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Steel Pipe Institute

Design of Underground Detention Systems for Stormwater Management

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Chapter One:

Introduction

Stormwater management measures have become a standard practice in urban developments throughout Canada. Few projects are exempt from stormwater management regulations that have been enacted to require the mitigation of the quantitative and qualitative impacts from stormwater runoff.

Traditional methods have relied almost exclusively on surface retention/detention facilities such as stormwater management ponds. Often the site under development does not lend itself to these traditional measures. Some of the problems include: Insufficient space, topography, high land values, and liability issues, to name a few.

Increasingly, engineers are looking toward subsurface methods that retain the above ground use of the site. A popular method is to use underground detention facilities constructed of Corrugated Steel Pipe (CSP). CSP installations make use of large diameter pipe to temporarily store runoff underground until it is released at a predetermined rate. These methods can be combined with other practices to provide water quality improvement.

Corrugated steel pipe products have been used almost exclusively for this application as a result of the considerable cost advantage when compared to other available materials. Other advantages of underground detention systems include:

- Retaining above ground use of the site and the value of that land;
- Ease of installation of CSP systems;
- Flexible configurations permitted with CSP systems;
- Limited liability as compared to other "attractive nuisances"; and
- Overall cost effectiveness.

This manual was developed by the Corrugated Steel Pipe Institute (CSPI) to aid the engineer in the design of underground detention facilities constructed of corrugated steel pipe. The chapters are summarized below.

HYDROLOGY

Chapter 2 presents information on the hydrologic aspects of detention design. An inflow hydrograph is a required element for design of stormwater management systems. While any appropriate hydrologic method can be used, this chapter focuses on the more common applications. Chapter 2 also presents procedures for estimating the required storage volume for corrugated steel pipe detention systems.

HYDRAULIC DESIGN

Chapter 3 contains detailed information on the more challenging design aspect, the hydraulic design. This includes the three major steps after the hydrology has been performed :

1. Sizing the structure and development of the stage-storage relationship;
2. Designing the release structure and development of the stage-discharge relationship;
3. Routing the hydrograph through the structure to check the design and obtain the outflow hydrograph.

These steps rely on the same engineering principles that are used to design traditional pond systems. The only difference is that the components are placed entirely underground.

WATER QUALITY

While the previous two chapters describe the quantitative aspects of design, water quality has become an important issue also. Underground facilities constructed of corrugated steel pipe can provide water quality benefits, the topic of chapter 4. The most common application for water quality is to use these systems in conjunction with infiltration practices. The runoff is stored in large diameter perforated pipe with a granular backfill material and is recharged back to the water table. Infiltration practices with corrugated steel pipe have proven effective over the years provided they are properly designed and constructed.

A new and promising water quality application for underground detention is to combine the storage benefits of corrugated steel pipe with sand filtration techniques for water quality improvement. The sand filter concept is being used in some parts of North America in conjunction with underground detention. With these systems, runoff is detained in the detention structure and slowly filtered through a sand media before being discharged from the site.

STRUCTURAL DESIGN

The structural design of corrugated steel pipe is the same for underground detention as it is for culverts and storm sewers. This topic is thoroughly discussed in chapter 5 and includes specific structural design methodology.

CONSTRUCTION & INSTALLATION

Construction aspects are covered in chapter 6. As with all pipe materials, proper bedding preparation, installation, and backfill are critical.

DURABILITY OF DETENTION SYSTEMS

Corrugated steel pipe products have been in service in North America for over 100 years for culvert and storm sewer applications. Underground detention systems have an even greater advantage in that there is an absence of abrasion in a storage facility. Chapter 7 discusses durability and provides methods for predicting service life when necessary.

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Chapter Two: Hydrology

SITE DISCHARGE ESTIMATION

The first requirement in the design of underground detention systems is the development of hydrographs that describe the pre- and post- developed site conditions. A hydrograph is a plot of discharge over time as shown in Figure 2.1.

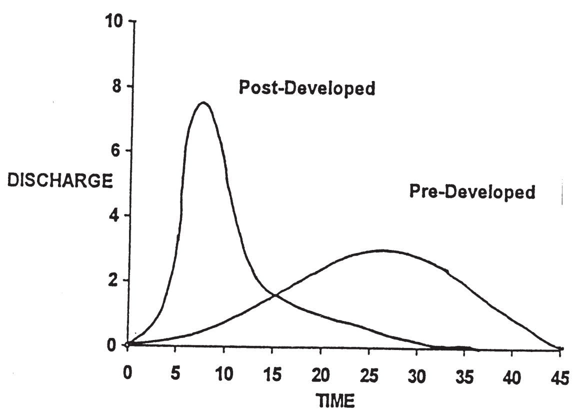


Figure 2.1: Hydrograph

The hydrograph for the post-developed conditions is referred to as the inflow hydrograph. Once the inflow hydrograph is known and compared to the allowable release rate from the detention system, the required storage volume can be estimated.

Numerous methods for developing the hydrograph are available. The method chosen is based on site conditions and local practices. Methods include:

- Rational Method
- Modified Rational Method
- Unit Hydrograph Method
- Soil Conservation Service Method

RATIONAL METHOD

This empirical method was introduced in 1889, and is still the most widely used technique in North America for calculating design storm runoff. The method can be used to calculate peak discharge for up to 80 hectares. The Modified Rational Method, of which an explanation follows, can be used to develop a hydrograph for up to 8 hectares.

The Rational Method is based on the formula:

$$Q = C\, i\, A * 2.78(10^{-3})$$

Where:
Q = maximum rate of runoff (m^3/s)
C = runoff coefficient
i = average intensity of rainfall (mm/hr)
A = contributing area (hectares)

The equation relates the quantity of runoff from a given area to the total rainfall falling at a uniform rate on the same area. Basic assumptions are made when the Rational Method is applied.

1. The peak discharge is greatest when the rainfall intensity lasts as long or longer than the time of concentration. A section detailing the computation of the time of concentration follows.
2. The probability associated with the peak discharge is the same as the probability of the average rainfall intensity used.
3. The rainfall intensity is constant throughout the duration of the storm and the area of the watershed.

Although the Rational Method is adequate for many designs, it has its limitations. The greatest drawback is that it normally provides one point on the runoff hydrograph. When a drainage basin is complex or sub-basins come together, the Rational Method will tend to overestimate actual flows, which may result in over sizing the drainage system.

Runoff Coefficient

The runoff coefficient (C) is the ratio of runoff to the average rate of rainfall. This variable is normally the most subjective variable to determine in the Rational Method formula.

"C" can vary from zero to unity depending on:

- percentage of impervious surface
- characteristics of soil
- duration of rainfall
- shape of drainage area

Table 2.1 shows recommended ranges of runoff coefficients for different types of surfaces.

Table 2.1: Recommended Runoff Coefficients

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	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
Character of Surface	**Runoff Coefficients**
Pavement	
Asphalt	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. A weighted "C" value may be attained.

$$\textbf{Weighted C} = \frac{C_1A_1 + C_2A_2 + C_3A_3...}{A_1 + A_2 + A_3...}$$

Where:
C_x = C value for area x
A_x = area x

Adjustment for Infrequent Storms

The runoff coefficients in Table 2.1 are applicable for storms of 5 to 10 year frequencies. Less frequent higher intensity storms require modification of the coefficients because infiltration and other losses have a proportionately smaller effect on runoff.

The adjustment can be made by multiplying the right side of the Rational Method formula by a frequency factor, C_f, which is used to account for antecedent precipitation conditions. The product of C_f and C should not exceed 1.0.

Thereby the formula becomes:

$$Q = C\ i\ A\ C_f \ * \ 2.78(10^{-3})$$

Where:
C_f = 1.1 for a recurrence interval of 25 years
= 1.2 for 50 years
= 1.25 for 100 years

Intensity

Rainfall intensity (i) is the average rainfall rate (mm/hr) that would be expected to occur during a storm of a certain duration. Storm frequency is the time (years) in which a storm, on average, of a certain intensity may reoccur. Design storm frequency selection for residential, commercial and industrial areas varies from two to one hundred years with two to ten years being the most common for detention design. Duration of rainfall is taken as the time of concentration (see "Time of Concentration") for peak discharge calculations. This is not the case when using the Modified Rational Method where the intensity is determined by the storm duration.

IDF Curves

Rainfall intensity is required in order to use the Rational Method and is usually taken from a rainfall Intensity-Duration-Frequency (IDF) Curve similar to that shown below in Figure 2.2. IDF curves are derived from the statistical analysis of rainfall records compiled over a number of years. These curves are available from various governmental agencies.

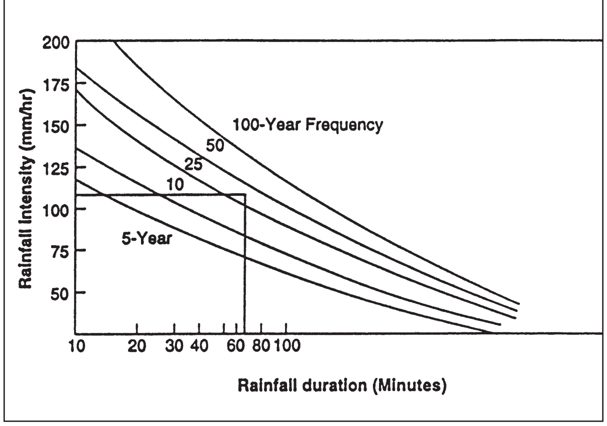


Figure 2.2: IDF Curves

IDF relationships may also be expressed as equations fitted to rainfall data and expressed in the form:

$$i = a / (t + c)^b$$

Where:
i = intensity (mm/hr)
t = time (minutes)
a,b,c = constants unique to each IDF curve

MODIFIED RATIONAL METHOD

The Modified Rational Method is a simplified technique that is applicable to small watersheds (less than 8 hectares) and preliminary design computations. This method provides a triangular or trapezoidal shaped hydrograph for a given storm duration. The time to peak and time to recede is equal to the time of concentration. The peak discharge is based on the Rational Method using the duration of the storm to determine the rainfall intensity. Figure 2.3 shows a typical hydrograph obtained using this method.

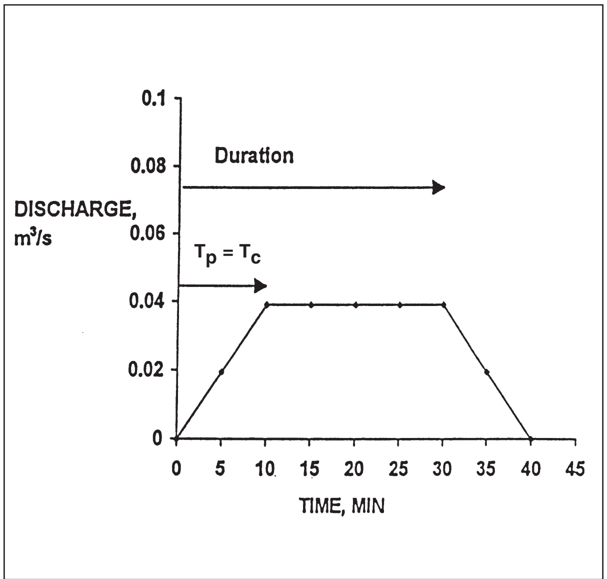


Figure 2.3: Modified Rational Method Hydrograph

UNIT HYDROGRAPH METHOD

The "unit hydrograph" method can be used in cases where a drainage area is quite large, or when a more refined solution of storm runoff is required.

