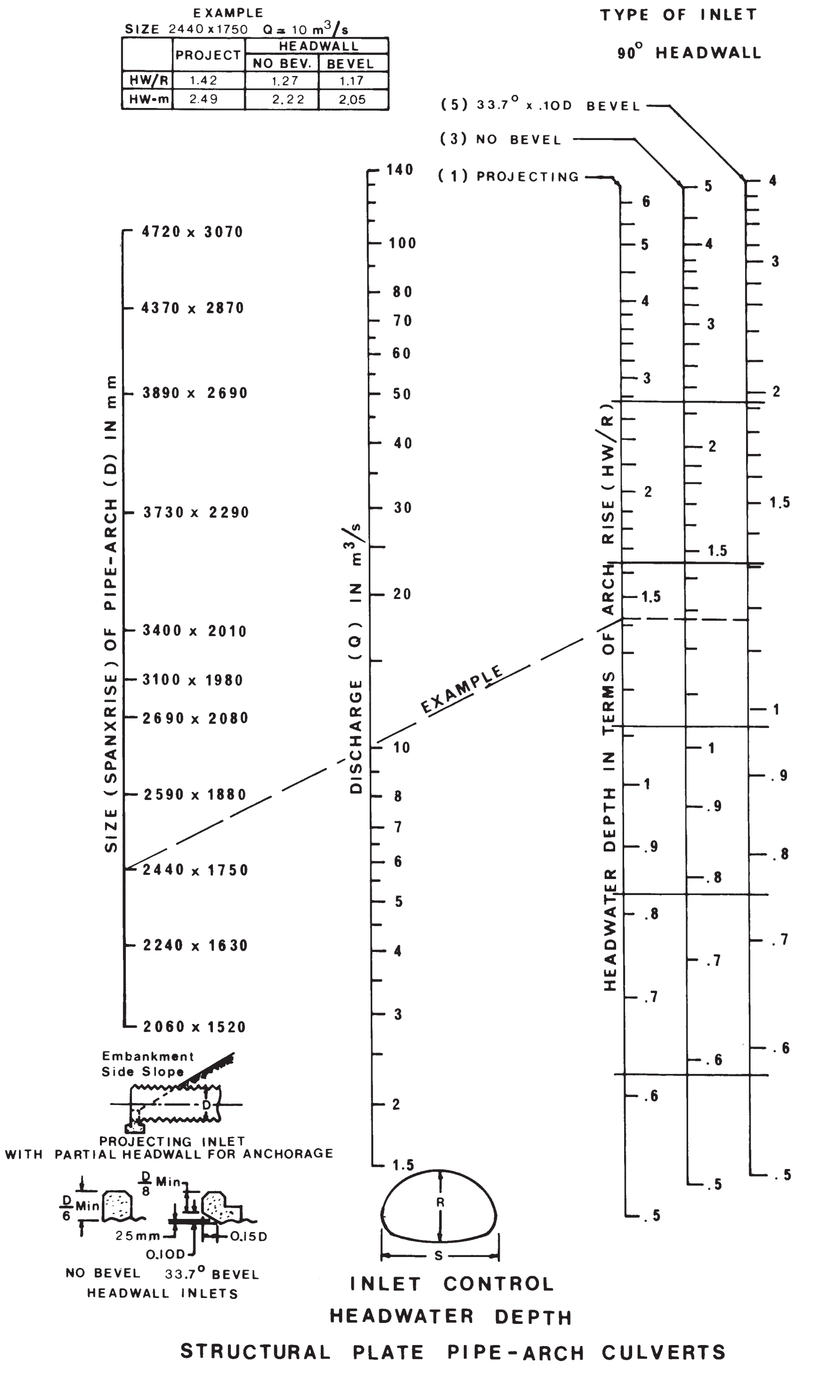


Figure 4.13 Headwater depth for structural plate corrugated steel pipe-arch under inlet control (size range: up to 4720 mm x 3070 mm).

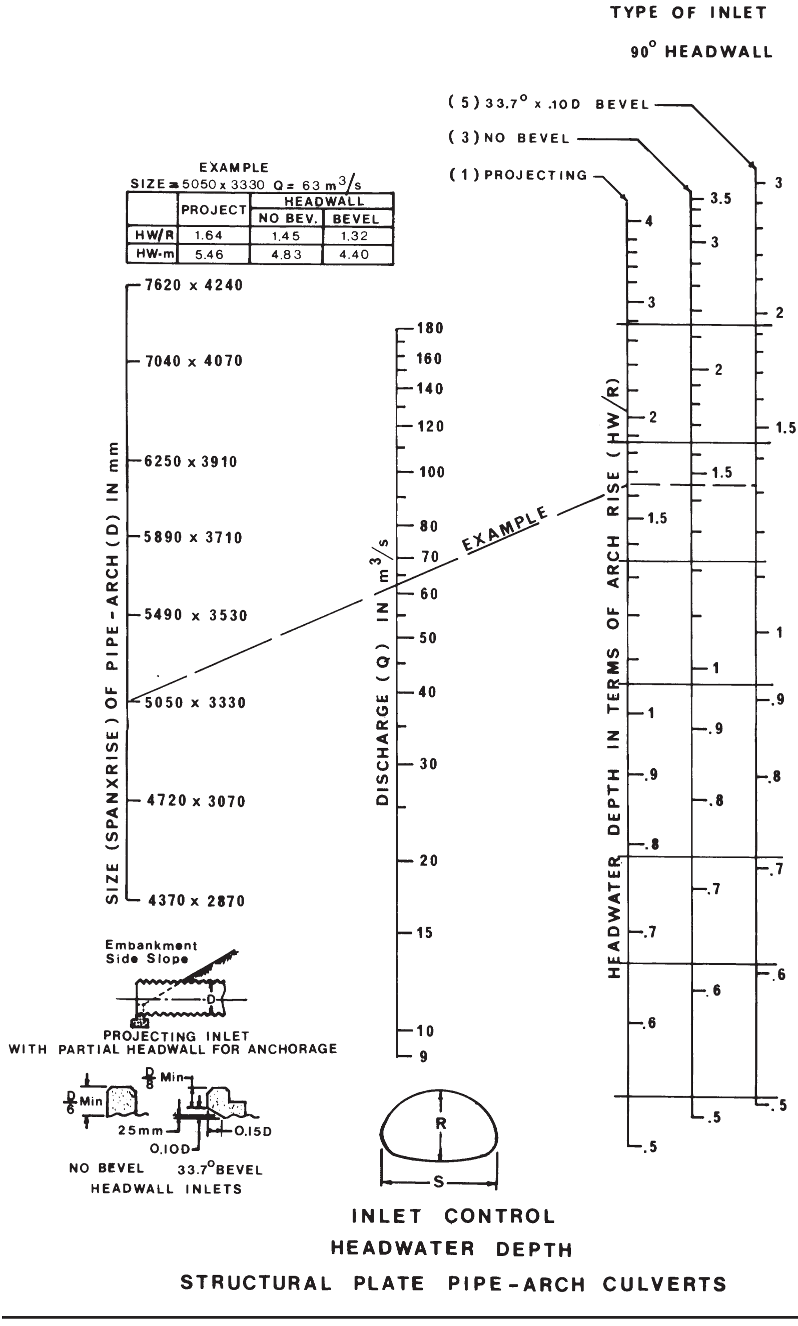


Figure 4.14 Headwater depth for structural plate corrugated steel pipe-arch under inlet control (size range: 4370 mm x 2870 mm and over).

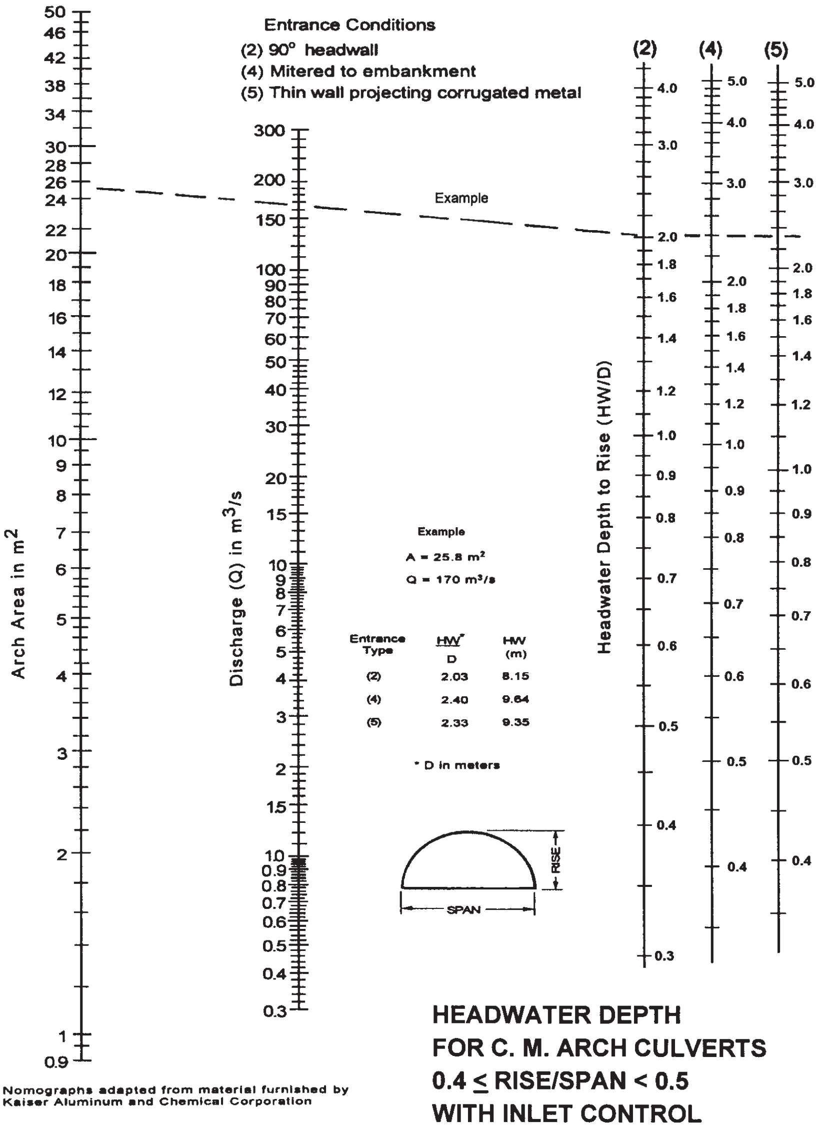


Figure 4.15 Headwater depth for structural plate corrugated steel arch with $0.4 \le$ rise/span < 0.5, under inlet control.

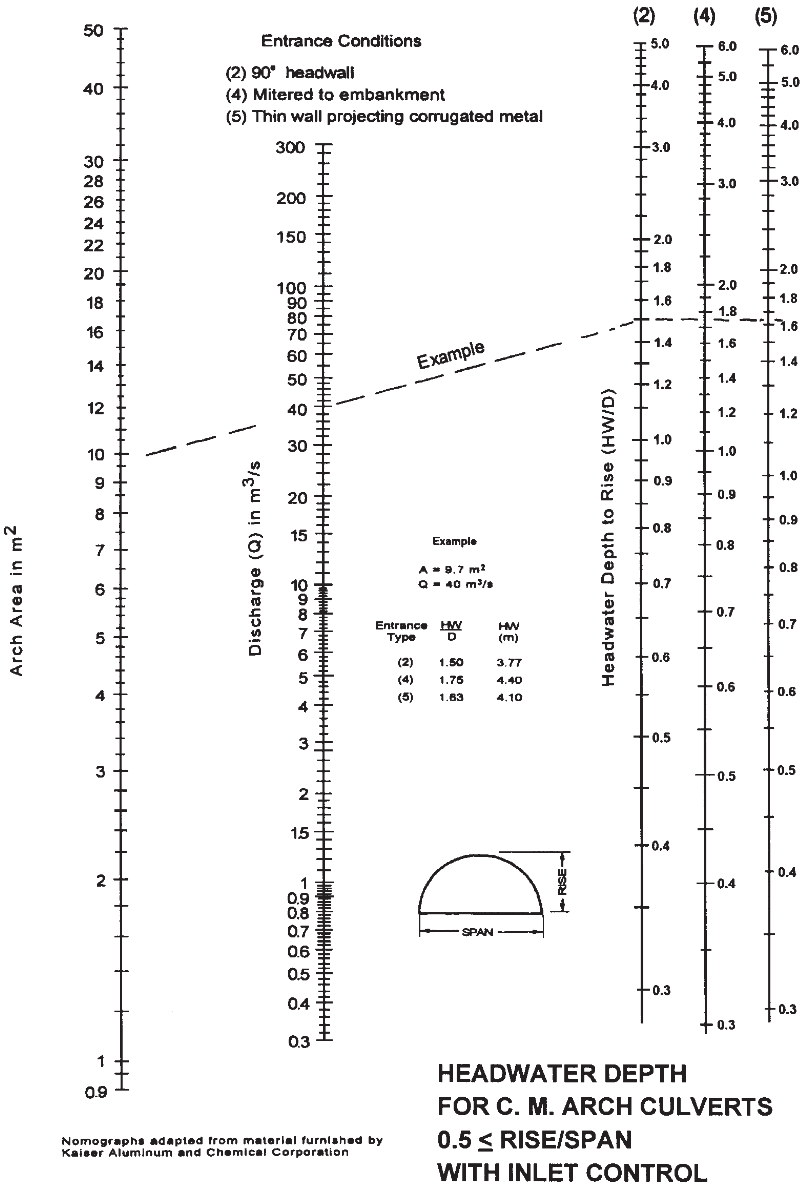


Figure 4.16 Headwater depth for structural plate corrugated steel arch with 0.5 ≤ rise/span, under inlet control.