A unit hydrograph can be defined as the hydrograph of one unit of direct runoff from the tributary area resulting from a unit storm. A unit storm is rainfall of such duration that the period of surface runoff is not noticeably less for any rain of shorter duration. Thereby the unit hydrograph represents the integrated effects of factors such as area, shape, street and land slopes, etc.

A basic premise of the unit hydrograph method is that individual hydrographs of successive increments of rainfall excess that occur during a storm period will be proportional in discharge throughout their length. When properly arranged with respect to time, the ordinates of individual unit graphs can be added to give ordinates representing the total storm drainage. A hydrograph of total storm discharge is obtained by summing the ordinates of the individual hydrographs.

Further detail on the hydrograph procedure can be found in many texts, including "Modern Sewer Design" which is available from the Corrugated Steel Pipe Institute.

THE SOIL CONSERVATION SERVICE (SCS) METHOD

The SCS method developed a relationship between rainfall (P), retention (S) and effective rainfall or runoff (Q). The retention, or potential storage in the soil, is established by selecting a curve number (CN). The curve number is a function of soils 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 and consisting chiefly of deep, well to excessively drained sands or gravel.

B. Soils having a moderate infiltration rate when thoroughly wetted and 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 and 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 and 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 2.2 lists typical CN values.

Three levels of Antecedent Moisture Conditions are considered in the SCS method. It 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 I — Soils are dry but not to the wilting point. This is the lowest runoff potential.

AMC II — The average case.

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 2.2 are based on antecedent condition II. Thus, if moisture conditions I or III are chosen, then a corresponding CN value must be determined using Table 2.3.

Table 2.2: Runoff Curve Numbers

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 of 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, U.S. Soil Conservation Service, 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.

Table 2.3 Curve Number Relationships for Different AMC's

CN for Condition II	CN for Condition I	CN for Condition III	CN for Condition II	CN for Condition I	CN for 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	36	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:

$$Q = (P - I_a)^2 / (P + S - I_a)$$

Where:
S = [(100 / CN) - 10] * 25.4

The original SCS method assumed the value of I_a to be equal to 0.2 S. However, many engineers have found that this may be overly conservative, especially for moderated 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 to the above equation.

TIME OF CONCENTRATION

The time of concentration is the time it takes for runoff to flow from the most hydraulically remote point to the detention facility. This is used to determine the rainfall intensity for the Rational Method, but not for the Modified Rational Method. Since a basic assumption of the method is that all parts of the drainage area are contributing runoff, the time of concentration is taken as the storm duration. When using the Modified Rational Method, the time of concentration is used as the time to peak of the hydrograph.

Time of concentration can be estimated by calculating the various overland distances and flow velocities between the most remote points and collection points.

A common error is to view the runoff from only a part of the drainage basin. This error is most often encountered in a basin where the upper portion contains grassy parkland and the lower contains developed urban land.

Remote areas often have flow that is very shallow and velocities cannot be calculated by channel equations. Special overland flow analysis, however, must be completed. In this case, the time of concentration 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.

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 greatest 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 long travel times and vice versa. Thus, 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). It seems reasonable to assume that 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 or surface moisture excess.

METHODS FOR ESTIMATING TIME OF CONCENTRATION

Methods to compute the time of concentration include:

- Kirpich Method
- Uplands Method
- Kinematic Wave Method
- SCS Method

The Kirpich Method

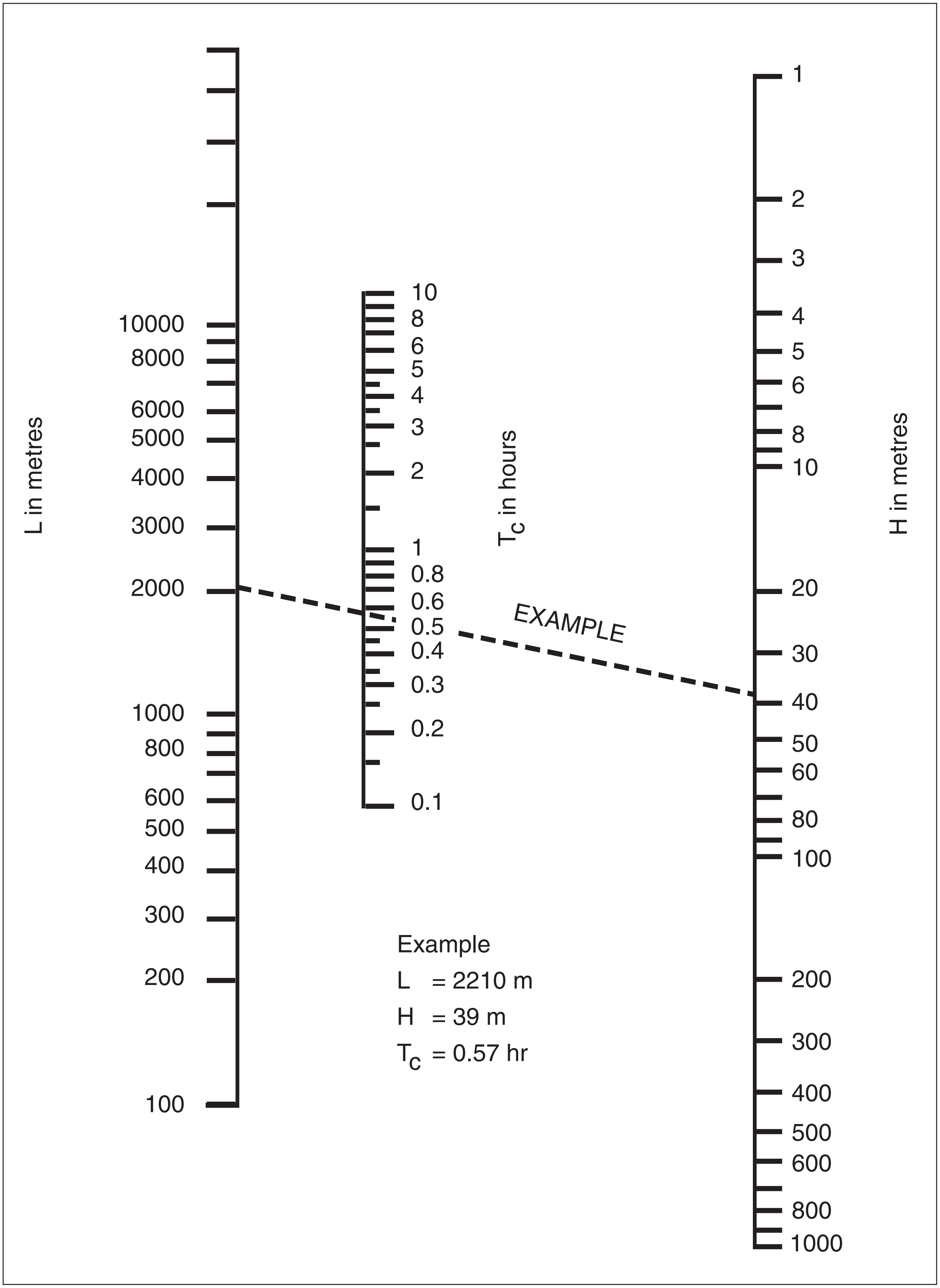
This empirical formula relates T_c to the length and average slope of the basin by the equation:

$$T_c = 0.00032\ L^{0.77}\ S^{-0.385}$$

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

A nomograph solution of the Kirpich equation is shown in Figure 2.4.

Figure 2.4: Kirpich Equation Nomograph



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 on concrete or asphalt surfaces the value should be reduced by multiplying by 0.4. For concrete channels, a multiplying factor 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.

The Uplands Method

The Uplands Method may be used for calculating travel times for overland flow in watersheds with a variety of land covers. This method relates the time of concentration to the basin slope, length and type of ground cover.

Values of $V/S^{0.5}$, for various types of land cover, are given in the following Table 2.4. The velocity is derived using these values and a known basin slope. The time of concentration is obtained by dividing the basin length by the velocity. The times for individual areas are calculated with their summation giving the total travel time.

The Kinematic Wave Method

A method commonly used for overland flow, which 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:

$$\mathbf{T_c = k\ (L\ n\ /\ S)^{0.6}\ i_{eff}^{-0.4}}$$

Where:
k = 0.126
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.

STORAGE VOLUME ESTIMATION

Storm water detention is intended to reduce the peak discharge from a drainage area.

Estimating the required volume of storage to accomplish the necessary peak reduction is an important task since an accurate first estimate will reduce the number of trials

Table 2.4: Uplands Method Values

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 arid mountain regions	3.0
Grassed waterway	4.6
Paved areas (sheet flow); small upland gullies	6.1

involved in the routing procedure. The following discussion presents methods for determining an initial estimate of the storage required to provide a specific reduction in peak discharge. All of the methods presented provide preliminary estimates only. It is recommended that the designer apply several of the methods and a degree of judgement to determine the initial storage estimate.

HYDROGRAPH METHOD

The general procedure for estimating the required storage volume of the detention system is to superimpose the allowable release rate, or outflow hydrograph, over the inflow hydrograph. The area between the hydrograph and release rate represents the storage volume required in the detention system. Graphically, this is shown in Figure 2.5. To determine the necessary storage, the shaded area can be planimetered or computed mathematically.

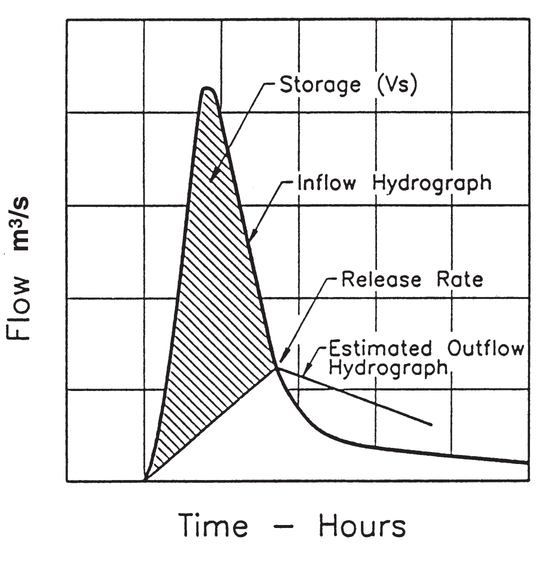


Figure 2.5: Hydrograph for Storage Estimation

TRIANGULAR HYDROGRAPH METHOD

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with standard triangular shapes. This method should not be applied if the hydrographs can not be approximated by a triangular shape. The procedure is illustrated by Figure 2.6.

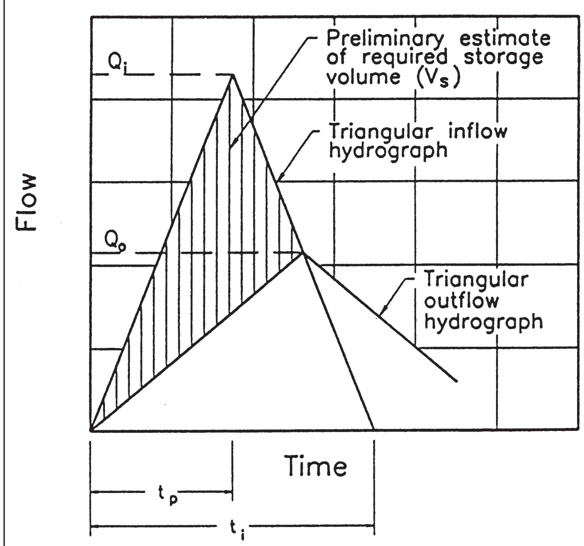


Figure 2.6: Triangular Hydrograph Method

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph with the following equation:

$$V_s = 0.5\ t_i\ (Q_i - Q_o)$$

Where:

V_s = storage volume estimate (m^3)
Q_i = peak inflow rate into the basin (m^3/s)
Q_o = peak outflow rate out of the basin (m^3/s)
t_i = duration of basin inflow (s)
t_p = time to peak of the inflow hydrograph (s)

The duration of basin inflow should be derived from the estimated inflow hydrograph. The triangular hydrograph procedure compares favorably with more complete design procedures involving reservoir routing.

MODIFIED RATIONAL METHOD

The Modified Rational Method can also be used to estimate the required storage volume of the facility. Assuming the allowable release rate is equal to the pre-developed peak discharge, the area between the pre- and post-developed hydrographs approximates the required storage volume as shown in Figure 2.7. This method tends to slightly underestimate the storage volume. It is suggested that this volume be increased by 10 to 20 percent.

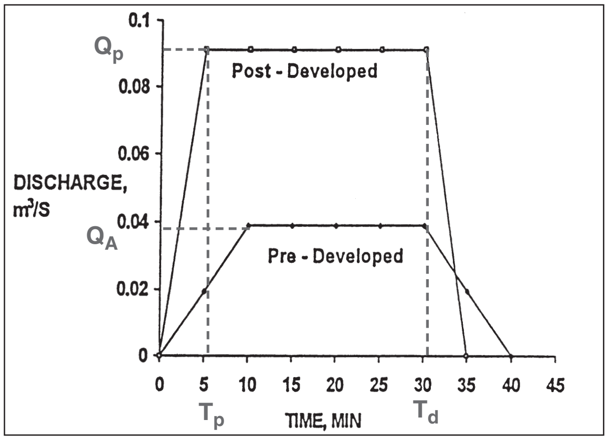


Figure 2.7: Modified Rational Method

The storage volume can be computed as:

$$V_S = Q_pT_d - Q_AT_d - Q_AT_p + (\alpha\, Q_AT_p) / 2 + (Q_A{}^2T_p) / 2Q_p$$

Where:

V_S = volume of storage needed (m^3)
T_d = duration of precipitation (s)
Q_p = peak discharge after development (m^3/s)
Q_A = peak discharge before development (m^3/s)
T_p = time to peak after development (s)
α = ratio of time to peak before development/time to peak after development

SCS STORAGE ESTIMATION

The Soil Conservation Service (SCS), in its TR-55 Second Edition Report, describes a manual method for estimating required storage volumes based on peak inflow and outflow rates. The method is based on average storage and routing effects observed for a large number of structures. A dimensionless figure relating the ratio of basin storage volume (V_s) to the inflow runoff volume (V_r) with the ratio of peak outflow (Q_o) to peak inflow (Q_i) was developed as illustrated in Figure 2.8 for rainfall types II and III and rainfall types I and IA, depending on geographical location. This procedure for estimating storage volume may have errors up to 25% and, therefore, should only be used for preliminary estimates.

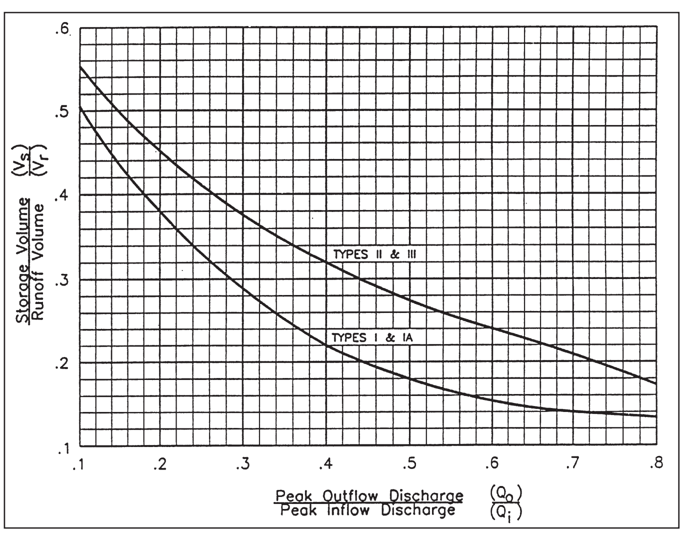


Figure 2.8: SCS Storage Estimation Curves

The procedure for using Figure 2.8, in estimating the detention storage required, is described as follows:

1. Determine the inflow and outflow discharges Q_i and Q_o
2. Compute the ratio Q_o/Q_i
3. Compute the inflow runoff volume V_r, for the design storm

   $$V_r = K_r\, Q_D\, A_m$$

   Where:
   V_r = inflow volume of runoff (m^2)
   K_r = 1.00
   Q_D = depth of direct runoff (m)
   A_m = area of watershed (m^2)
4. Using Figure 2.8, the ratio Q_o/Q_i and the AMC, determine the ratio V_s/V_r using the appropriate line for local rainfall conditions.
5. Determine the storage volume V_s, as

   $$V_s = V_r\,(V_s/V_r \text{ ratio})$$

REGRESSION EQUATION

An estimate of the storage volume required for a specified peak flow reduction can be obtained by using the following regression equation procedure first presented by Wycoff & Singh in 1986.

1. Determine the volume of runoff in the inflow hydrograph (V_r), the peak inflow rate (Q_i), the allowable peak outflow rate (Q_o), the time base of the inflow hydrograph (t_i), and the time to peak of the inflow hydrograph (t_p).
2. Calculate a preliminary estimate of the ratio V_s/V_r using the input data from step 1 and the following equation:

   $$(V_s/V_r) = \frac{1.291\,(1 - Q_o/Q_i)^{0.753}}{(t_i/t_p)^{0.411}}$$
3. Multiply the inflow hydrograph volume (V_r) times the volume ratio computed from this equation to obtain an estimate of the required storage volume.

Chapter Three: Hydraulic Design

Once the hydrologic analysis is complete and the preliminary estimate of the storage requirement is known, the initial hydraulic analysis can be performed. The required steps are:

1. Select the pipe dimensions and develop the stage-storage relationship.
2. Design the flow regulator and develop the stage-discharge relationship.
3. Route the inflow hydrograph through the structure to check that the proper storage volume is available and that the allowable release rate has not been exceeded. This step produces an outflow hydrograph.
4. Make any necessary adjustments to the system and re-analyze for the final design.

The hydraulic analysis is an iterative process for each storm analyzed. In most cases, at least two storms are analyzed, a design storm as well as a check storm, typically the 100year flood event, to ensure there is no surcharging of the system. It is also not uncommon to have more than one design storm, such as the 2-year and 10-year storm.

STORAGE ANALYSIS

PIPE SELECTION

The pipe lengths and diameters can be selected to provide the required storage volume. While the final layout of the pipe system may be comprised of various lengths and different sizes, for the initial layout, it is best to treat the systems as one length and diameter. After an adequate preliminary hydraulic analysis is complete, a more complex system of pipes can be selected and analyzed.

For underground detention facilities, it is desirable to have as large a pipe diameter as the site will allow. Large diameter pipes provide for a smaller footprint of the system and permit better access for inspection and maintenance.

The pipe diameter is most often based on the constraints of outfall elevation and cover requirements. The minimum pipe diameter is 1200 mm to 1800 mm.

STORAGE IN PIPES

The tables at the end of this chapter (Tables 3.5 to 3.10) provide dimensions for standard pipe sizes and their cross sectional areas for determining storage volumes in pipes. They also provide water area to total cross sectional end area ratios for various water depth to rise ratios for pipe-arch shapes. For additional product details, refer to the "Handbook of Steel Drainage and Highway Construction Products" distributed by the CSPI.

Most detention systems are constructed on flat or nearly flat grades so that slope is not a factor when computing storage volumes. Pipes placed on slopes greater than a few percent are also less efficient as storage devices. If site constraints require large changes in elevation, then it is recommended that vertical pipes be used to connect a series of pipes with slopes less than 2%.

If for some reason steeper slopes cannot be avoided, the equation below describing the ungula of a cone can be used to adjust for changes in storage volume.

$$V = H\,(2/3\,a^3 \pm c\,B) / (r \pm c)$$

Where:
V = volume of ungula (m^3)
B = cross sectional area of flow at the end of pipe (m^2)
H = wetted pipe length (m)
r = pipe radius (m)
d = flow depth (m)
a = $[(2r - d)\,d]^{0.5}$
c = d - r

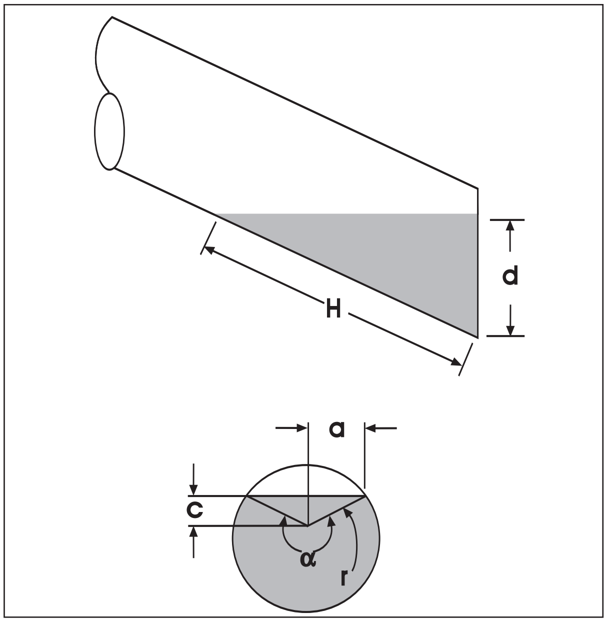


Figure 3.1: Ungula of a Cone

STAGE-STORAGE RELATIONSHIP

The stage-storage table or curve represents the total cumulative storage provided in the detention system with the water at specified elevations. When the system has been sized, this relationship can be computed for the routing procedure.

DESIGN OF FLOW REGULATORS

The flow regulator, or release structure, is the component that controls the flow out of the structure. It may consist of an orifice or weir, or a combination of the two. The flow regulator is sized based on a predetermined allowable release rate and when analyzed, will produce the stage-discharge relationship.