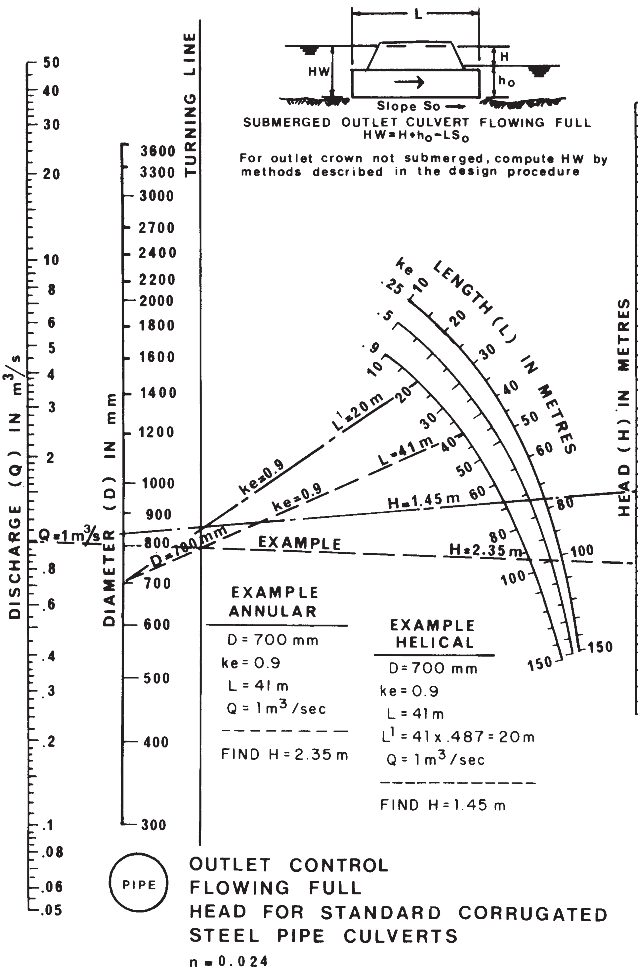


Figure 4.17 Head for round corrugated steel pipe flowing full under outlet control.

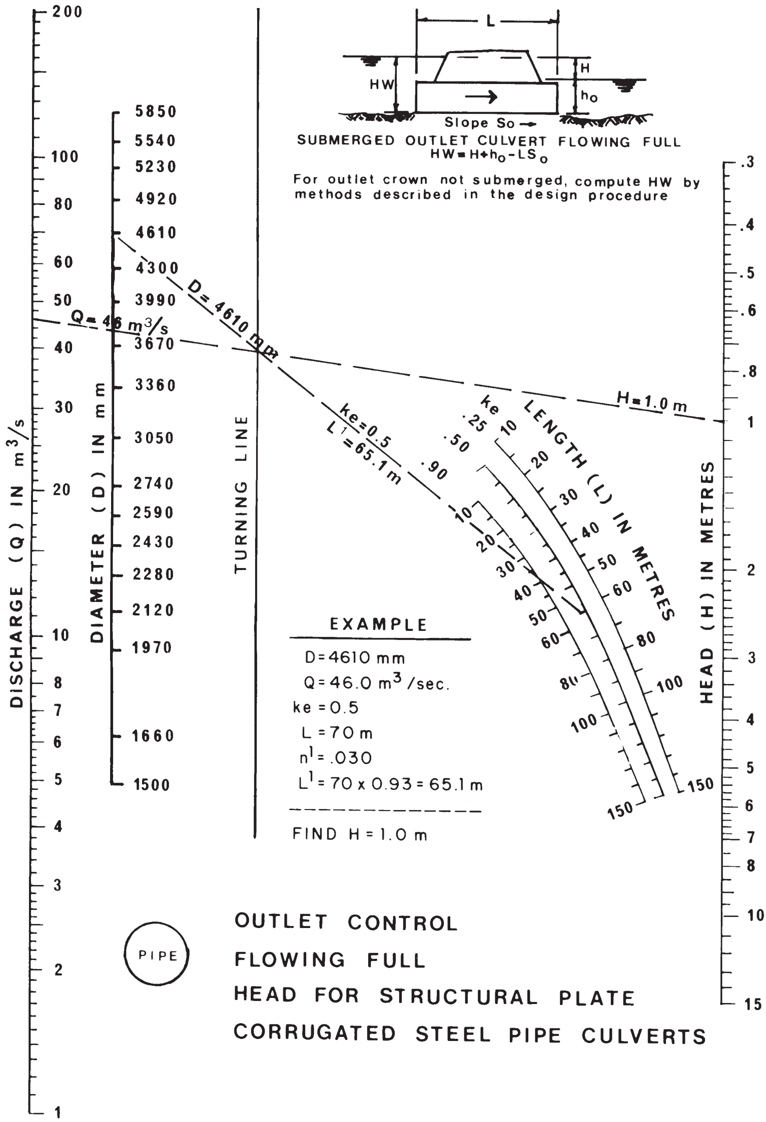


Figure 4.18 Head for round structural plate corrugated steel pipe flowing full under outlet control.

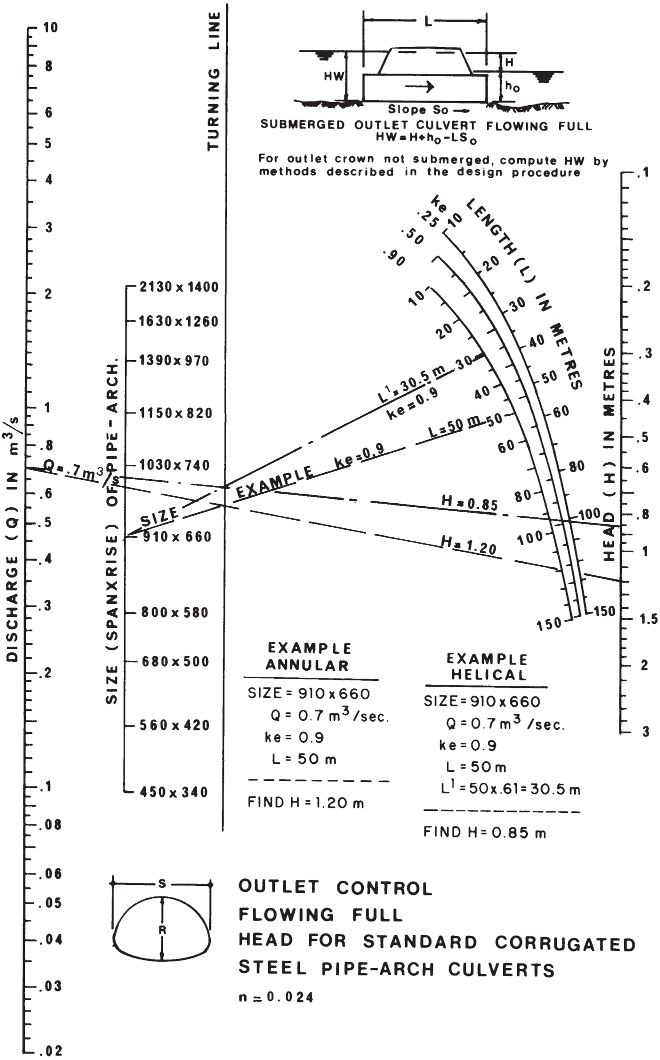


Figure 4.19 Head for corrugated steel pipe-arch flowing full under outlet control.

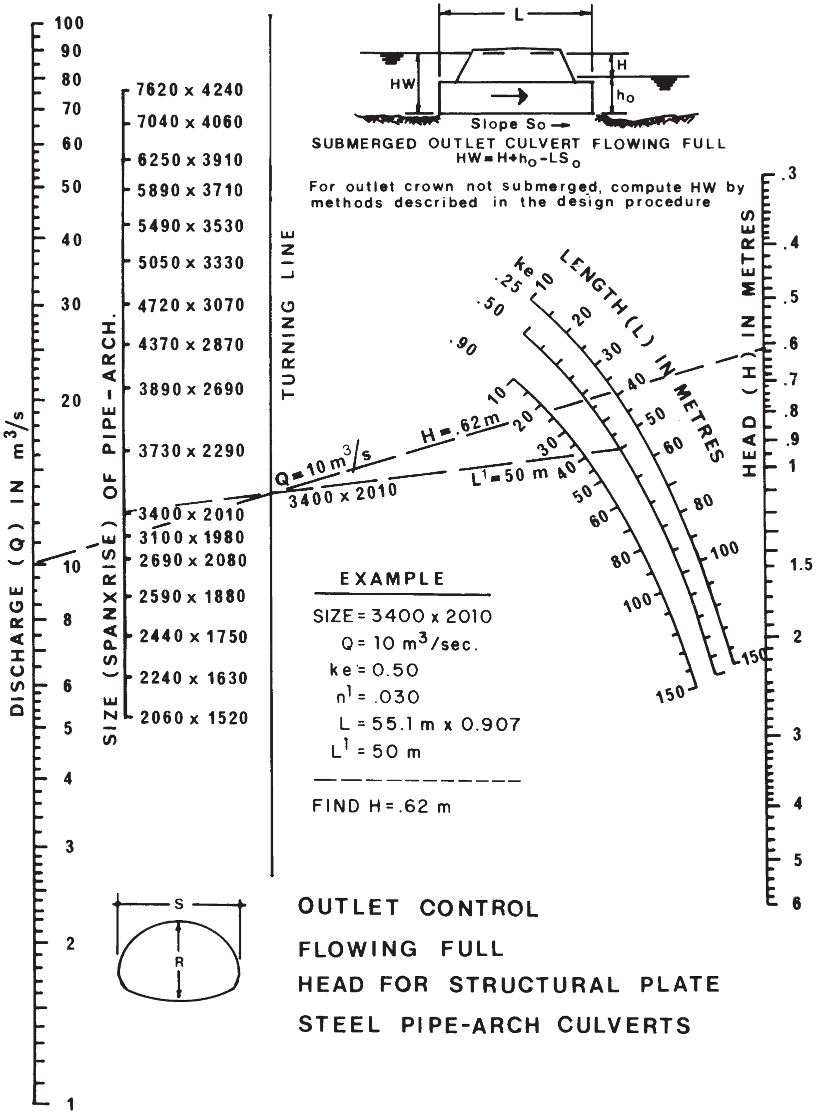


Figure 4.20 Head for structural plate corrugated steel pipe-arch flowing full under outlet control.

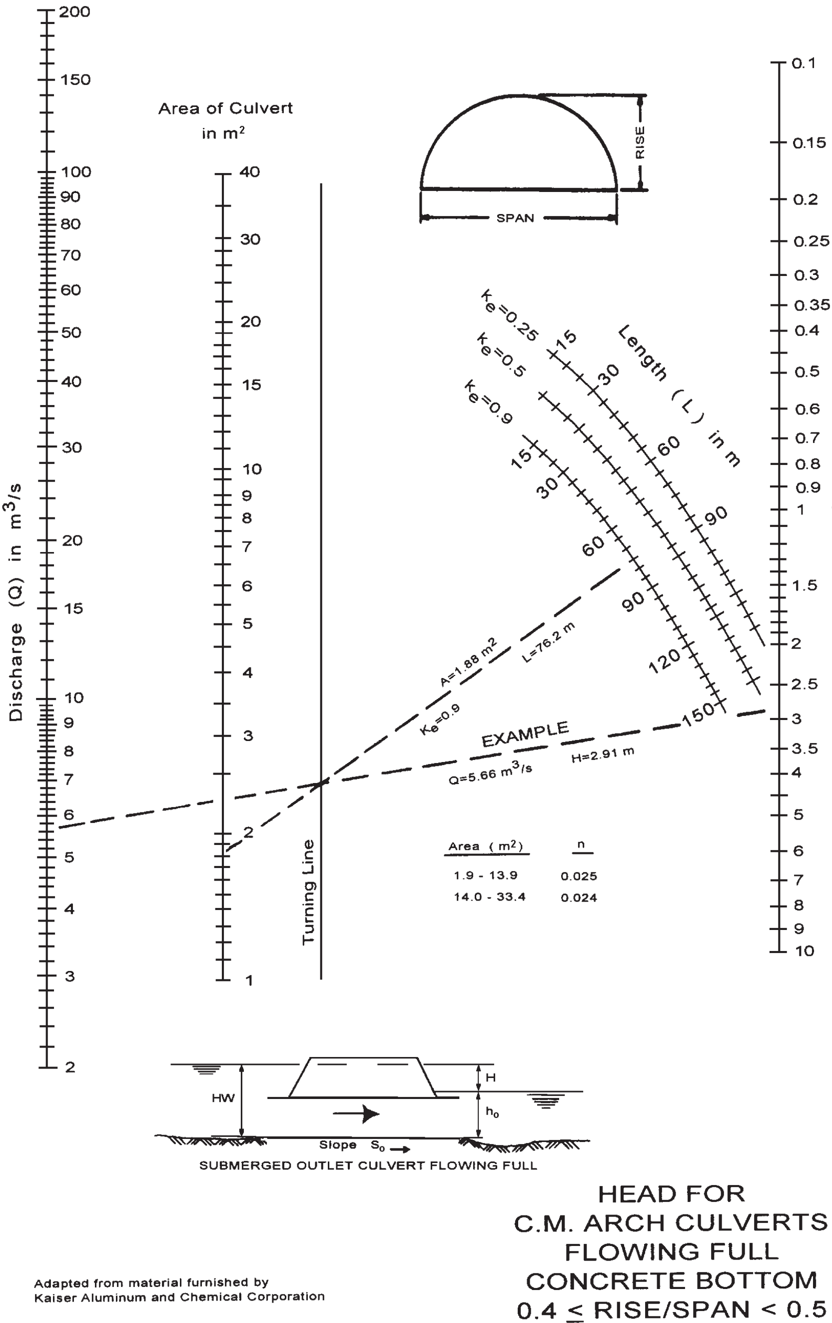


Figure 4.21 Head for structural plate corrugated steel arch with concrete bottom and $0.4 \le$ rise/span < 0.5, flowing full under outlet control.