TYPES OF FLOW REGULATORS

Flow regulators can be classified as either an orifice or a weir. Weirs can be sharp- or broad-crested; orifices could include short and long tubes. The release structure may also be a perforated riser pipe or a small diameter pipe. The depth of water (head) acting on the release structure will determine the flow out of the system and may also affect how that structure will function.

The release structures should include both a primary, or principal, flow regulator and an emergency overflow device. The flow regulator may be a single or multiple opening orifice, while emergency overflow capacity may be provided by a weir.

Vertical perforated pipes can also be used. They usually consist of uniform sized holes spaced uniformly around the perimeter of the pipe with several rows of holes along the length of the pipe. Perforated pipes are analyzed hydraulically by treating the holes as orifices. The discharges through each hole in each row below water are added together to obtain the total discharge for each depth of water. If the depth of water is above the top of the vertical pipe, the flow through the top of the pipe is analyzed either as a weir or an orifice, depending on the head above the pipe.

Vertical Orifice

The simplest flow regulating structure is an orifice installed in the side of a bulkhead in the detention system, as shown in Figure 3.2. When the orifice is small in comparison to the depth of water, the discharge through the orifice can be calculated using the basic equation for an orifice:

$$Q = C_d A (2gh)^{1/2}$$

Where:
Q = peak discharge rate (m^3/s)
C_d = coefficient of discharge
A = cross sectional area of orifice (m^2)
g = acceleration due to gravity (9.81 m/s^2)
h = head on the center of the orifice (m)

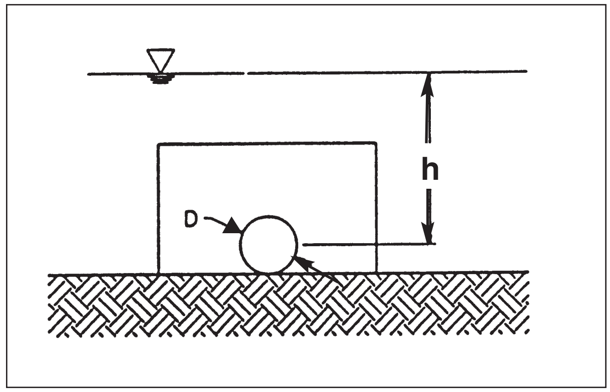


Figure 3.2: Vertical Orifice

This equation assumes that there is no back pressure from downstream (i.e., the outlet is not submerged). If the outlet is submerged, this equation can still be used. If that is the case, use the difference in the water surface elevations on either side of the orifice, as shown in Figure 3.3, as the depth h in the equation.

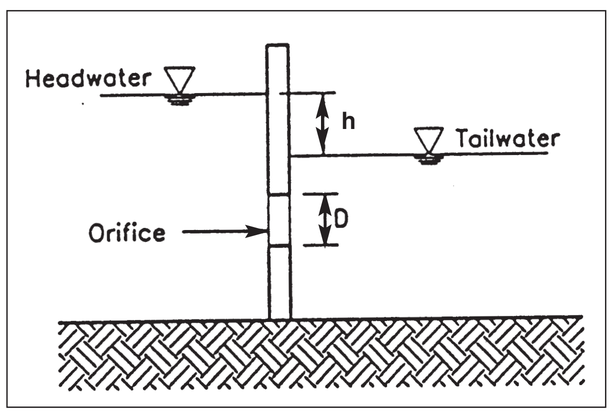


Figure 3.3: Submerged Vertical Orifice

Flow through multiple orifices can be accurately modeled by analyzing each individual orifice separately and then summing the combined flows, as shown in Figure 3.4.

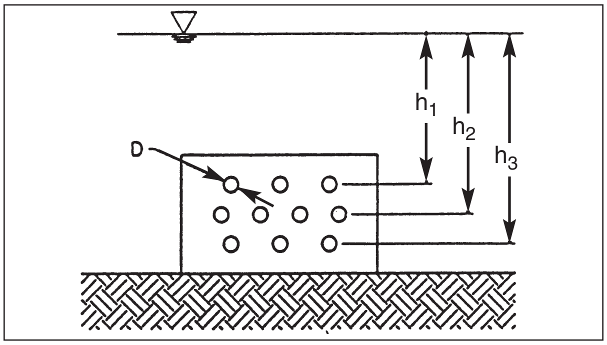


Figure 3.4: Multiple Orifices

The coefficient of discharge C_d, can vary significantly with the shape and the type of the orifice. As a result, it is important to pay careful attention to the details of the orifice shape during construction to ensure that the completed facility will operate as designed.

The coefficient for a sharp-edged orifice with complete contraction varies from 0.59 to 0.66. A nominal value of 0.60 can be used for the types of orifices and range of heads normally used for outlet structures. For orifices with partially suppressed contractions, the coefficient ranges from 0.62 to 0.71. For orifices with fully rounded edges, (i.e., the contraction of the jet is fully suppressed) the coefficient ranges from 0.94 to 0.95. If holes are drilled through a steel or concrete pipe and the surface and edges of the holes are left rough, the coefficient could be 0.4. Additional information can be found in "King's Handbook of Hydraulics".

The opening of the outlet can be round, square, rectangular, or any other convenient shape. It can be a gate or a plate attached to a headwall located in front of the outlet pipe. The orifice in the latter case is cut into a plate which is then installed so that it can be easily removed for maintenance or replacement.

Outlets with openings of less than 100 to 150 mm are susceptible to clogging. The chances of clogging can be significantly reduced by installing trash racks having a net opening that is more than 20 times the opening of the outlet. Larger outlets also need trash racks, but the ratio of the net opening to the outlet opening does not need to be as large.

When the size of the orifice is relatively large when compared to the water depth, orifice equations are not accurate. When water depth above the invert of the orifice is less than two to three times the height of the orifice, more accurate results are possible through the use of inlet control nomographs for culverts which are published by the Federal Highway Administration. They are reproduced in the "Handbook of Steel Drainage and Highway Construction Products" which is available from the Corrugated Steel Pipe Institute. A nomograph for calculating the discharge of circular culvert operating under inlet control conditions is shown on the following page as Figure 3.5.

Horizontal Orifice

Although it is not common, the flow regulating orifice can be installed in the bottom of the storage system similar to a bathtub drain. When the depth, h, is relatively large when compared to the orifice opening, its flow capacity can be calculated using the formula:

$$\mathbf{Q = C_d\, A\, [2g\, (h - (d/2))]^{1/2}}$$

Where:
Q = discharge rate through the outlet (m^3/s)
C_d = discharge coefficient
A = area of the orifice or nozzle (m^2)
g = acceleration due to gravity (9.81 m/s^2)
h = water depth above outlet (m)
d = diameter of the orifice (m)

The discharge coefficients for horizontal orifice are the same as for a vertical orifice. At smaller depths (less than three times the diameter of the orifice) severe vortex action develops which is not accounted for in the equation.

Flow Restricting Pipe

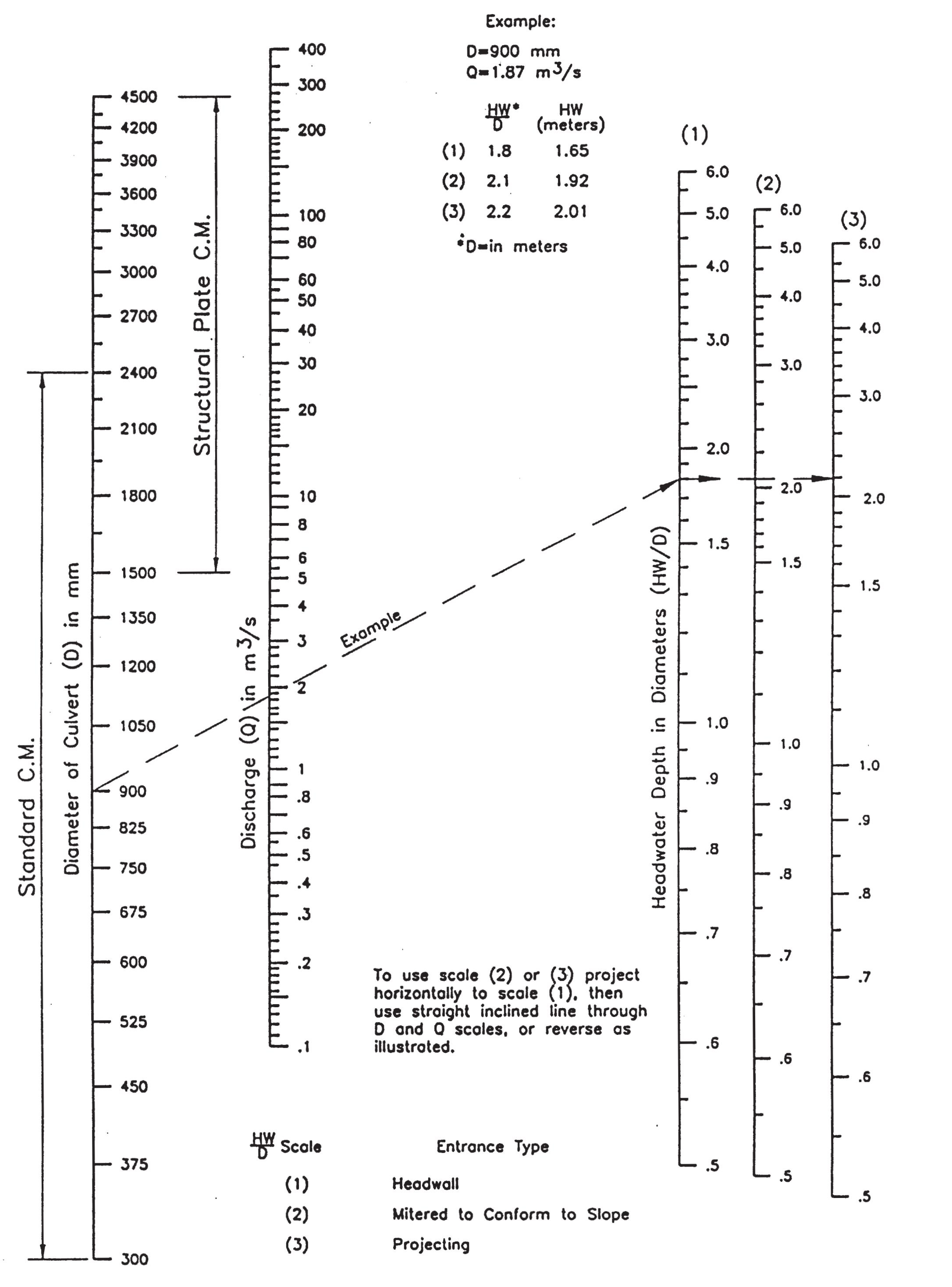
In some cases, the detention system may discharge into a flow restricting pipe. The net flow restricting effect of the pipe is mostly a function of the pipe length and pipe roughness characteristics. This is similar to a culvert flowing under outlet control conditions.

A pipe outlet may also be used to provide greater flow reduction while using a larger diameter outlet. If the pipe is set at a slope that is less than the hydraulic friction slope, outlet capacity can be reduced without the use of a small diameter orifice.

If it is assumed that the pipe is flowing full and the discharge end of the pipe is not submerged, the outlet capacity can be calculated as:

$$\mathbf{Q = A\, [2g\, (h + S\, L - m\, D) / (K_L)]^{1/2}}$$

Figure 3.5: Inlet Control Nomograph



Where:
Q = outlet capacity (m^3/s)
A = cross sectional area (m^2)
g = acceleration due to gravity (9.81 m/s^2)
h = depth of water above outlet pipe's invert (m)
S = slope of outlet pipe (m/m)
L = length of outlet pipe (m)
m = ratio of water depth to pipe diameter at the outlet end of the pipe
D = diameter of outlet pipe (m)
K_L = sum of loss factors

Losses

The total energy loss through the pipe may be computed as:

$$K_L = k_t + k_e + k_f + k_b + k_o$$

Where:
k_t = trash rack loss factor
k_e = entrance loss factor
k_f = friction loss factor
k_b = bend loss factor
k_o = outlet loss factor

Trash Rack Loss Factor

The loss factor at a trash rack can be approximated using the following equation:

$$k_t = 1.45 - 0.45(an / ag) - (an / ag)^2$$

Where:
an = net open area between the rack bars
ag = gross area of the rack and supports

One approach is to assume that 50% of the rack area is blocked when estimating the maximum potential losses at the rack. Also, calculate the maximum outlet capacity assuming no blockage. It is recommended minimum and maximum outlet capacities be checked to ensure that the installation will function adequately, under both possible operating scenarios, in the field.

Entrance Loss Factor

An entrance loss factor can be developed by taking the orifice equation, rearranging its terms, and recognizing that the depth term in the equation is actually the sum of the velocity head and the head loss:

$$k_e = 1 / (C_d)^2 - 1$$

Where:
C_d = orifice discharge coefficient

Friction Loss Factor

The pipe friction loss factor for a pipe flowing full is expressed as:

$$k_f = f\ (L / D)$$

Where:
f = Darcy-Weisbach friction loss coefficient which, under simplifying assumptions, can be expressed as a function of Manning's n

$$f = 185\ (n^2 / D^{1/3})$$

Bend Loss Factor

Bend losses in a closed conduit are a function of bend radius, pipe diameter, and the deflection angle at the bend. For 90 degree bends having a radius at least twice the pipe diameter, a value of $k_b = k_{90} = 0.2$ is often used. For bends having other than 90 degree bends, the bend loss factor can be calculated using the following equation:

$$k_b = K\ k_{90}$$

Where:
K = adjustment factor taken from Table 3.1
k_{90} = loss factor for 90 degree bend

Table 3.1: Adjustment Factors for Other Than 90 Degree Bend Losses

Angle of Bend (Degrees)	Adjustment Factor
20	0.37
40	0.63
60	0.82
80	0.90
100	1.05
120	1.13

U.S. Bureau of Reclamation, 1973

Outlet Loss Factor

Virtually no recovery of velocity head occurs where the pipe freely discharges into the atmosphere or is submerged under water. As a result, unless a specially shaped flared outlet is provided, it is safe to assume that $k_o = 1.0$.

Weirs

Weirs can also be used as primary overflow control devices. They can provide the first level of emergency overflow, or actually be a part of the outflow regulating system. Weirs can come in different shapes and sizes, and equations are available to accurately compute the discharge.

The two basic types of weirs are sharp-crested weirs and broad-crested weirs. A sharp-crested weir is one in which the top of the weir is like a knife edge and the jet, or nappe, springs free as it leaves the upstream face. A broad-crested weir, on the other hand, supports the flow in the longitudinal direction. Depending on the physical configuration of a weir, it can be a broad-crested weir at low heads and a sharp-crested weir at higher heads.

The general equation for horizontal-crested weir is:

$$\mathbf{Q = C\ L\ h^{3/2}}$$

Where:
Q = discharge over the weir (m^3/s)
C = discharge coefficient
L = effective length of the weir crest (m)
h = head above the weir crest (m)

Sharp-Crested Weir

The flow over a sharp-crested weir having no end contractions, as shown in Figure 3.6, can be calculated using the above general equation for a horizontal crested weir.

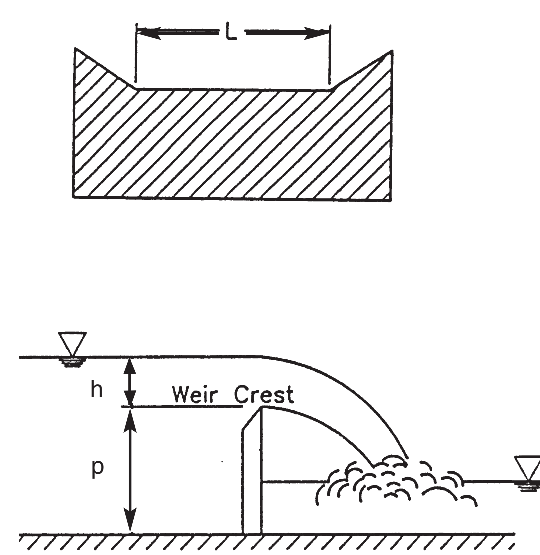


Figure 3.6: Sharp Crested Weir with No End Contractions

The measurement of a representative head h for a sharp-crested weir is made at a distance of approximately 2.5 * h upstream of the weir crest. According to Chow (1959), the discharge coefficient for a sharp-crested weir can be calculated using:

$$\mathbf{C = 1.81 + 0.22\ (h / P)}$$

Where:
P = height of weir crest above the channel bottom (m)

For h/P values up to o.3, a constant C of 1.84 is often used.

This equation gives accurate results if the weir nape is fully aerated and is not submerged (tailwater is below the weir crest). If the nape is not aerated (tailwater is approaching the elevation of the weir crest), a partial vacuum develops under the nape and the flow over the weir increases. The flow also becomes very unstable and undulating.

In many instances, the weir crest does not extend completely across the release structure. This is called a weir with end contractions, as shown in Figure 3.7.

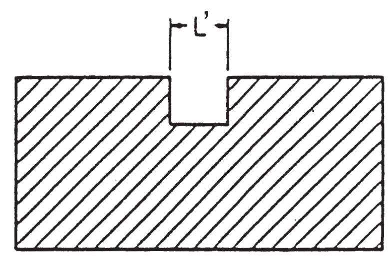


Figure 3.7: Weir with End Contractions

Therefore, the length needs to be corrected for flow contractions at each end of the sharp-crested weir. The effective weir length is calculated using:

$$\mathbf{L = L' - (0.1n\ h)}$$

Where:
L = effective length of the weir crest (m)
L' = measured length of the weir crest (m)
n = number of end contractions
h = head of water above the crest (m)

Triangular Sharp-Crested Weir

A triangular sharp-crested weir should be considered whenever the weir needs to control low flows. As shown in Figure 3.8, the water surface width over this weir varies with depth.

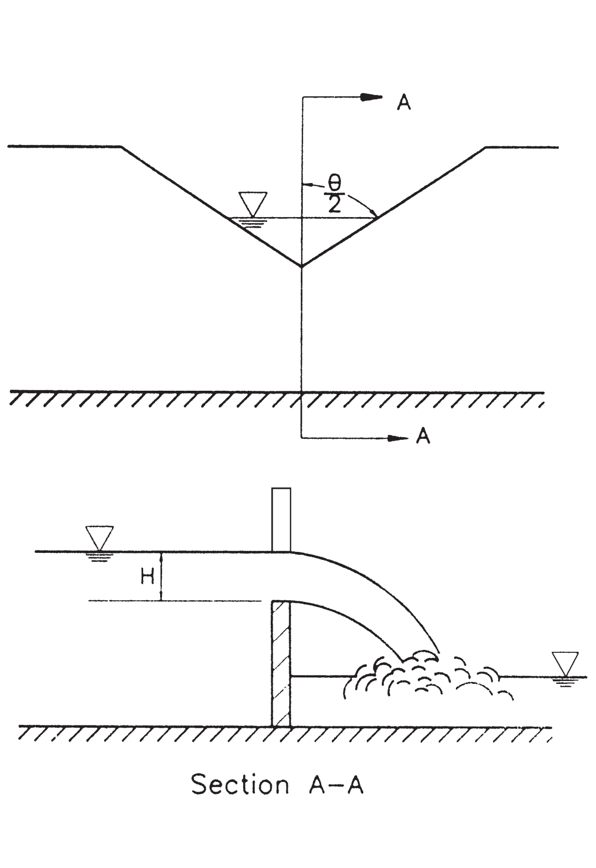


Figure 3.8: Triangular Weir

As a result, weir capacity is sensitive to the water depth at low flows. The discharge over a triangular sharp-crested weir is given by:

$$Q = C_t\, h^{5/2} \tan(\theta / 2)$$

Where:
C_t = discharge coefficient for a triangular weir
θ = weir notch angle in degrees
h = head above weir notch bottom (m)

The head h is measured from the bottom of the notch to the water surface elevation at a distance of 2.5h upstream of the weir. The value of C_t usually used for design is 1.38.

Submergence of a Sharp-Crested Weir

When the tailwater rises to above the weir crest, as shown in Figure 3.9, the discharge calculations for the non-submerged case must be corrected for submergence.

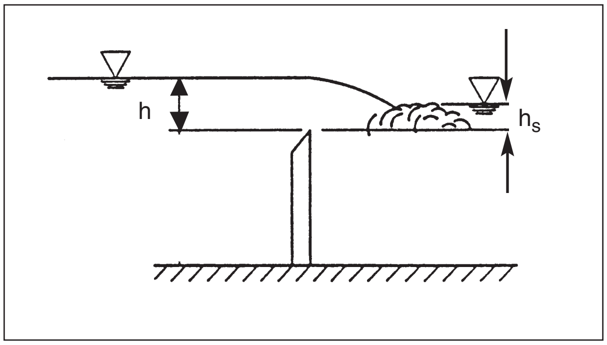


Figure 3.9: Submerged Weir

The following equation may be used to correct for submergence on a rectangular or triangular sharp-crested weir as long as the appropriate exponent is used:

$$Q_s = Q\,[1.0 - (h_s / h)^n]^{0.385}$$

Where:
Q = discharge calculated for a non-submerged weir equation (m^3/s)
Q_s = discharge for a submerged weir (m^3/s)
h = head upstream of the weir (m)
h_s = tailwater depth above the weir crest (m)
n = 1.5 for rectangular, 2.5 for triangular

Broad-Crested Weir

Broad-crested weirs are sometimes used in detention facilities as overflow devices or spillways. The discharge over a broad-crested weir is given by the equation:

$$Q = C\, L\, H_t^{3/2}$$

Where:
Q = discharge (m^3/s)
C = coefficient of discharge
L = effective length of the weir crest (m)
H_t = $h + V^2 / (2g)$
h = head upstream of the weir (m)
V = approach velocity at 3 * h upstream of crest (m/s) (usually taken at V = 0 for detention overflow)
g = acceleration due to gravity (9.81 m/s^2)

Note that, if the velocity is zero, the equation is the same as that for a sharp-crested weir.