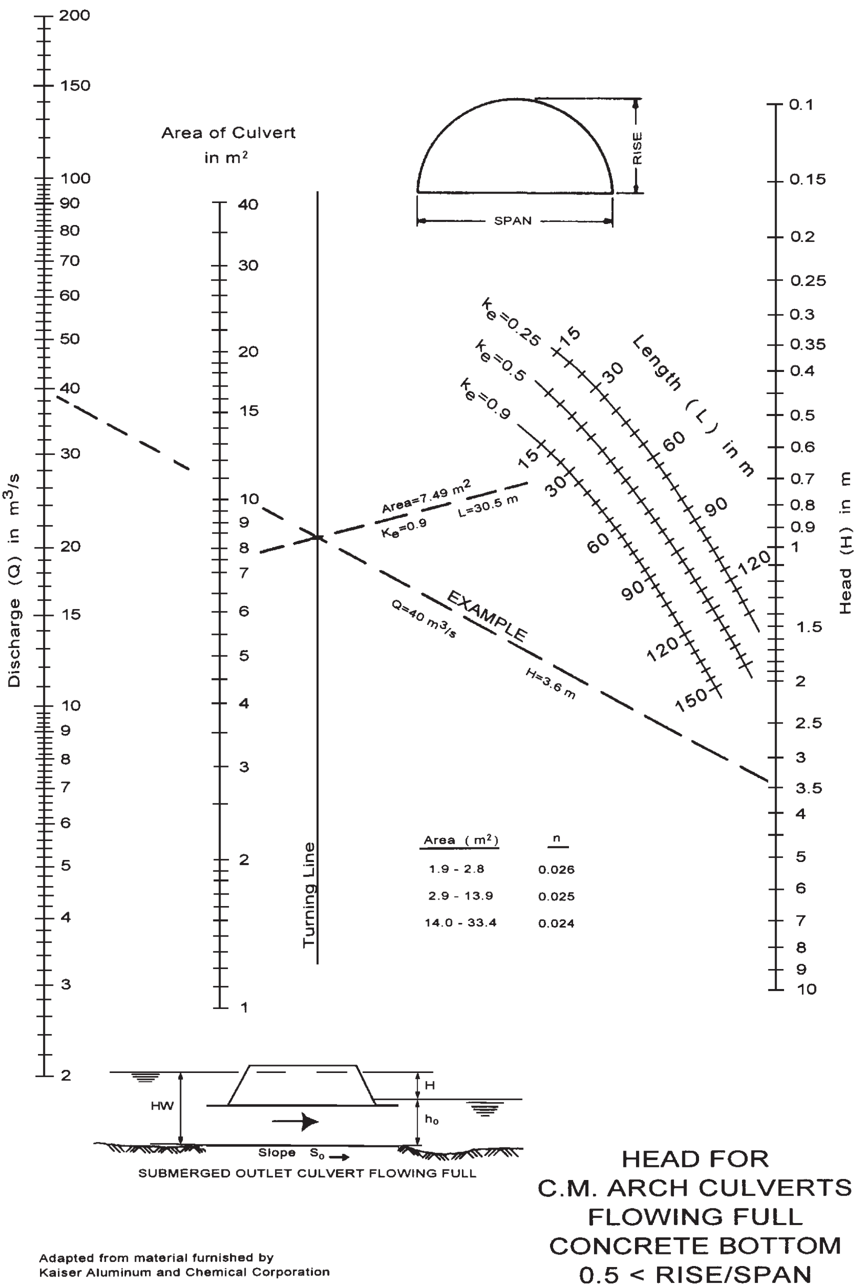


Figure 4.22 Head for structural plate corrugated steel arch with concrete bottom and 0.5 ≤ rise/span, flowing full under outlet control.

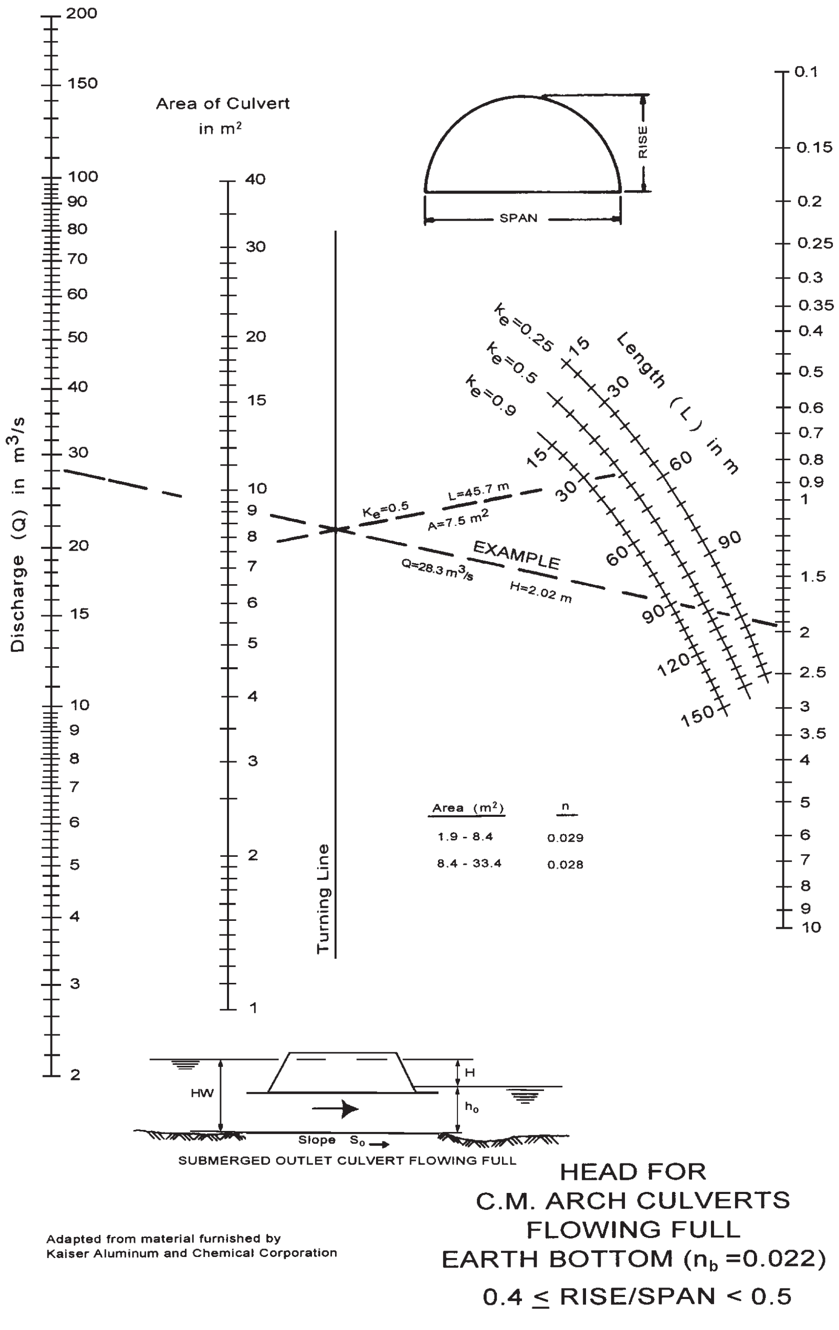


Figure 4.23 Head for structural plate corrugated steel arch with earth bottom and $0.4 \le$ Rise/Span < 0.5, flowing full under outlet control.

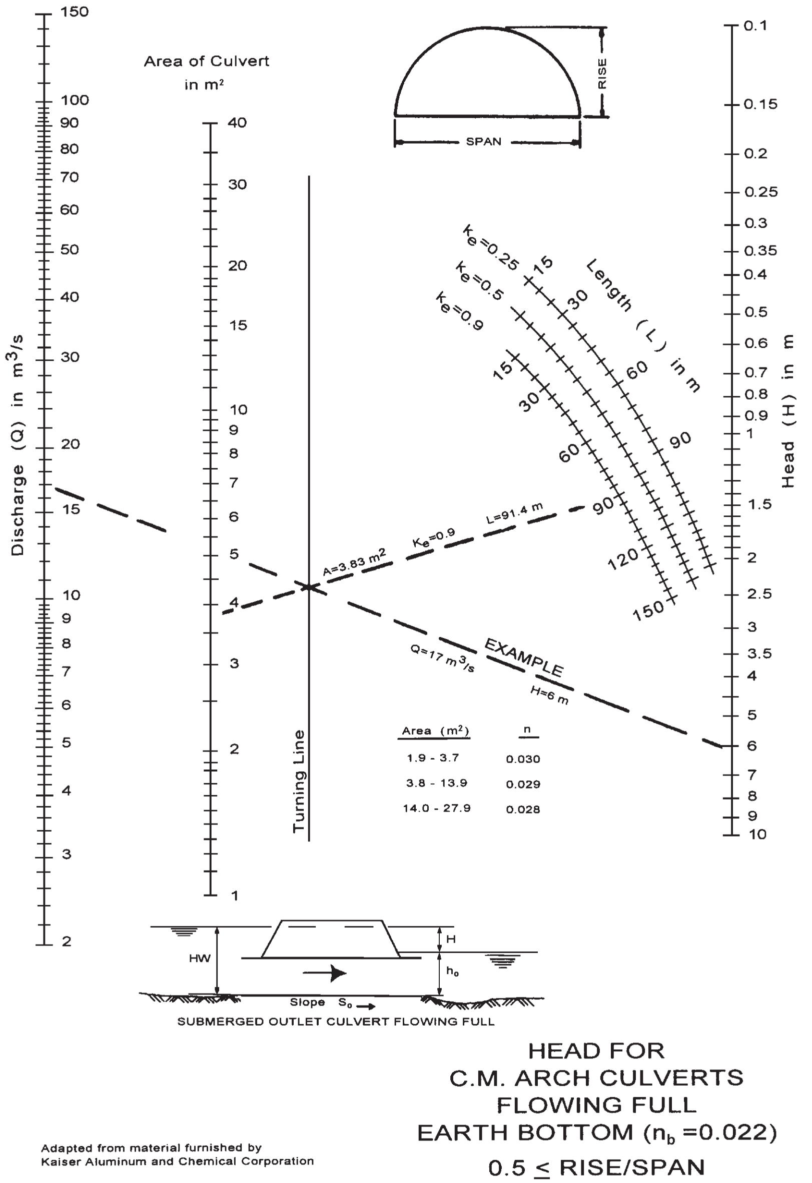


Figure 4.24 Head for structural plate corrugated steel arch with earth bottom and $0.5 \leq$ Rise/Span, flowing full under outlet control.

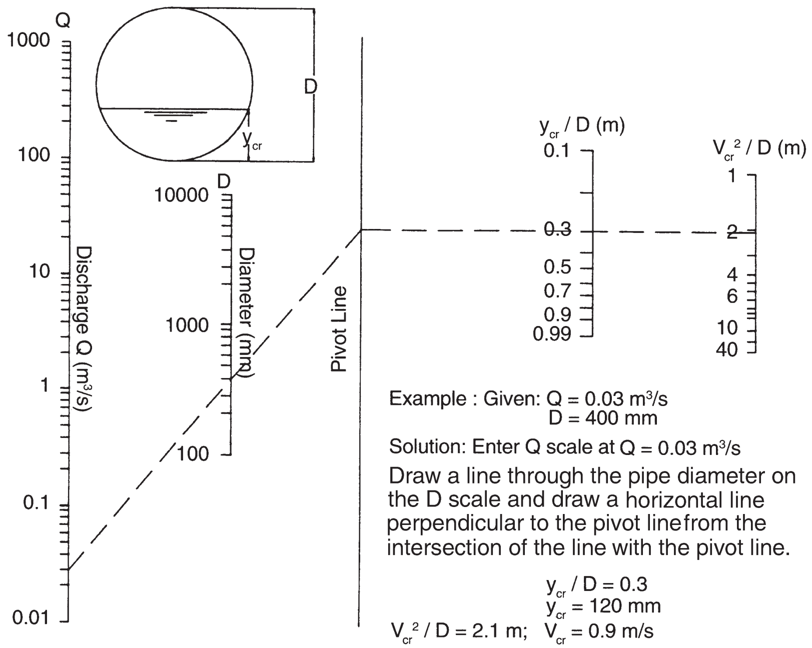


Figure 4.25 Critical depth for round corrugated steel and structural plate corrugated steel pipe.