The coefficient C for a broad-crested weir has been determined experimentally to range between 1.44 to 1.71. A value of C = 1.705 is often used for the design of detention overflow structures and spillways.

STAGE-DISCHARGE RELATIONSHIP

When the type and dimensions of the release structures have been determined, the discharges are calculated at predetermined water levels in the detention structure. These elevations must correspond to the same elevations used in the stage-storage relationship. For each elevation, the head acting on the orifice, or the depth of water over the weir, is determined and the discharges are computed using the appropriate equations.

If a structure is used that has multiple orifices or weirs, each one is analyzed individually and then a composite stage-discharge relationship is constructed by summing the individual discharges.

ROUTING PROCEDURE

When the stage-storage relationship and stage-discharge relationship have been defined, the inflow hydrograph can be routed through the structure to obtain the outflow hydrograph. The routing is performed using the Storage Indicator Method. The Storage Indicator Method treats the two unknowns, the storage at time two and the outflow at time two, as one quantity as shown below:

$$\mathbf{(I_1 + I_2)/2 + (S_1/t - O_1/2) = S_2/t + O_2/2}$$

Where:
I_1 = inflow at time one
I_2 = inflow at time two
S_1 = storage at time one
S_2 = storage at time two
O_2 = outflow at time two
O_1 = outflow at time one
t = time interval

By constructing a Storage Indicator Table and Storage Routing Table, the Storage Indicator equation can be solved to obtain the outflow hydrograph. The actual storage volume required can then be compared to the volume provided in the pipes. The size and length of the pipe can be adjusted accordingly to optimize the design.

The manual solution to the above equation is provided below. The first step is to construct the Storage Indicator Table and Curve, as shown in Figures 3.10 and 3.11 respectively, as follows:

Step 1: The Discharge, (O_2) column (2), and Storage, (S_2) column (3), are obtained from the Stage-Storage and Stage-Discharge relationships previously developed for each Elevation, column (1).

Step 2: Column (4) is obtained by dividing the Discharge in column (2) by 2.

Step 3: Column (5) is obtained by dividing the Storage in column (3) by the time interval in seconds. This time interval must be the same time interval used in the inflow hydrograph.

Step 4: Column (6) is obtained by adding columns (4) and (5). This number is called the Storage Indicator Number.

Step 5: A Storage-Indicator Curve can be developed by plotting the Discharge, column (2), on the y-axis against the Storage Indicator Number, column (6), on the x-axis. A similar curve can be constructed by plotting the Storage against the Storage Indicator Number. This plot will show the actual storage volume consumed during the length of time covered by the hydrograph.

(1) Elevation	(2) Discharge O_2	(3) Storage S_2	(4) $O_2/2$	(5) S_2/t	(6) $S_2/t + O_2/2$

Figure 3.10: Storage Indicator Table

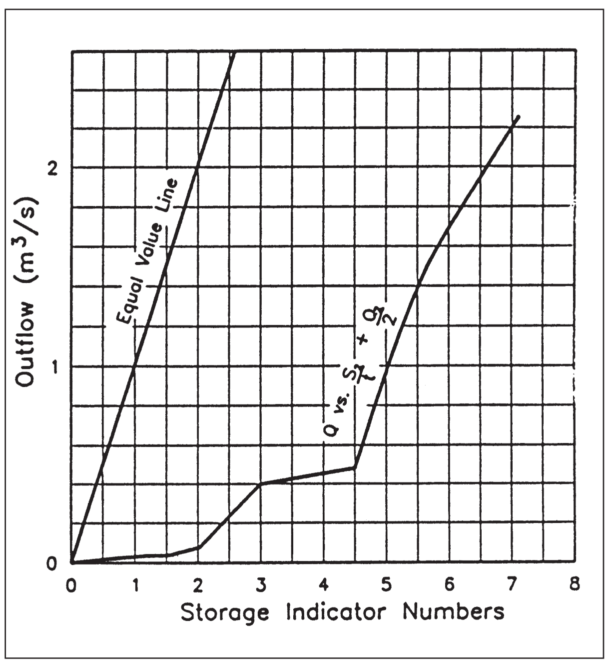


Figure 3.11: Storage Indicator Curve

Next, the Routing Table, as shown in Figure 3.12, is completed as follows:

Step 1: At time zero, the Discharge, (O_2) column (7), and Storage, (S_2), are usually zero, resulting in a value of zero in column (6). The values in columns (6) and (7) are transferred to columns (4) and (5), respectively, on the next line.

Step 2: Column (2) values are obtained directly from the inflow hydrograph for the time step under consideration.

Step 3: Column (3) values are obtained by taking the inflow of the current time step and averaging it with the inflow of the previous time step.

Step 4: Column (6) is obtained by summing columns (3)+(4)-(5), which are in fact the left side of the Storage Indicator Equation. The resultant represents the right side of the equation which is the Storage Indicator Number.

Step 5: Column (7) is obtained from the Storage-Indicator Curve using the Storage Indicator Number in column (6).

Step 6: Columns (6) and (7) are transferred to columns (4) and (5) on the next line. Steps 4 through 6 are repeated until the hydrograph is completely routed.

Step 7: The outflow hydrograph is the plot of time, column (1), against Outflow, column (7).

Step 8: A plot of storage against time can be obtained by referring to the Storage Indicator Table and the Storage Routing Table.

(1) Time	(2) Inflow	(3) $(I_1+I_2)/2$	(4) $S_1/t + O_1/2$	(5) O_1	(6) $S_2/t + O_2/2$	(7) O_2

Figure 3.12: Storage Routing Table

EXAMPLE PROBLEM

A 0.4 hectare parcel of land is to be developed for commercial use. The existing land use is an undisturbed meadow. Design an underground detention chamber to maintain the 10-year post-developed peak flow at pre-developed conditions. A 30-minute duration storm is specified.

C_{PRE} = 0.3 $\quad$ C_{POST} = 0.7
$t_{c,\ PRE}$ = 10 min. $\quad$ $T_p = t_{c,\ POST}$ = 5 min.
T_d = 30 min.
I_{10} = 117 mm/hr for a 30-minute duration

STEP 1: Develop Inflow Hydrograph using Modified Rational Method

$$Q_A = Q_{PRE} = C_{PRE} I_{10} A\,(2.78)(10)^{-3}$$
$$= (0.3)(117)(0.40)(2.78)(10)^{-3}$$
$$= 0.039\ m^3/s$$

$$Q_P = Q_{POST} = C_{POST} I_{10} A\,(2.78)(10)^{-3}$$
$$= (0.7)(117)(0.40)\ X\ 2.78(10)^{-3}$$
$$= 0.091\ m^3/s$$

STEP 2: Estimate Required Storage Volume

Using the Modified Rational Method with a storm duration of 30 minutes, the required storage volume is estimated as:

$$V_S = T_d Q_p - Q_A T_d - Q_A T_p + (\alpha\, Q_A T_p) / 2 + (Q_A^2 T_p)/2Q_p$$

$$V_S = (30)(0.091) - (0.039)(30) - (0.039)(5) + (2)(0.039)(5)/2 + (0.039)^2(5)/(2)/(0.091)$$

$$V_S = (2.73 - 1.17 - 0.195 + 0.195 + 0.042)(60\ s/min)$$

$$V_S = 96.1\ m^3$$

STEP 3: Size Pipe and Compute Stage-Storage Table

Based on the site constraints, which include the invert elevation of an existing storm sewer system outfall and minimum cover requirements, a 1600 mm maximum pipe diameter can be used. Assuming uniform pipe size, a 47.8 metre pipe length is required to meet the estimated required storage volume. Since this method underestimates the storage volume, the length is increased to 55 metres. Using the dimensions of the pipe, the Stage-Storage Table can be obtained by geometric relationships. The resulting stage-storage table is shown below in Table 3.2.

STEP 4: Size Release Structure and Compute Stage-Discharge Table

An orifice will be used to regulate the discharge from the pipe. Since the maximum release rate based on pre-developed conditions is 0.039 m^3/sec, an orifice is sized to release close to this amount at the maximum stage of approximately 1600 mm. The coefficient of discharge is 0.61.

Based on the orifice equation,

$$Q = C_d A (2g h)^{1/2}$$

A 125 mm diameter orifice is selected and the Stage Discharge Table is computed and combined with the Stage-Storage Table as shown below. Note that the discharge for the 100 mm stage is estimated since the water elevation is not yet over the top of the orifice. The resulting stage-discharge table is shown below in Table 3.2.

STEP 5: Develop the Storage-Indicator Table

The storage-indicator table for this example is presented as Table 3.3. A time increment of 5 minutes was chosen for this example.

STEP 6: Perform Routing

The inflow hydrograph is routed through the detention tank using the method set out for the storage routing table.

The values of the resulting outflow hydrograph are compared to the allowable release rate to ensure that it is not exceeded. These values are also used to check that the proper storage volume is available. The storage routing table for this example is shown in Table 3.4.

STEP 7: Design Review

Note that the maximum discharge rate of 0.039 m^3/s does not exceed the pre-developed discharge rate.

This suggests that the tank is sized correctly to accommodate that flow. However, further iterations must be carried out to ensure that the system will accommodate larger flows. For instance, the pipe should likely be increased in size and an overflow weir should be added so that the system does not surcharge under higher inflows. The discharge pipe must also be sized to accommodate the flows.

TABLE 3.2: Example Stage-Storage and Stage-Discharge Table

Stage (mm)	Storage (m^3)	Discharge (m^3/sec)
0	0	0
100	2.878	0.006
200	7.978	0.012
300	14.354	0.016
400	21.619	0.019
500	29.525	0.022
600	37.877	0.024
700	46.515	0.026
800	55.292	0.028
900	64.069	0.030
1000	72.707	0.032
1100	81.060	0.034
1200	88.965	0.035
1300	96.230	0.037
1400	102.606	0.038
1500	107.706	0.040
1600	110.584	0.041

TABLE 3.3: Example Storage Indicator Table

Elevation (mm)	Discharge (m^3/sec)	Storage (m^3)	$O_2/2$ (m^3/sec)	S_2/t (m^3/sec)	$S_2/t + O_2/2$ (m^3/sec)
0	0	0	0	0	0
100	0.006	2.878	0.003	0.010	0.013
200	0.012	7.978	0.006	0.027	0.033
300	0.016	14.354	0.008	0.048	0.056
400	0.019	21.619	0.010	0.072	0.082
500	0.022	29.525	0.011	0.098	0.109
600	0.024	37.877	0.012	0.126	0.138
700	0.026	46.515	0.013	0.155	0.168
800	0.028	55.292	0.014	0.184	0.199
900	0.030	64.069	0.015	0.214	0.229
1000	0.032	72.707	0.016	0.242	0.258
1100	0.034	81.060	0.017	0.270	0.287
1200	0.035	88.965	0.017	0.297	0.314
1300	0.037	96.230	0.018	0.321	0.339
1400	0.038	102.606	0.019	0.342	0.361
1500	0.040	107.706	0.020	0.359	0.379
1600	0.041	110.584	0.020	0.369	0.389