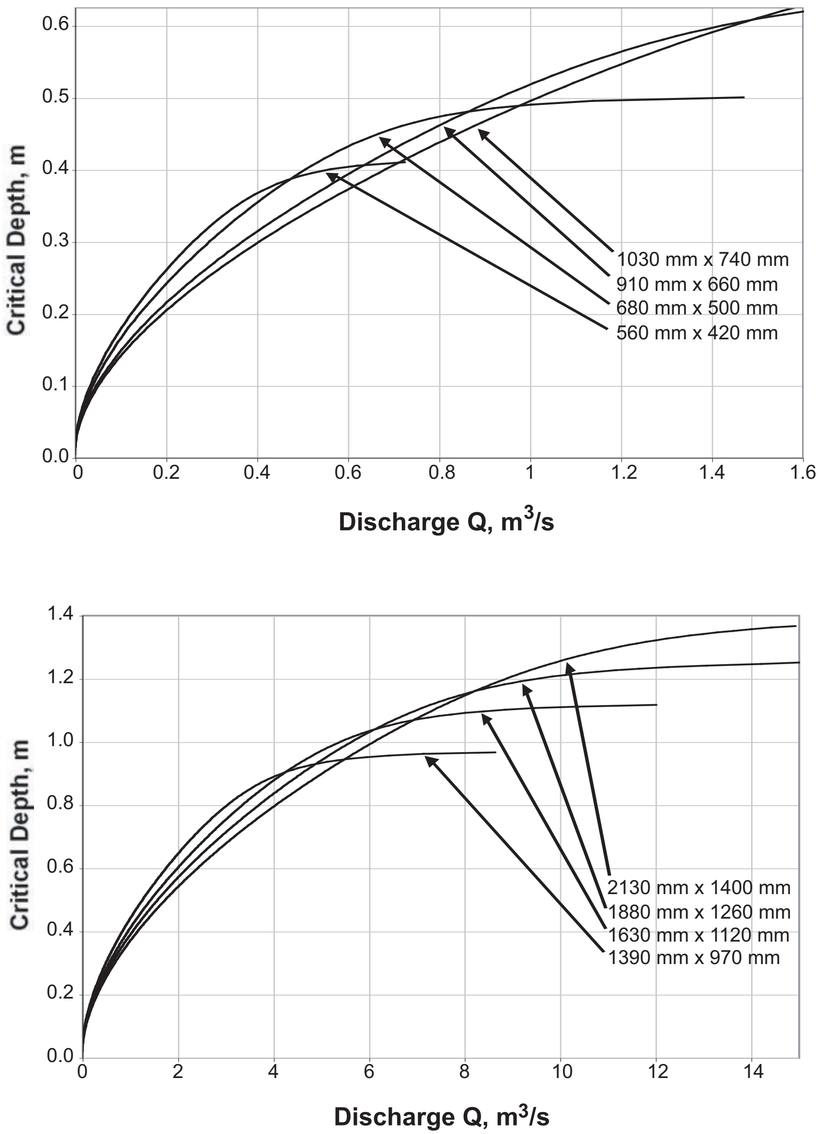
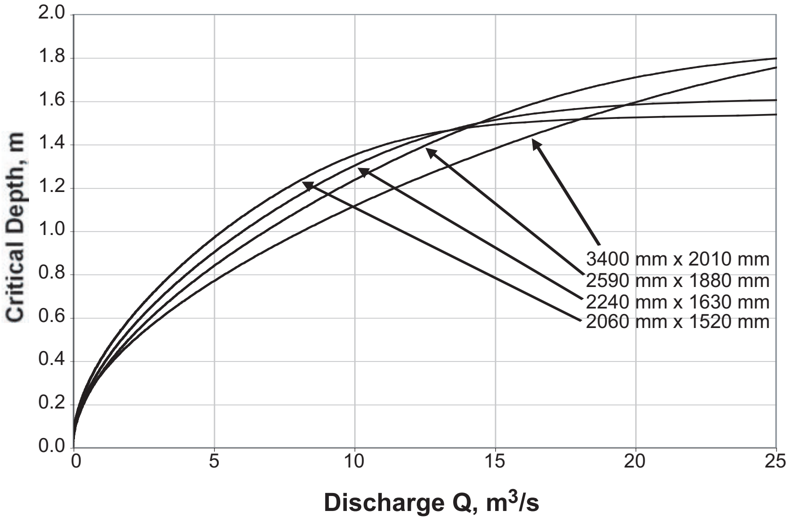


Figure 4.26 Critical depth for corrugated steel pipe-arch.



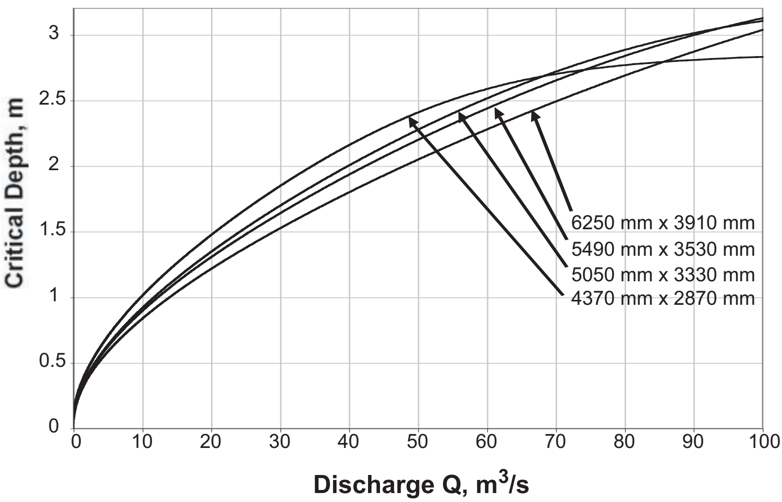


Figure 4.27 Critical depth for structural plate corrugated steel pipe-arch.

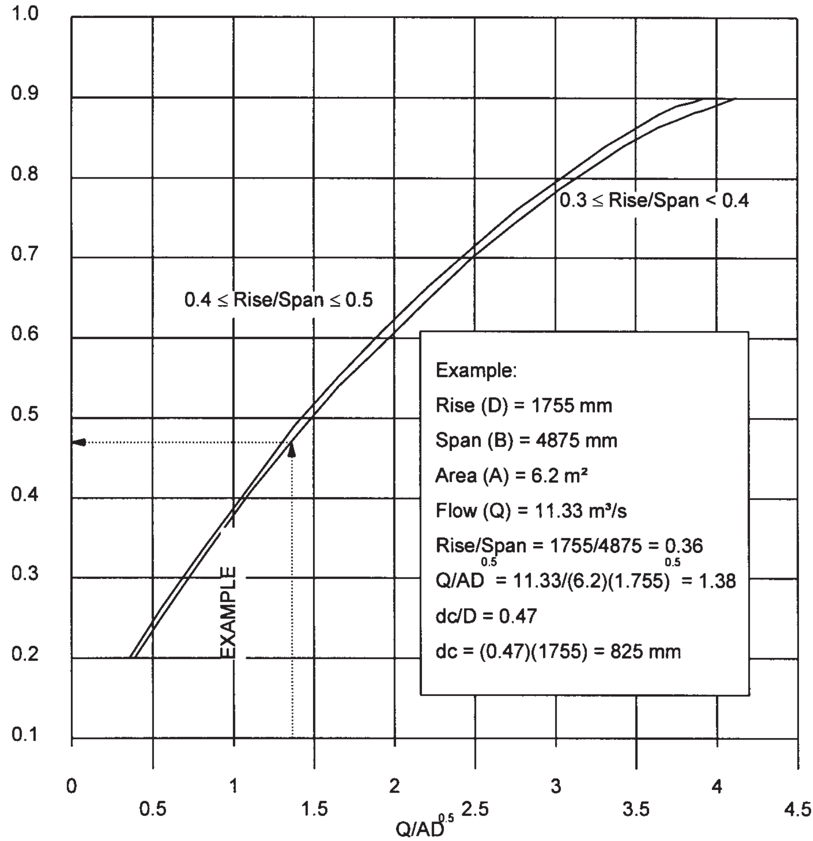
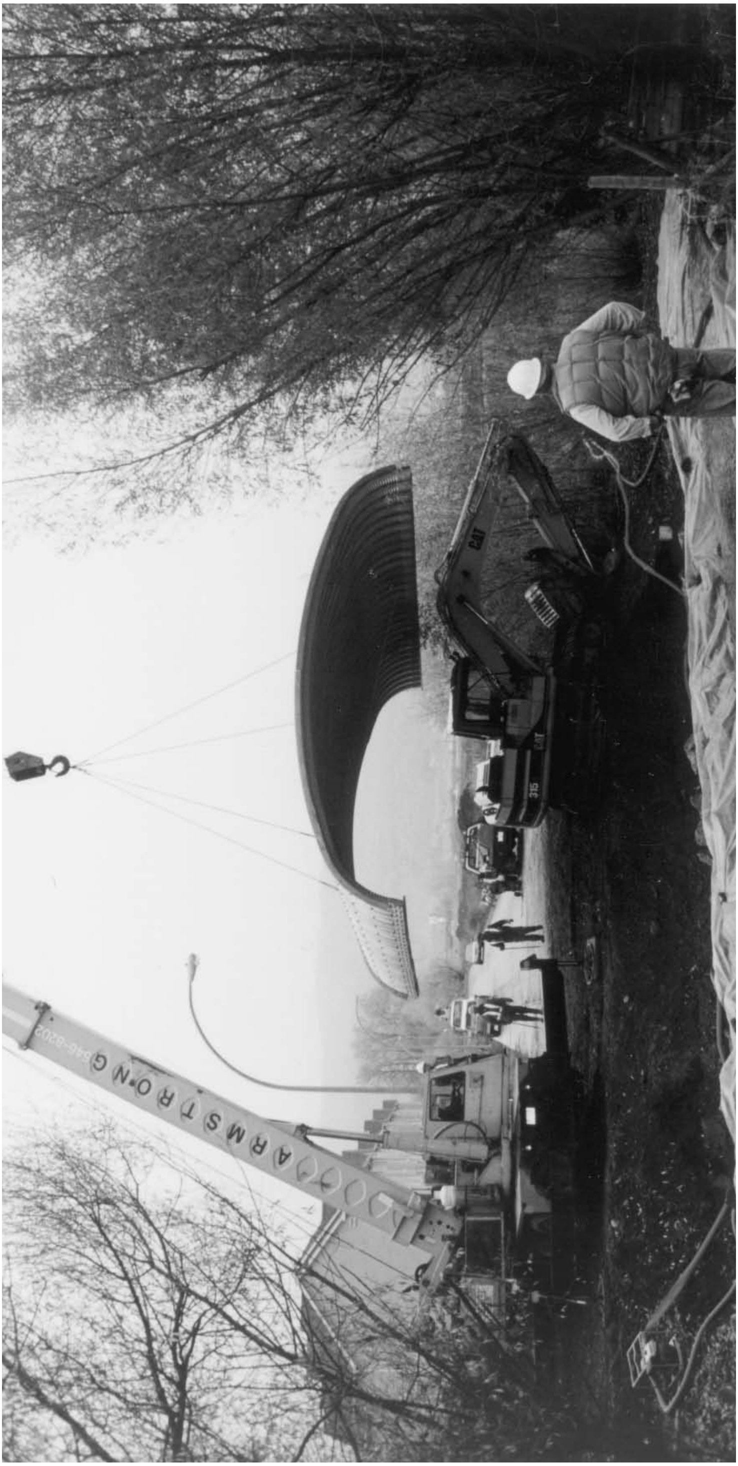


Figure 4.28 Critical depth for structural plate corrugated steel arch.



Long span structure under construction.

HYDRAULICS OF LONG SPAN STRUCTURES

Introduction

Standard procedures are presented here to determine the headwater depth resulting from a given flow through a long span structure under both inlet and outlet control conditions. The most common long span hydraulic shapes are the horizontal ellipse, the low profile arch, and the high profile arch. Useful hydraulic data pertaining to these shapes are presented in tabular and graphic form. Basic hydraulic formulas, flow conditions and definitions have been given previously. However, long span hydraulics include factors which are not considered in the earlier calculations.

Design

Long span structures are often small bridges which span the flood channel. This type of structure ordinarily permits little or no ponding at the inlet. Maximum headwater is usually below the top of the structure. In other words, there is usually some freeboard between the water surface and the top of the structure. This condition is quite different from the ordinary culvert which normally presents a small opening in an embankment crossing a larger flood channel.

The typical long span hydraulic conditions just described maintain effective approach velocity. The following long span hydraulic design procedure considers this approach velocity. The formulas and coefficients taken from the U.S. Federal Highway Administration (FHWA) methodology have been modified to include the approach velocity. In this discussion, headwater, HW, refers to the water surface and not to the energy grade line. This is different than the FHWA procedures, where HW refers to the energy grade line which corresponds to HW + Φ in this discussion.

Design Chart

Inlet control is expected to govern in most long spans. Figure 4.29 allows the designer to conveniently calculate the headwater depth for three standard shapes having the most typical inlet condition. This figure is a plot of the two design equations below (for unsubmerged and submerged inlets), and is based on an inlet that is either a square end with a headwall or a step-beveled end with a concrete collar (Type 1 in Table 4.10). The accuracy of the curves is within the degree to which the graph can be read. Using the design discharge and the structure span and rise, the curve for the structure desired gives the ratio of the headwater depth, approach velocity head and slope correction to the structure rise. The headwater depth is determined by subtracting the velocity head and slope correction from the product of the ratio and the structure rise. Figure 4.29 also includes a table of velocity heads for a variety of approach velocities.

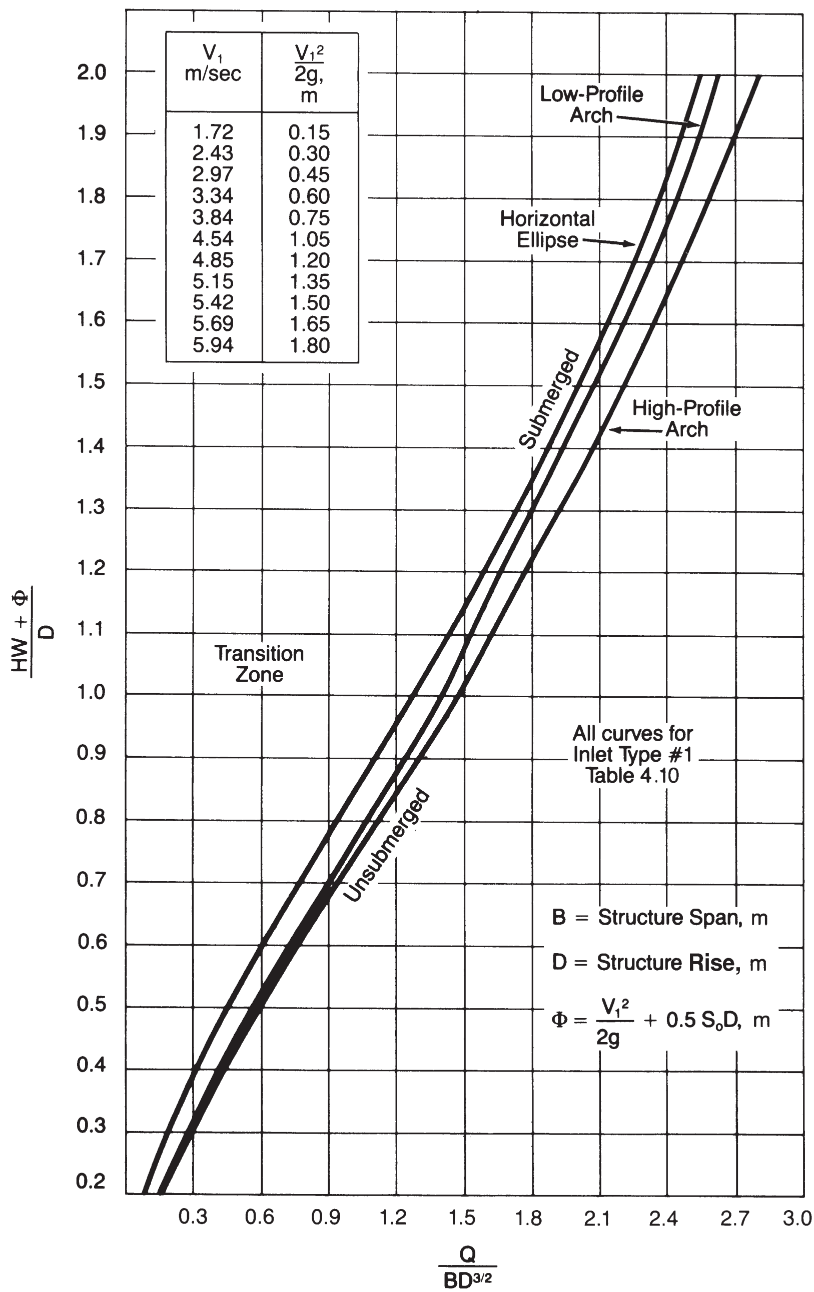


Figure 4.29 Headwater depth for long span corrugated steel structures under inlet control.

Table 4.10 **Inlet loss coefficients for long spans**

Inlet Type	Enhanced Coefficients					$\left\{\frac{Q}{AD^{1/2}}\right\}$	
	k_d	k_p	k	j	k_e	Unsubmerged Maximum	Submerged Minimum
1	0.1243	0.69	0.0272	2.0	0.5	1.82	2.10
2	0.0984	0.74	0.0079	2.5	0.2	1.82	2.32

Notes:
1) Type I inlet is square end with headwall or step-beveled end with concrete collar.
2) Type 2 inlet is square or step-beveled end with mitered edge on headwall.
3) Special improved inlet or outlet configurations can reduce headwater depths.
4) Coefficients k and k_d are not dimensi2nless.

Design Calculations

Inlet Control

The equations for calculating headwater depth for long span structures under inlet control are:

For unsubmerged inlets:

$$HW = H_c + H_e - 0.5\, S_o\, D - \frac{V_1^2}{2g}$$

For submerged inlets:

$$HW = k_d\, D \left\{\frac{Q}{AD^{1/2}}\right\} + k_p\, D - 0.5\, S_o\, D - \frac{V_1^2}{2g}$$

where: HW = headwater depth from the invert to the water surface, m
H_c = critical head, m
H_e = increment of head above the critical head, m
S_o = slope of the structure, m/m
D = rise of the structure, m
V_1 = approach velocity, m/s
g = gravitational constant, 9.806 m/s^2
k_d, k_p = coefficients based on inlet type (Table 4.10)
Q = design discharge, m^3/s
A = full cross-sectional end area of the structure, m^2

To determine if the flow condition is submerged or unsubmerged, the value of $\left\{\frac{Q}{AD^{1/2}}\right\}$ is calculated and reference is made to Table 4.10. If the flow is in the transition zone between unsubmerged and submerged, a reasonable approximation can be made by using both equations and interpolating based on where the value occurs relative to the limits in the table. When a performance curve is plotted, such as in Figure 4.29, the transition zone is filled in manually.

The critical head is equal to the critical depth in the structure at design flow plus the velocity head at that flow:

$$H_c = d_c + \frac{V_c^2}{2g}$$

where: d_c = critical depth, m
V_c = critical velocity, m/s

The critical depth can be interpolated from Tables 4.11 through 4.13. Using the design discharge, the critical depth, as a decimal fraction of the structure rise, is estimated by interpolating between known discharges for a number of set critical depth decimal fractions.

Table 4.11 Hydraulic data for structural plate horizontal ellipse

		Full Flow Data				Discharge – Q (m^3/s) When Critical Depth =					
Span, mm	Rise, mm	Area, m^2	WP, m	R, m	$AR^{2/3}$, $m^{8/3}$	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
1630	1350	1.74	4.717	0.369	0.895	1.255	1.996	2.821	3.792	4.973	6.704
2130	1420	2.41	5.692	0.425	1.362	1.799	2.859	4.011	5.352	6.956	9.288
2900	1930	4.36	7.643	0.570	2.997	3.782	5.928	8.434	11.407	15.172	20.590
3200	2260	5.64	8.618	0.654	4.249	5.295	8.299	11.796	15.940	21.151	28.681
3680	2440	6.85	9.593	0.714	5.472	6.620	10.364	14.819	20.177	26.934	36.467
4420	2790	9.78	11.544	0.847	8.755	10.244	16.101	22.756	30.568	40.178	54.775
4953	3251	12.86	13.007	0.988	12.756	14.838	23.255	33.051	44.661	59.252	80.344
5156	3683	14.87	13.983	1.063	15.488	17.795	27.966	39.627	53.381	70.401	95.650
5715	3988	18.08	15.446	1.170	20.074	22.585	35.670	50.336	67.541	88.457	119.91
6230	3840	18.40	15.938	1.155	20.255	22.326	35.004	49.902	67.657	90.356	122.69
6680	3990	20.49	16.909	1.212	23.292	25.340	39.743	56.725	77.022	103.01	139.87
7010	4290	23.15	17.884	1.294	27.489	29.710	46.584	66.421	90.070	120.31	163.36
7470	4470	25.49	18.859	1.351	31.151	33.271	52.183	74.483	101.13	135.27	183.67
7950	5540	34.25	21.298	1.608	47.009	50.168	78.689	111.73	150.82	199.87	271.35
8280	5820	37.59	22.273	1.687	53.270	56.526	88.678	125.87	169.87	224.93	305.36
8970	6070	42.23	23.736	1.779	62.002	64.763	101.54	144.28	194.9	258.70	351.26
10110	6120	47.57	25.687	1.852	71.739	72.906	114.32	163.07	221.24	295.69	401.48
10640	6500	53.29	27.150	1.963	83.546	84.182	131.99	188.22	255.28	341.05	463.09
11250	7800	68.25	30.076	2.269	117.84	118.66	186.12	264.29	356.79	472.92	642.06
11790	8510	78.31	32.027	2.445	142.12	142.45	223.60	317.20	427.82	565.47	767.49

Table 4.12 Hydraulic data for structural plate low profile arch

		Full Flow Data				Discharge – Q (m^3/s) When Critical Depth =					
Span, mm	Rise, mm	Area, m^2	WP, m	R, m	$AR^{2/3}$, $m^{8/3}$	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
6120	2290	11.18	14.608	0.765	9.351	12.490	18.699	31.558	40.739	51.808	67.973
5920	2080	9.75	13.901	0.701	7.693	9.964	15.085	26.384	34.024	43.237	56.703
6550	2360	12.39	15.544	0.797	10.650	14.606	21.714	35.771	46.139	58.642	76.911
6780	2410	13.01	16.013	0.812	11.323	15.722	23.305	37.985	48.974	62.228	81.600
7010	2440	13.64	16.481	0.827	12.017	16.878	24.953	40.270	51.901	65.928	86.438
7240	2490	14.29	16.949	0.843	12.752	18.074	26.659	42.628	54.919	69.743	91.426
7470	2540	14.94	17.418	0.858	13.489	19.311	28.422	45.060	58.03	73.675	96.565
7670	2570	15.62	17.886	0.873	14.267	20.588	30.245	47.565	61.234	77.724	101.85
7900	2620	16.30	18.354	0.888	15.058	21.906	32.127	50.144	64.532	81.891	107.30
8310	3280	22.04	20.130	1.094	23.400	33.827	49.090	66.715	95.236	121.28	159.27
8760	3350	23.74	21.067	1.127	25.709	37.437	54.205	80.649	104.26	132.72	174.24
9420	3480	26.39	22.472	1.174	29.368	43.197	62.369	91.840	118.61	150.89	198.01
9630	3680	28.69	23.179	1.237	33.060	48.759	70.211	94.988	132.35	168.42	221.06
9860	3730	29.64	23.647	1.253	34.449	50.928	73.281	106.62	137.73	175.24	229.99
10080	3780	30.61	24.116	1.269	35.878	53.145	76.420	110.91	143.23	182.21	239.10
10110	3610	29.15	23.874	1.221	33.300	49.377	71.137	103.81	133.95	170.3	223.39
10490	4040	34.09	25.288	1.348	41.599	61.795	88.615	119.53	164.48	209.26	274.62
10540	3680	31.06	24.814	1.251	36.061	53.736	77.326	112.23	144.74	183.93	241.22
11560	4780	44.30	28.325	1.564	59.688	89.104	127.07	170.48	230.88	293.77	385.53
10770	3730	32.03	25.282	1.266	37.484	55.988	80.526	116.58	150.30	190.96	250.41
11790	4800	45.51	28.793	1.580	61.737	92.194	131.43	176.31	238.49	303.43	398.19

Table 4.13 Hydraulic data for structural plate high profile arch

Span, mm	Rise, mm	Full Flow Data: Area, m^2	WP, m	R, m	$AR^{2/3}$, $m^{8/3}$	Discharge – Q (m^3/s) When Critical Depth = 0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
6300	3680	20.34	17.558	1.158	22.429	26.746	37.644	44.668	56.453	69.591	84.429
6550	3560	20.46	17.878	1.144	22.379	24.665	34.666	40.150	50.502	61.991	74.810
6780	3610	21.36	18.359	1.163	23.622	26.004	36.533	42.028	52.888	64.935	78.382
7010	3660	22.28	18.839	1.182	24.907	27.389	38.465	43.973	55.360	67.99	82.093
7240	3680	23.21	19.318	1.201	26.224	28.821	40.463	45.987	57.920	71.155	85.945
7670	3740	25.09	20.273	1.238	28.927	31.828	44.661	50.224	63.311	77.829	94.086
7870	4655	32.98	22.230	1.484	42.908	41.100	57.882	76.634	86.947	105.88	126.90
8100	4650	34.17	22.718	1.504	44.854	42.896	60.383	79.911	90.075	109.73	131.54
8560	5020	38.74	24.118	1.606	53.128	47.769	67.26	89.035	112.93	122.79	146.64
8590	4630	35.51	23.524	1.509	46.717	41.377	58.155	76.855	97.333	103.06	123.00
9220	4920	40.28	25.135	1.602	55.148	52.619	73.941	97.696	107.09	130.73	156.93
9450	4970	41.53	25.615	1.621	57.308	54.719	76.871	101.54	110.78	135.29	162.46
9680	5260	45.25	26.537	1.705	64.580	58.103	81.670	107.93	136.70	144.89	173.35
9910	5280	46.58	27.017	1.724	66.971	60.327	84.774	112.01	141.83	149.68	179.15
10360	5380	49.28	27.976	1.761	71.864	64.938	91.210	120.46	131.08	159.63	191.21
10360	5830	54.58	28.864	1.891	83.463	67.355	94.744	125.30	158.81	195.08	203.30
11350	6910	69.09	32.061	2.155	115.26	121.36	171.01	205.53	259.30	319.27	386.82
10570	5440	50.65	28.454	1.780	74.392	67.326	94.545	124.84	135.25	164.80	197.49
10590	5870	56.07	29.346	1.910	86.315	69.778	98.125	129.74	164.39	176.33	209.47
11580	6930	70.85	32.554	2.176	118.97	124.78	175.78	210.35	265.38	326.70	395.74

The critical velocity is calculated by dividing the design discharge by the partial flow area corresponding to the critical depth. The partial flow area can be determined from Figures 4.30 through 4.32 using the critical depth as a percentage of the structure rise. The partial flow area is the product of the proportional value from the figure and the full cross sectional area of the structure. The critical velocity is then:

$$V_c = \frac{Q}{A_c}$$

where: A_c = partial flow area based on the critical depth, m^2

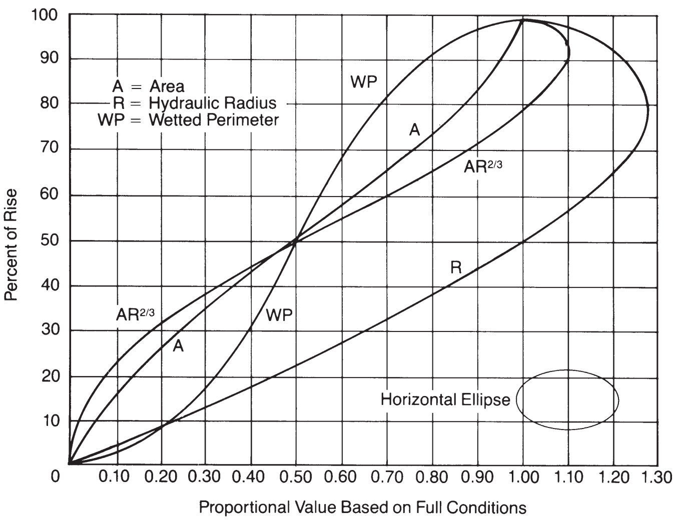


Figure 4.30 Hydraulic properties of long span horizontal ellipse.