TABLE 3.4: Example Storage Routing Table

Time (min)	Inflow (m^3/sec)	$(I_1+I_2)/2$ (m^3/sec)	$S_1/t + O_1/2$ (m^3/sec)	O_1 (m^3/sec)	$S_2/t + O_2/2$ (m^3/sec)	O_2 (m^3/sec)
0	0				0	0
5	0.091	0.045	0	0	0.045	0.014
10	0.091	0.091	0.045	0.014	0.122	0.023
15	0.091	0.091	0.124	0.023	0.190	0.028
20	0.091	0.091	0.193	0.028	0.253	0.032
25	0.091	0.091	0.257	0.032	0.312	0.035
30	0.091	0.091	0.317	0.035	0.368	0.039
35	0	0.045	0.374	0.039	0.375	0.039
40	0	0	0.380	0.039	0.335	0.037
45	0	0	0.341	0.037	0.299	0.034
50	0	0	0.305	0.034	0.264	0.032
55	0	0	0.271	0.032	0.232	0.031
60	0	0	0.239	0.031	0.201	0.029
65	0	0	0.209	0.029	0.173	0.027
70	0	0	0.181	0.027	0.146	0.025
75	0	0	0.155	0.025	0.121	0.023
80	0	0	0.131	0.023	0.098	0.021
85	0	0	0.109	0.021	0.077	0.019
90	0	0	0.089	0.019	0.059	0.016
95	0	0	0.071	0.016	0.042	0.014
100	0	0	0.055	0.014	0.028	0.011
105	0	0	0.041	0.011	0.017	0.008
110	0	0	0.030	0.008	0.010	0.005
115	0	0	0.021	0.005	0.005	0.002
120	0	0	0.015	0.002	0.002	0.001
125	0	0	0.011	0.001	0.001	0.001

Table 3.5: Round CSP Pipe Sizes (68 mm x 13 mm and 125 mm x 25 mm corrugations)

Diameter (mm)	End Area (m^2)
1200	1.13
1400	1.54
1600	2.01
1800	2.54
2000	3.14
2400	4.52
2700	5.73
3000	7.07
3300	8.55
3600	10.18

Table 3.6: CSP Pipe-Arch Sizes (68 mm x 13 mm corrugation)

Equiv. Diameter (mm)	Span (mm)	Rise (mm)	End Area (m^2)
1200	1390	970	1.06
1400	1630	1120	1.44
1600	1880	1260	1.87
1800	2130	1400	2.36

Dimensions shown are not for specification purposes; subject to manufacturing tolerances

Table 3.7: Round Structural Plate CSP Pipe Sizes (152 mm x 51 mm corrugation)

Diameter (mm)	End Area (m^2)
1500	1.77
1660	2.16
1810	2.58
1970	3.04
2120	3.54
2280	4.07
2430	4.65
2590	5.26
2740	5.91
3050	7.32
3360	8.89
3670	10.61
3990	12.47
4300	14.49
4610	16.66
4920	18.99
5230	21.46
5540	24.08
5850	26.86
6160	29.79
6470	32.87
6780	36.10
7090	39.48
7400	43.01
7710	46.70
8020	50.53

Table 3.8: Structural Plate CSP Pipe-Arch Sizes (152 mm x 51 mm corrugation)

Span (mm)	Rise (mm)	End Area (m^2)
2060	1520	2.49
2240	1630	2.90
2440	1750	3.36
2590	1880	3.87
2690	2080	4.49
3100	1980	4.83
3400	2010	5.28
3730	2290	6.61
3890	2690	8.29
4370	2870	9.76
4720	3070	11.38
5050	3330	13.24
5490	3530	15.10
5890	3710	17.07
6250	3910	19.18

Dimensions are to inside crests and are subject to manufacturing tolerances.

Table 3.9: Water Area to Full End Area Ratios Based on Water Depth to Rise Ratios for CSP Pipe-Arch Sizes (68 mm x 13 mm Corrugation)

y/D	0.00	0.02	0.04	0.06	0.08
0.0	0.000	0.006	0.017	0.032	0.049
0.1	0.068	0.088	0.108	0.130	0.153
0.2	0.176	0.199	0.223	0.247	0.272
0.3	0.297	0.322	0.347	0.372	0.397
0.4	0.423	0.448	0.473	0.498	0.523
0.5	0.548	0.572	0.597	0.621	0.644
0.6	0.668	0.691	0.713	0.736	0.758
0.7	0.779	0.800	0.820	0.839	0.858
0.8	0.877	0.894	0.911	0.927	0.942
0.9	0.955	0.968	0.979	0.989	0.996
1.0	1.000				

Example: A 1400 mm equivalent diameter pipe-arch (1630 mm span x 1120 mm rise, 1.44 m^2 end area) with a water depth of 830 mm would have a depth to rise ratio of 0.74 and a water area to full end area ratio of 0.820. The water area would then be 0.820 x 1.44 or 1.181 m^2.

Table 3.10: Water Area to Full End Area Ratios Based on Water Depth to Rise Ratios for Structural Plate CSP Pipe-Arch Sizes (152 mm x 51 mm Corrugation)

y/D	0.00	0.02	0.04	0.06	0.08
0.0	0.000	0.006	0.018	0.033	0.050
0.1	0.069	0.089	0.110	0.133	0.156
0.2	0.179	0.203	0.227	0.251	0.276
0.3	0.301	0.326	0.352	0.377	0.402
0.4	0.427	0.452	0.478	0.502	0.527
0.5	0.552	0.576	0.600	0.625	0.648
0.6	0.671	0.694	0.717	0.739	0.760
0.7	0.782	0.802	0.822	0.842	0.861
0.8	0.878	0.896	0.912	0.928	0.943
0.9	0.956	0.968	0.980	0.989	0.996
1.0	1.000				

See example under Table 3.9 for guidance on the use of this matrix.

Chapter Four: Water Quality

QUALITY TREATMENT SYSTEMS

Water quality issues associated with urban runoff have gained more attention in recent years. While stormwater management concerns once focused only on quantitative issues such as minimizing increases in peak discharge as a result of development, land owners are now often required to collect and treat urban runoff. This requires the use of Best Management Practices (BMP's) that reduce pollutant loads. Subsurface systems constructed of corrugated steel pipe can incorporate either filtration or infiltration in order to achieve relatively high pollutant removal efficiencies.

Filtration systems that use sand as a filter media have recently been developed in parts of North America. A corrugated steel pipe chamber contains a sand layer 450 to 600 mm thick with a system of underdrains to collect the runoff. The runoff is filtered through the sand before being released from the site.

Infiltration systems are constructed of perforated corrugated steel pipe surrounded by stone backfill. Runoff is temporarily stored in the pipe and voids in the stone before it eventually infiltrates into the surrounding soils. Infiltration practices require that the proper soils are present.

Both systems require relatively sediment free runoff.

CONSTITUENTS IN RUNOFF

Constituents in urban runoff have been monitored since the 1970's when the US Environmental Protection Agency (EPA) began the massive Nationwide Urban Runoff Program (NURP). This study found that pollutant concentration varied by land use and could generally be divided into three categories: commercial, industrial, and residential. Runoff concentrations from parking lots in commercial and industrial areas are shown in Table 4.1 where they are compared to US mean concentrations. Highway runoff has been studied by various institutions, including the US Federal Highway Administration, and resulting concentrations are reported in Table 4.2.

Table 4.1: Parking Lot Runoff Concentrations (Claytor and Schuler, 1996)

Parameter	U.S. Mean Concentrations	Commercial	Industrial
T.S.S. (mg/l)	100	27	228
Cadmium (μg/l)	0.5	8	2
Copper (μg/l)	10	51	34
Lead (μg/l)	18	28	85
Zinc (μg/l)	160	139	224
Oil/Grease (mg/l)	1-2	8.5	15

The most accepted theory is that atmospheric deposition is the primary source of pollutants. Pollutants are deposited from the atmosphere as either dry deposition or wet deposition where they accumulate on impervious surfaces. The majority of the pollutants are washed off of these surfaces during rain events. The initial runoff tends to have the highest pollutant concentrations. This corresponds to the rising limb of the hydrograph and is referred to as a "first flush". An efficient approach to qualitative stormwater management is to capture and treat only the first flush. The remainder of the storm event may be managed separately through quantitative control as discussed in chapters 2 and 3.

Table 4.2: Highway Runoff Concentrations (Claytor and Schuler, 1996)

Parameter	U.S. Mean Concentrations	Residential Street	Commercial Street	Urban Highway
T.S.S. (mg/l)	100	172	468	142
Cadmium (μg/l)	0.5	1.0	6.7	1.0
Copper (μg/l)	10	25	73	54
Lead (μg/l)	18	51	170	410
Zinc (μg/l)	160	173	450	329
Oil/Grease (mg/l)	1-2	2.0	3.7	

WATER QUALITY VOLUME

Most water quality design practices are sized based on the water quality volume (WQV) which is the volume of runoff to be treated. This approach applies to both infiltration and filtration systems. In order to determine the WQV, the volumetric runoff coefficient, R_v, is first computed as:

$$R_v = 0.05 + 0.009(I)$$

Where:
R_v = volumetric runoff coefficient
I = impervious portion of the site, as a percentage of the total area

The water quality volume from the watershed is then calculated by:

$$WQV = P\, R_v\, A\, (10^{-3})$$

Where:
WQV = water quality volume (m^3)
P = rainfall depth (mm)
A = watershed area (m^2)

Selection of the rainfall depth, P, is often made using the 90% rule. That is, selecting a rainfall depth so that 90% of average daily rain events are captured. In humid regions of North America, this corresponds to a rainfall depth between 13 and 25 mm. Rainfall data can be studied for a specific region to determine the appropriate rainfall depth.

SUBSURFACE SYSTEMS

FILTRATION SYSTEMS

Sand filter technology originates from water treatment plants and has been adapted to treat runoff from urban areas. Sand filters are most often used as a “first flush” treatment system for drainage areas less than 2 hectares. Runoff is filtered through a sand bed and collected through a series of underdrains, then discharged to the receiving system. They are extremely efficient at removing pollutants but are susceptible to clogging if excessive sediment is present.

While a variety of sand filters are used in stormwater management, the DC (District of Columbia) Sand Filter has been adapted to corrugated steel pipe as shown in Figure 4.1.