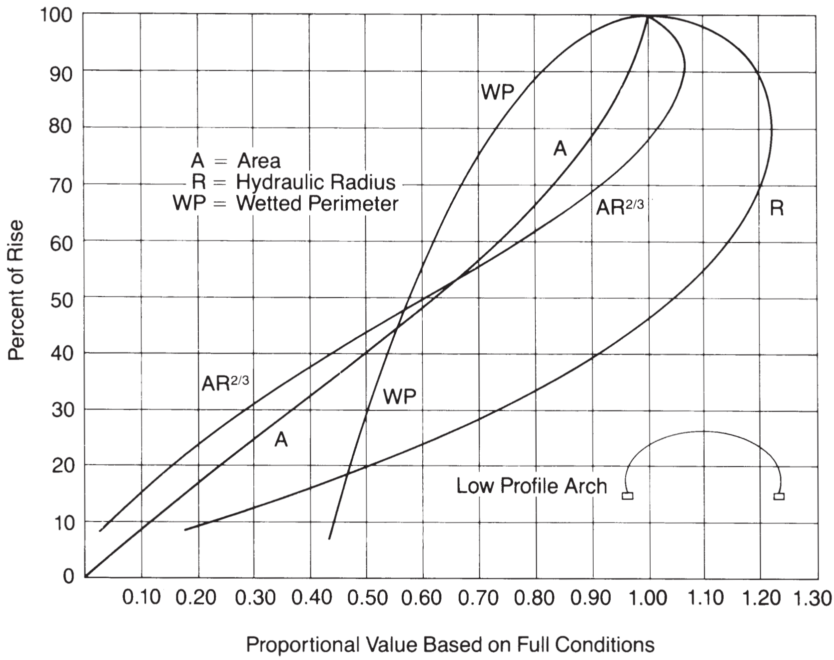


Figure 4.31 Hydraulic properties of long span low profile arch.

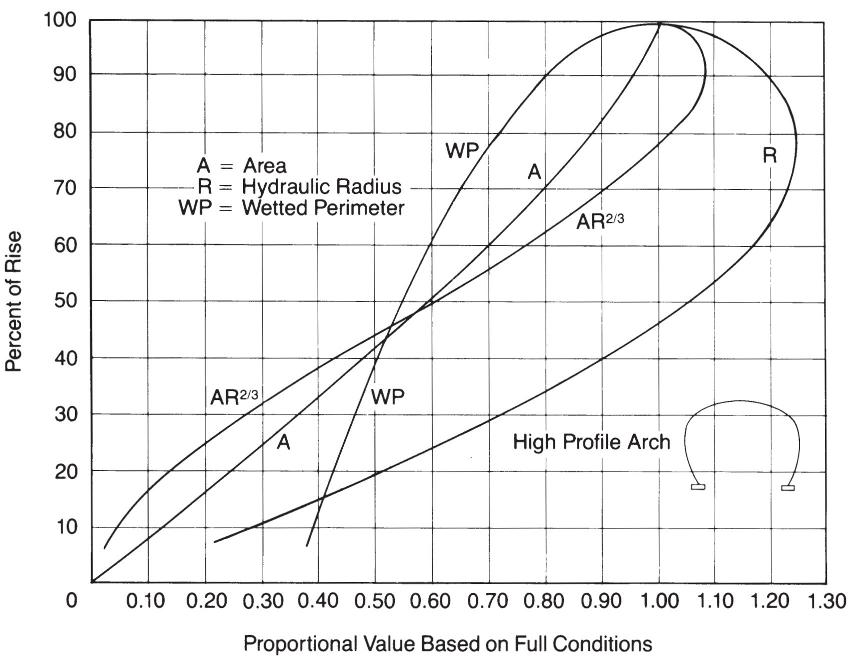


Figure 4.32 Hydraulic properties of long span high profile arch.

The accuracy of the critical depth may be checked using the basic formula for critical flow:

$$Q_c = \sqrt{\frac{gA_c^3}{T_c}}$$

where: T_c = width of the water surface for the critical depth case, m

For this calculation, detailed structure cross section geometry is required in order to calculate the water surface width when the water depth is the critical depth.

The increment of head above the critical head is:

$$H_e = k\,D \left\{\frac{Q}{AD^{1/2}}\right\}^j$$

where: k, j = coefficients based on inlet type (Table 4.10)

Outlet Control

Free Water Surface

The situation where a long span has a free water surface extending through its full or nearly full length, as shown in Figure 4.5 D (possibly the most common flow condition), exists when the headwater depth is less than:

$$D + (1 + k_e)\,\frac{V_c^2}{2g}$$

where: k_e = entrance loss coefficient based on inlet type (Table 4.10)

Under this condition, the headwater depth must be determined by a backwater analysis if accurate results are required. Datum points d_1 and d_2 are established upstream and downstream from the structure, beyond the influence of the entrance and outlet. The backwater analysis determines the water surface profile by starting at the downstream point and moving to the upstream point. The backwater analysis must consider channel geometry between the downstream point and the outlet end of the structure, outlet loss, changing geometry of flow within the structure, inlet loss, and conditions between the inlet end of the structure and the upstream point.

Long span hydraulic properties are provided in Tables 4.11 through 4.13 and Figures 4.30 through 4.32. Entrance loss coefficients are in Table 4.10. The exit loss for these types of structures is typically very small and is often assumed to be zero.

Backwater analyses are considered outside the scope of this handbook. There are references that provide guidance for this procedure. In particular, the FHWA's "Hydraulic Design of Highway Culverts" CDROM contains a discussion and example of the backwater analysis procedure.

Full Flow

When full flow or nearly full flow exists, the headwater depth is determined by the formula:

$$HW = \left(k_e + \frac{2gn^2\,L}{R^{4/3}} + 1\right)\frac{V^2}{2g} + h_o - L\,S_o - \frac{V_1^2}{2g}$$

where: HW = headwater depth, m
k_e = entrance loss coefficient (Table 4.10)
g = gravitational constant = 9.806 m/s^2
n = Manning's friction factor (Table 4.7)
L = length of long span, m
R = hydraulic radius, m = $\frac{A}{WP}$
A = full cross sectional area of the long span, m^2
WP = perimeter of the long span, m
V = velocity, m/s
h_o = outlet datum, m
S_o = slope of structure, m/m
V_1 = approach velocity, m/s

These conditions are as shown in Figure 4.5 A through C. They occur when the headwater depth is greater than:

$$D + (1 + k_e)\ \frac{V_c^2}{2g}$$

For arches or lined structures, a composite Manning's n value must be developed. A method described in an FHWA document is based on the assumption that the conveyance section can be broken down into a number of parts with associated wetted perimeters and Manning's n values. Each part of the conveyance section is then assumed to have a mean velocity equal to the mean velocity of the entire flow section. These assumptions lead to:

$$n = \left[\frac{\sum_{i=1}^{G} (p_i n_i^{1.5})}{p} \right]^{0.67}$$

where: n = weighted Manning's n value
G = number of different roughnesses in the perimeter
p_i = wetted perimeter influenced by material i, m
n_i = Manning's n value for material i
p = total wetted perimeter, m

In the case of arches, the wetted perimeter used in hydraulic radius calculations includes that portion of the structure above the natural channel and the natural channel itself.

For flow conditions as shown in Figure 4.5 A and B, when the tailwater depth is equal to or greater than the structure rise:

$$h_o = TW$$

For flow conditions as shown in Figure 4.5 C, when the tailwater depth is less than the structure rise:

$$h_o = \frac{d_c + D}{2} \text{ or TW (whichever is greater)}$$

The velocity, V, is determined by dividing the design discharge by the area, where the area is the full cross sectional area of the long span structure.

The remaining terms in the equation can be determined as previously discussed.

Summary of Procedure

Step 1. Collect all available information for the design. This includes the required design discharge, the structure length and slope, an allowable headwater elevation or depth, the average and maximum flood velocities in the channel, the proposed entrance type, and a desired structure shape.

Step 2. Select an initial structure size. This may be an arbitrary choice, or estimated using a maximum allowable velocity. To estimate a structure size, the minimum structure end area is determined by dividing the design discharge by the maximum allowable velocity. Geometric constraints may also influence the choice of an initial structure size. An example of this is where a minimum structure span is required to bridge a channel.

Step 3. Use Figure 4.29 and the design parameters to obtain a value for HW + Φ and then the headwater depth, HW. When required, more accurate results can be achieved by using the inlet control formulas to calculate the headwater depth.

Step 4. Check the calculated headwater depth against the allowable headwater depth. If the calculated headwater depth is greater than the allowable, select a larger structure and repeat Step 3. If the calculated headwater depth is less than the allowable, this is the resulting headwater depth for the structure selected under inlet control.

Step 5. Calculate $D + (1 + k_e) \dfrac{V_c^2}{2g}$

If this value is greater than the allowable headwater depth, use the backwater curve method to determine the water surface profile through the structure and the headwater depth. If this value is equal to or less than the allowable headwater depth, the full flow formula should be used to determine the headwater depth. The resulting headwater depth is for the structure selected under outlet control.

Step 6. Compare the inlet and outlet control headwater depths and use the larger. If the resulting headwater depth is greater than the allowable, a larger size or different shape structure should be chosen and the procedure repeated. If the headwater depth is significantly less than the allowable, a smaller size can be chosen and the procedure repeated in order to economize on the structure size.

SPECIAL HYDRAULIC CONSIDERATIONS

In addition to flow hydraulics, the drainage designer must consider hydraulic forces and other hydraulic phenomena that may be factors in assuring the integrity of the culvert and embankment.

Uplifting Forces

Uplifting forces on the inlet end of a culvert result from a variety of hydraulic factors that may act on the inlet during high flows. These may include; vortexes and eddy currents that cause scour, which in turn undermine the inlet and erode the culvert supporting embankment slope; debris blockage that accentuates the normal flow constriction, creating a larger trapped air space just inside the inlet, resulting in a significant buoyancy force that may lift the inlet; and sub-atmospheric pressures on the inside of the inlet, combined with flow forces or hydraulic pressures on the outside, that may cause the inward deflection of a skewed or beveled inlet, blocking flow and creating the potential for hydraulic uplift.

Buoyancy type failures can be prevented by structural anchorage of the culvert entrance. This anchorage should be extended into the embankment both below and to the sides of the pipe. Cut-end treatment of the culvert barrel in bevels or skews should have hook bolts embedded in some form of slope protection to protect against bending.

Piping

Piping is a hydraulic phenomena resulting from the submersion of the inlet end of a culvert and high pore pressure in the embankment. Hydrostatic pressure at the inlet will cause the water to seek seepage paths along the outside of the culvert barrel or through the embankment. Piping is the term used to describe the carrying of fill material, usually fines, caused by seepage along the barrel wall. The movement of soil particles through the fill will usually result in voids in the fill. This process has the potential to cause failure of the culvert and/or the embankment. Culvert ends should be sealed where the backfill and embankment material is prone to piping.

Weep Holes

These are perforations in the culvert barrel which are used to relieve pore pressure in the embankment. Generally, weep holes are not required in culvert design. For an installation involving prolonged ponding, there may be merit in considering a separate sub-drainage system to relieve pore pressure and control seepage in the embankment.

Anti-Seepage Collars

Vertical cutoff walls may be installed around the culvert barrel at regular intervals to intercept and prevent seepage along the outer wall of the culvert. These may also be referred to as diaphragms. They are most often used in small earth fill dams or levees and are recommended when ponding is expected for an extended time. An example of this is when the highway fill is to be used as a detention dam or temporary reservoir. In such cases, earth fill dam design and construction practices should be considered.

Single vs. Multiple Openings

A single culvert opening is, in general, the most satisfactory because of its greater ability to pass floating debris and driftwood. However, in many cases, the design requires that the waterway be wide in order to get the water through quickly without ponding and flooding of the land upstream. In such cases, the solution may consist of using either an arch, a pipe-arch, or a battery of two or more openings. See Figure 4.33.

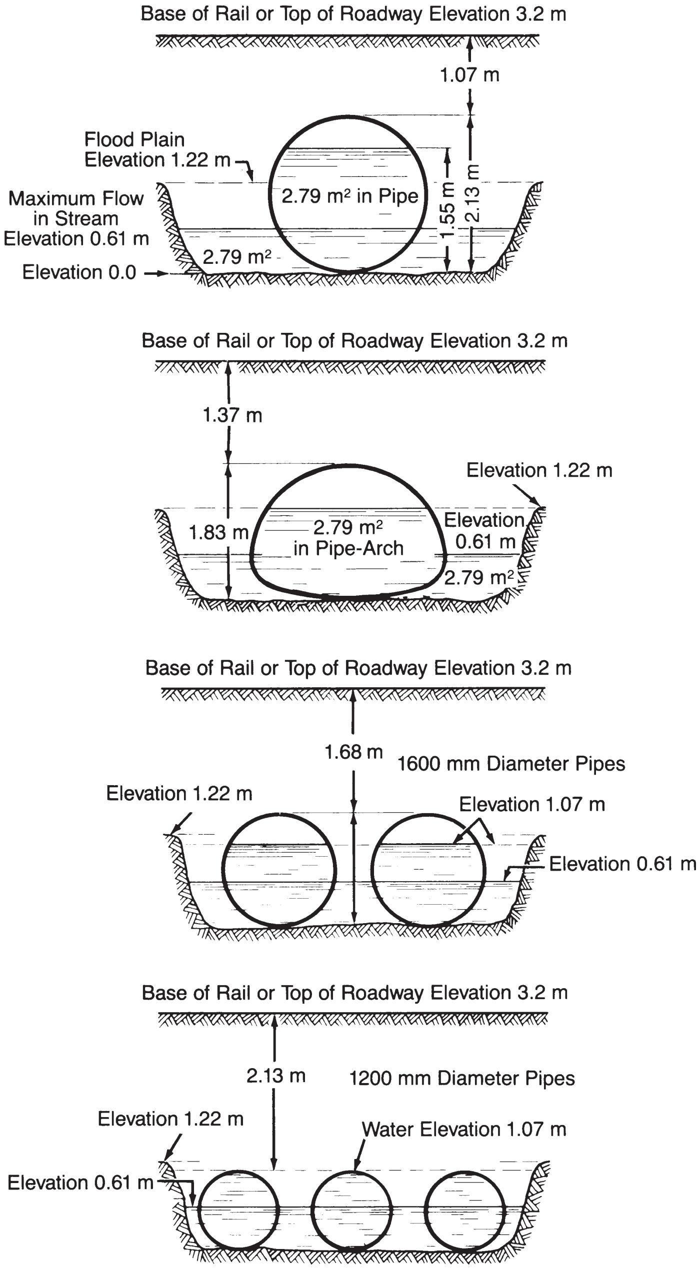


Figure 4.33 Culvert opening choices.

HYDRAULICS OF SUBDRAINS

Free Water

Ground water may be in the form of an underground reservoir or it may be flowing through a seam of pervious material. If it is flowing, it may be seeping or percolating through a seam between impervious strata, or be concentrated in the form of a spring.

Free water moves by gravity. It may consist of storm water seeping through cracks in the pavement or entering the ground along the edges of the road. It may be ground water percolating from a higher water-bearing stratum to a lower one, or from a water-bearing layer into the open as in the case of an excavation. A number of subdrainage applications are discussed in Chapter 1.

Water seeping through cracks in the pavement is especially noticeable in springtime and also visible shortly after rains when the remainder of the road has dried off. Passing traffic pumps some of this water, sometimes mixed with subgrade soil, up through the cracks or joints onto the road surface. This water is harmful because it may freeze on the surface and become an unexpected traffic hazard, and it can also destabilize the road subgrade. It can and should be removed in order to establish a stable subgrade and to prevent potential problems.

Subsurface Runoff Computation

In general, the amount of available ground water is equivalent to the amount of water that soaks into the ground from the surface less the amount that is lost by evaporation and that is used by plants. The nature of the terrain and the catchment area size, shape and slopes, as well as the character and slopes of the substrata, are contributing factors to the amount of ground water available and the volume of subsurface runoff.

A practical way to determine the presence of ground water and the potential flow rate is to dig a trench or test pit. This is helpful especially where an intercepting drain is to be placed across a seepage zone to intercept the ground water and divert the flow, as shown in Figure 4.34.

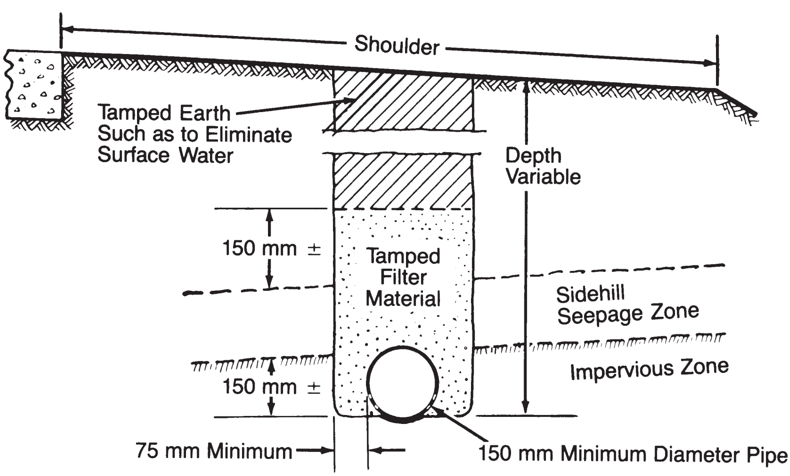


Figure 4.34 Intercepting drain.

Determining a correct size for subdrainage pipe requires an indirect approach. For problems other than those involving large flat areas, size determination becomes a matter of personal judgment and local experience. The following procedure applies to relatively flat areas.

The rate of runoff for average agricultural soils has been determined by agricultural engineering experiment stations to be about 10 mm in 24 hours. For areas of heavy rainfall or more pervious soils, this factor may be increased to 20 or 25 mm. The runoff expressed in mm per 24 hours is converted to $m^3/s/ha$ for design discharge calculations. Table 4.14 provides a conversion table.

Table 4.14 **Constants for subsurface runoff for various soil permeability types Depth of water measured in 24 hours**

Soil Permeability Type	Depth, mm	Quantity of Water per Lateral, $m^3/s/ha \times 10^{-4}$, constant c
Slow to Moderate	2	2.32
Slow to Moderate	4	4.63
Slow to Moderate	6	6.94
Moderate	8	9.26
Moderate	10	11.57
Moderate	12	13.9
Moderate	14	16.2
Moderate to Fast	16	18.5
Moderate to Fast	18	20.8
Moderate to Fast	20	23.1
Moderate to Fast	22	25.5
Moderate to Fast	24	27.8
Moderate to Fast	26	30.1
Moderate to Fast	28	32.5
Moderate to Fast	30	34.7

The design discharge can be calculated using the following formula:

$$Q = CA$$

where:
- Q = discharge or required capacity, m^3/s
- C = subsurface runoff factor, $m^3/s/ha$
- A = area to be drained, ha

Example

Assuming a drainage runoff rate of 10 mm in 24 hours (runoff factor, $C = 11.57 \times 10^{-4}$) and laterals 180 m long spaced on 15 m centers, the following result is obtained:

$$Q = (11.57 \times 10^{-4}) \left\{ \frac{180(15)}{10^4} \right\} = 3.12 \times 10^{-4} \ m^3/s$$

Size of Pipe

The size of pipe can be determined using Manning's formula, or by the use of a nomograph. For standard subdrainage applications, approximately 150 m of 150 mm diameter perforated steel pipe may be used before increasing to the next size.

Where possible, a minimum slope of 0.15 percent should be used for subdrainage lines. It is often permissible to use an even flatter slope to achieve a free outlet, but the steeper slope provides a self-cleansing flow velocity.

Geotextiles

There has been a trend, in recent years, toward the use of geotextiles or filter cloths in lieu of graded aggregate filters. They are used as a filter, to allow the free flow of water into the subdrainage pipe network while preventing fine erodible soils or clogging fines from entering the system, and as a separator, to provide a barrier to soil migration between the surrounding trench wall material and permeable trench backfill.

The diminishing availability and increasing costs of good quality aggregates for graded filters, and the increasing availability and lower cost of geotextiles engineered for these types of applications, has provided the impetus to substitute filter fabrics. Their use also expedites construction and, in many cases, they are used with graded aggregate filters as added insurance against soil migration.

A wide range of filter fabrics are available in a variety of styles and materials. Geotextiles are available as either woven or non-woven products. The properties that are relevant to this application include; permeability, tensile strength, pore size, equivalent opening size (EOS), puncture strength, alkali or acid resistance, freeze-thaw resistance, burst strength, and ultra violet stability.

In the selection of a geotextile, it is important to recognize that the role of the product is as a separator or as a separator and filter. For separation and filtration, the major parameter used for the selection of a filter cloth is the Equivalent or Effective Opening Size (EOS). The choice of fabric EOS must take into consideration the grain size distribution and nature of the soil materials that it is to separate and the desired system permeability. Fabrics for separation and filtration usually have an EOS of between 150 and 200 mm.

A typical cross-section of a filter trench design utilizing filter fabric as a separator/filter is shown in Figure 4.35.

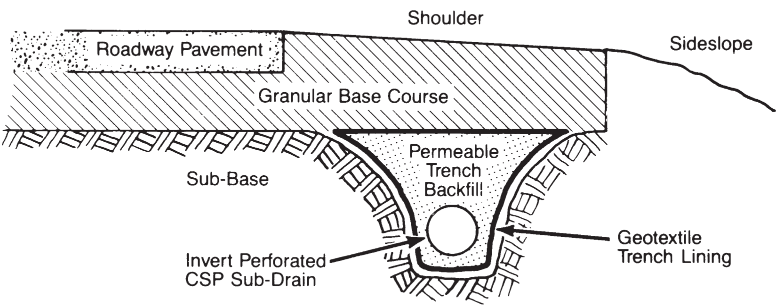


Figure 4.35 Trench drain utilizing geotextile.

HYDRAULICS OF STORM WATER INLETS

Storm water inlets are the means by which storm runoff enters the sewer system. Their design is often neglected, or it receives very little attention, during the design of storm drainage systems.

If inlets are unable to transmit the design inflow into the sewer system, then the system will not be utilized to its hydraulic design capacity. In some cases, though, it may be desirable to limit the inflow into the sewer system as a means of storm water management. In such cases it is imperative that more emphasis be placed on inlet design to assure that the type, location and capacity of the inlet will achieve the overall drainage requirements.

Slotted drain at work in Montreal, Quebec.

No single inlet type is best suited for all conditions. Many different types of inlets have been developed, as shown in Figure 4.36, based on practical experience and rules of thumb. The hydraulic capacities of some of these inlets is often unknown, resulting in erroneous capacity estimates.

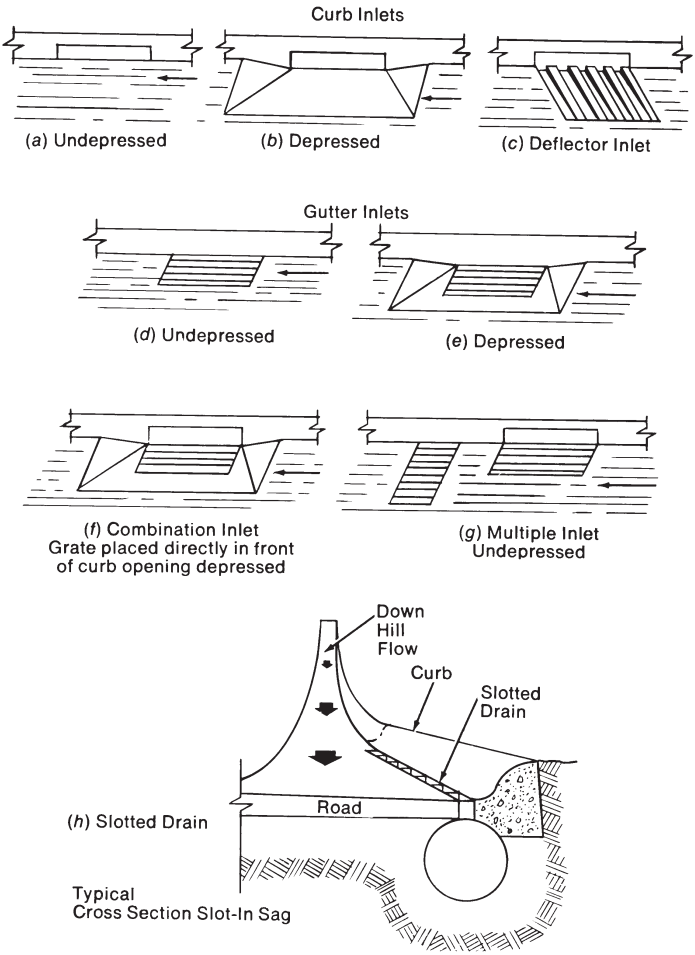


Figure 4.36 Stormwater inlets.

The hydraulic efficiency of inlets is a function of street grade and cross-slope, and inlet and gutter depression geometry. A steeper street cross-slope will increase the depth of flow in the gutter. Depressed gutters concentrate the flow at the inlets. The depth of flow in a gutter may be estimated from the nomograph in Figure 4.37.

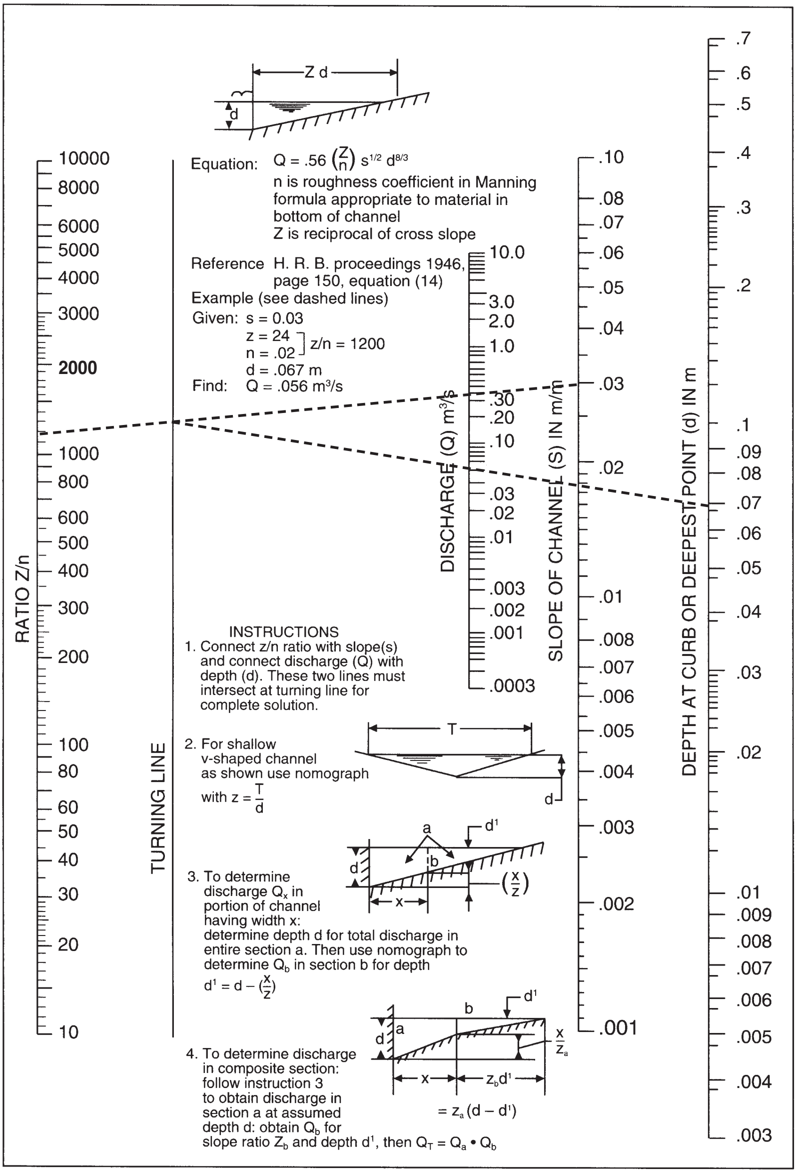


Figure 4.37 Nomograph for flow in triangular channels.

Research work on inlet capacities carried out by various agencies, institutions and municipalities has resulted in the development of empirical equations, hydraulic capacity charts and nomographs to help the designer with storm water inlet selection.

The inlet capacity of an undepressed curb inlet may be expressed by the equation:

$$Q = (4.82 \times 10^{-3})\, d\, l\, \sqrt{gd}$$

where:
Q = discharge into inlet, m^3/s
d = depth of flow in gutter, m
l = length of opening, m
g = gravitational constant, 9.806 m/s^2

If the gutter is of a wedge shape cross section with a street cross-slope of between 0.001 and 0.100 m/m, the inlet capacity of an undepressed curb inlet may be expressed by the equation:

$$Q = 1.29\, i^{0.579}\, l \left\{ \frac{Q_o}{\sqrt{s/n}} \right\}^{0.563}$$

where:
i = cross-slope, m/m
Q_o = flow in the gutter, m^3/s
s = hydraulic gradient of the gutter (street grade), m/m
n = Manning's n of the gutter

Slotted drain inlets are typically located as spaced curb inlets on a grade (sloping roadway) to collect downhill flow, or located in a sag (low point). The necessary length of slot can be determined using Figures 4.37 through 4.39.

For a series of slotted drain curb inlets on a grade, each inlet will collect all or a major portion of the flow to it. The anticipated accumulated flow at points along the curb can be determined by the methods described above.

Once the initial upstream inlet flow is established, Figure 4.38 is used to determine the required length of slot to accommodate the total flow at the inlet.

The length of slot actually used may be less than required by Figure 4.38. Carryover is that portion of the flow that does not form part of the flow captured by the slotted drain. While some of the flow enters the drain, some flows past the drain to the next inlet. The efficiency of a slotted drain, required in order to consider carryover, is shown in Figure 4.39.

If carryover is permitted, the designer assumes an actual slot length, L_A, such that the ratio of the actual slot length to the length of slot required for no carryover (L_A/L_R) is less than 1.0 but greater than 0.4. Standard slot lengths are 3 and 6 m. Economics favor slotted drain inlets designed to allow carryover rather than for total flow interception. If carryover is allowed, there must be a feasible location to which the carryover may flow.

The slotted drain efficiency can also be calculated using the following equation:

$$E = 1 - 0.918 \left\{ 1 - \frac{L_A}{L_R} \right\}^{1.769}$$

where:
E = efficiency, fraction
L_A = actual slot length, m
L_R = slot length required for no carryover, m

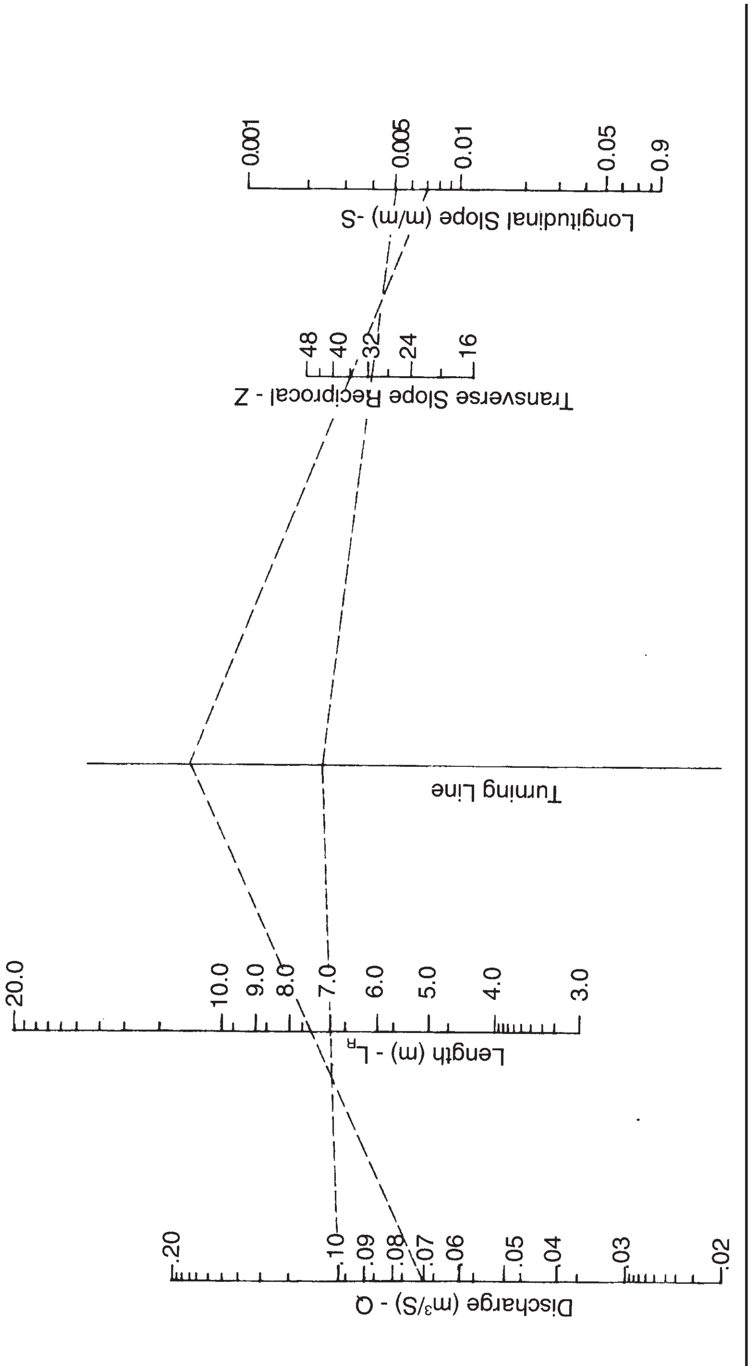
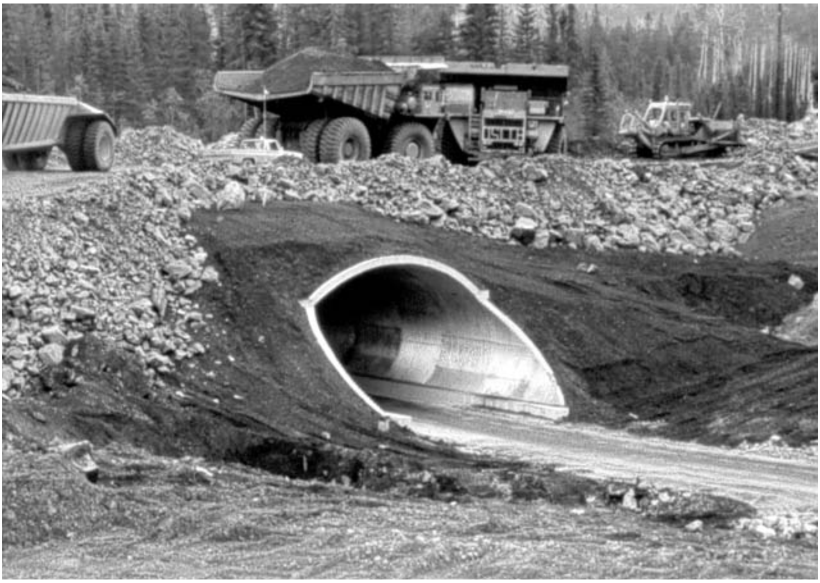
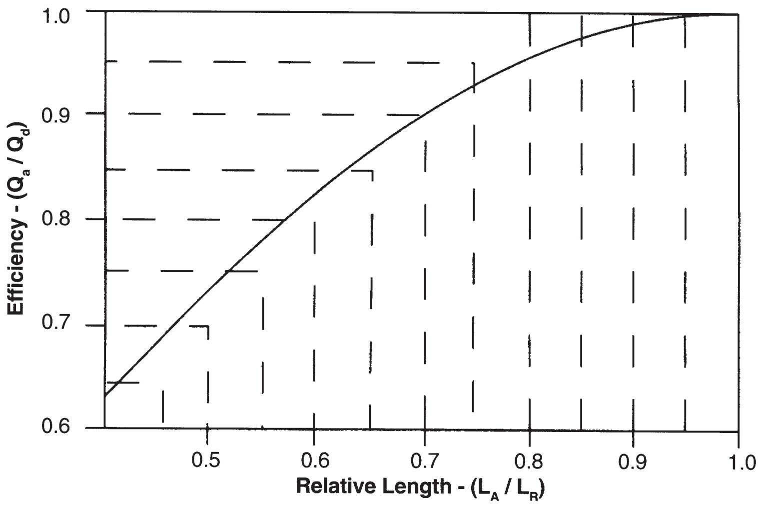


Figure 4.38 Slotted drain design nomograph.



Step-bevel end treatment.

Structural plate CSP with concrete end treatment.



Example: if 20% carryover (Q_a / Q_d = 80%) is allowed, then only 58% (L_A / L_R) of the total slotted drain length is required resulting in a 42% savings in material and installation costs.

Figure 4.39 Slotted drain carryover efficiency.

The amount of carryover can be calculated using the following equation:

$$CO = Q_d\,(1 - E)$$

where: CO = carryover flow, m^3/s
Q_d = total design flow, m^3/s

Combining the above two equations results in the following equation for carryover flow:

$$CO = 0.918\,Q_d \left\{1 - \frac{L_A}{L_R}\right\}^{1.769}$$

When slotted drain is used for sag inlets, the required slot length should be based on the orifice equation, which is:

$$Q_d = C\,A\,\sqrt{2gd}$$

where: C = orifice coefficient = 0.61
A = open area of slot based on the width for which the hydraulic characteristics were measured (0.044 m), $m^2 = L_R\,(0.044)$
g = gravitational constant, 9.806 m/s^2
d = maximum allowable depth of water in the gutter, m

Solving for the required slot length:

$$L_R = \frac{8.413\ Q_d}{\sqrt{d}}$$

For a slotted drain in a sag at the end of a series of drains on a grade, the flow to the drain will include any carryover from the immediately adjacent drain up grade. Unlike a drain-on-grade situation, a slotted drain in a sag will produce significant ponding if its capacity will not accommodate the design flow. Therefore, the actual length of sag inlets should be at least 2 times the calculated required length.

Carryover is not usually permitted at level grade inlets. In that case, the actual slotted drain length must be at least the required length.

Attractive end finishes can be developed.

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