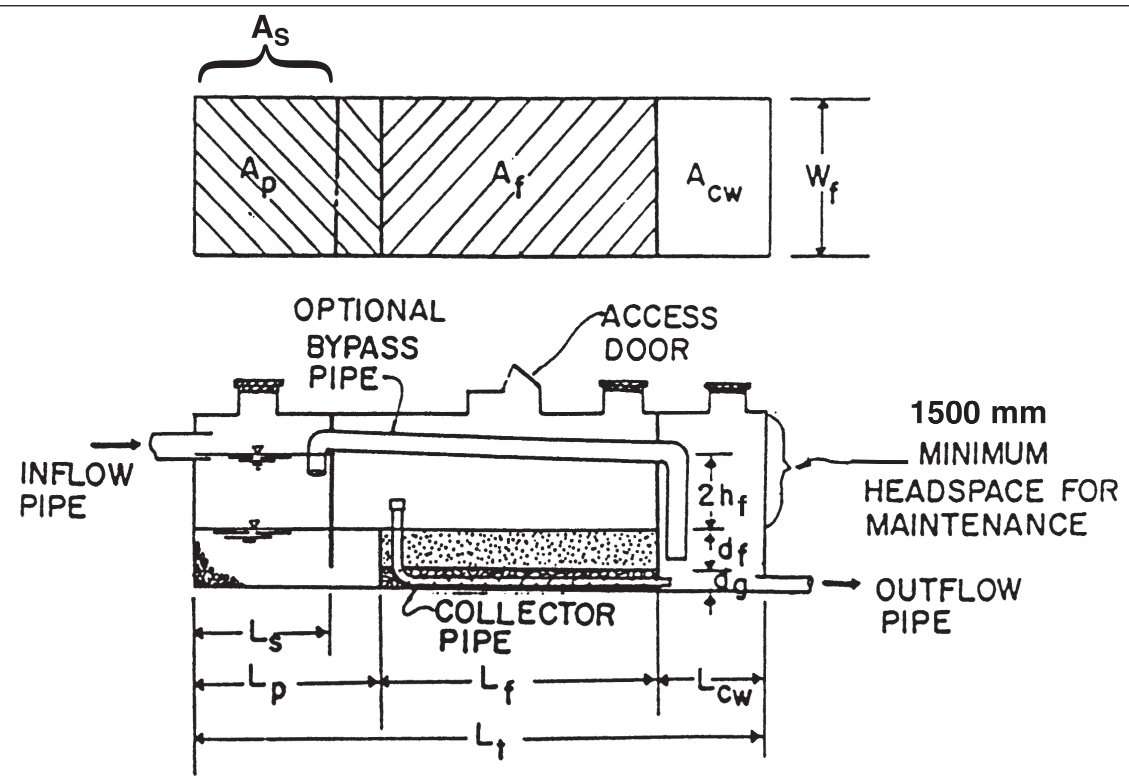


Figure 4.1: DC Sand Filter

Pretreatment for Sand Filters

Pretreatment is required, prior to sand filters, to remove high sediment loads that can lead to clogging of the filter media. While the underground sand filter can contain a sedimentation chamber sized to promote settling, other pretreatment measures can be incorporated into the design. These include:

- Sedimentation Basins
- Vegetative Filter Strips
- Grass Swales
- Catch Basins with Sumps
- Water Quality Structures (Oil/Grit Separators)

Some jurisdictions suggest sizing an in-line sedimentation chamber to provide approximately 20% of the WQV storage. Clayton and Schuler (1996) recommend that the following equation be used to size the sedimentation chamber surface area:

$$\mathbf{A_s = (Q_o / w) \ln(1 - E)}$$

Where:
A_s = sedimentation basin surface area (m^2)
Q_o = rate of outflow from the basin (m^3/s)
= WQV / t_f
t_f = time required for the WQV to filter through the sand bed (s)
= 40 hr = 144,000 s (typically)
w = particle settling velocity (m/s)
= 0.00012 m/s for silt
= 0.001 m/s if site is more than 75% impervious
E = target trap efficiency
= 70% to 90% with 90% recommended

Design of Sand Filters

The following equation is based on Darcy's Law and is used to size the sand filter bed area:

$$\mathbf{A_f = 24\ WQV\ d_f / [k (h_f + d_f) t_f]}$$

Where:
A_f = sand filter bed surface area (m^2)
WQV = water quality volume (m^3)
d_f = sand filter bed depth (m)
= 0.45 to 0.6 m
k = filter media (sand) coefficient of permeability (m/day)
h_f = average height of water above the sand bed (m)
= $h_{max} / 2$
h_{max} = elevation difference between the invert of the inlet pipe and the top of the sand filter bed (m)
t_f = time required for the WQV to filter through the sand bed (hr)
= 40 hr (typically)

k values for sand are available in text books but usually do not account for clogging associated with accumulated sediment. The City of Austin, Texas recommends a value of 1.05 m/day. Monitoring of Austin facilities demonstrated values ranging from 0.15 to 0.81 m/day, with an average of 0.45 m/day.

Design Steps

1. Compute the water quality volume, WQV, as described above.

2. Compute the required size of the sand filter bed surface area, A_f, as described above.

3. Compute the required size of the sedimentation basin surface area, A_s, as described above.

4. Compute the required minimum system storage volume, using:

   $$\mathbf{V_{min} = 0.75\ WQV}$$

5. Choose a pipe size (diameter) based on site parameters such as; storm sewer invert elevation and thus allowable sand filter tank outlet invert elevation, elevation of inlet invert relative to allowable outlet invert elevation, and elevation of finished ground relative to allowable outlet invert elevation. A minimum of 1500 mm should be provided between the top of the filter media and the top of the sand filter tank. Use:

   $$\mathbf{D = 1.5 + d}$$

   Where:
   D = pipe diameter (m)
   d = depth of sand filter and collector pipe media (m)
   = $d_g + d_f$
   d_g = collector pipe media depth (m)
   d_f = sand filter bed depth (m)

6. Compute the filter width, based on the pipe geometry, using:

$$W_f = 2 [R^2 - (R - d)^2]^{0.5}$$

Where:
W_f = filter width (m)
R = pipe radius (m)

7. Compute the filter length using:

$$L_f = A_f / W_f$$

Where:
L_f = filter length (m)

8. Compute the cross sectional end area of the filter bed and collector pipe media using:

$$A_{xf} = 0.5 R^2 \theta_f - (2 d R - d^2)^{0.5} (R - d)$$

Where:
A_{xf} = cross sectional end area of the filter bed and collector pipe media (m^2)
θ_f = 2 ACOS (1 - d / R) (radians)

9. Compute the water volume in the filter bed and collector pipe media using:

$$V_f = n A_{xf} L_f$$

Where:
V_f = volume of water in the filter bed and collector pipe media (m^3)
n = void ratio of the filter media (%)

10. Compute the width of the sedimentation basin at the average depth of water above filter elevation, based on the pipe geometry, using:

$$W_s = 2 [R^2 - (R - d_f)^2]^{0.5}$$

Where:
W_s = width of sedimentation basin (m)
d_f = depth of water (m)
= $d + h_f$

11. Compute the sedimentation basin length using:

$$L_s = A_s / W_s$$

Where:
L_s = sedimentation basin length (m)

12. Compute the cross sectional end area of the water in the sedimentation basin, with the water level at h_{max}, using:

$$A_{xs} = 0.5 R^2 \theta_s - (2 d_h R - d_h^2)^{0.5} (R - d_h)$$

Where:
A_{xs} = cross sectional end area of the water in the sedimentation basin (m^2)
d_h = depth of water (m)
= $d + 2h_f$
θ_s = 2 ACOS (1 - d_h / R) (radians)

13. Compute the minimum length of the sedimentation basin, to contain 20% of the WQV as per the Austin practice, using:

$$L_{smin} = 0.2\ WQV / (A_{xs} - A_{xf})$$

Where:
L_{smin} = minimum sedimentation basin length (m)

14. Compute the temporary water volume in the sedimentation basin using:

$$V_s = (A_{xs} - A_{xf}) L_s$$

Where:
V_s = temporary volume of water in sedimentation basin (m^3)
L_s = maximum of the sedimentation basin length and the minimum sedimentation basin length (m)

15. Compute the water volume above the filter bed using:

$$V_{tf} = L_f (A_{xs} - A_{xf})$$

Where:
V_{tf} = water volume above filter bed (m^3)

16. Compute the additional storage required using:

$$V_{add} = V_{min} - V_f - V_s - V_{tf}$$

Where:
V_{add} = additional water volume storage required (m^3)

17. Compute the minimum length of the permanent pool to accommodate the additional storage required, using:

$$L_{pmin} = L_s + V_{add} / (A_{xs} - A_{xf})$$

Where:
L_{pmin} = minimum length of permanent pool (m)

18. Review the length of the permanent pool and set the final length of the permanent pool. The length of the permanent pool should allow at least 0.6 m between the baffle and the filter bed.

    If $L_{pmin} < L_s + 0.6$, use $L_p = L_s + 0.6$

    If $L_{pmin} \geq L_s + 0.6$, use $L_p = L_{pmin}$

19. Check that the length of the permanent pool makes logistical and economic sense. The design can be refined by changing the pipe diameter, if possible, and repeating the above calculations beginning at step 6.

20. Set the length of the clearwell. The length of the clearwell, L_{CW}, should allow for maintenance and/or access for monitoring the flow rate and chemical composition of the effluent. Usually, 1.0 m is sufficient.

Pollutant Removal

The primary pollutant removal mechanisms for sand filters are mechanical filtration, adsorption, and microbial decomposition on the filter material. Larger sediments are removed in the pretreatment process. Table 4.3 provides removal efficiencies that are based on actual monitoring studies.

Table 4.3: Estimated Pollutant Removal Rates for Sand Filters

Pollutant	Removal Rate %
Sediment	85
Total Phosphorus	40
Total Nitrogen	35
Trace Metals	50 - 90
BOD	70 - 80
Oil and Grease	70 - 85

Inspection and Maintenance

The performance of the sand filter system should be monitored on a quarterly basis during the first year of operation. After that, the systems are typically inspected on an annual basis unless the performance dictates otherwise.

The sedimentation chamber of the sand filter system should be cleaned out when the sediment depth exceeds 300 mm. Trash and debris should be removed on an annual or semi-annual basis. Sediment should be removed from the sand filter bed when accumulation exceeds approximately 15 mm or if the draw down time significantly exceeds the design period.

INFILTRATION SYSTEMS

In areas where natural well-drained soils exist, subsurface disposal of storm water may be used as an effective means of storm water management. The major advantages of using subsurface disposal are;

a. it replenishes groundwater reserves, which is especially significant where municipal water is dependent on groundwater sources and where overdraft of water is causing intrusion of sea water,
b. it is an economical alternative to disposing of storm runoff through the use of pumping stations, extensive outlet piping or drainage channels,
c. it is an effective method of reducing runoff rates, and
d. it is a beneficial way to naturally treat storm water by allowing it to percolate through the soil.

Numerous projects involving subsurface disposal of stormwater have been constructed and have proven to be successful. However, whether runoff is being conveyed overland or discharged to underground facilities, careful consideration should be given to any adverse impact that may result. In subsurface disposal, this may include the adverse impact of percolated water on the quality of the groundwater.

Infiltration Basins

Infiltration basins are depressions of varying size, either natural or excavated, into which storm water is conveyed and then permitted to infiltrate into the underlying soil. Such basins may serve dual functions as both infiltration and storage facilities if storage is provided with corrugated steel pipe. Infiltration basins may be integrated into park lands and open spaces in urban areas. In highway design they may be located in rights-of-way or in open space within freeway interchange loops.

Infiltration Trenches

Infiltration trenches may be;

1. unsupported open cuts with stable side slopes,
2. vertical wall trenches with a concrete cover slab, void of both backfill or drainage conduits, or
3. trenches backfilled with porous aggregate and with perforated pipes as shown in Figure 4.2.

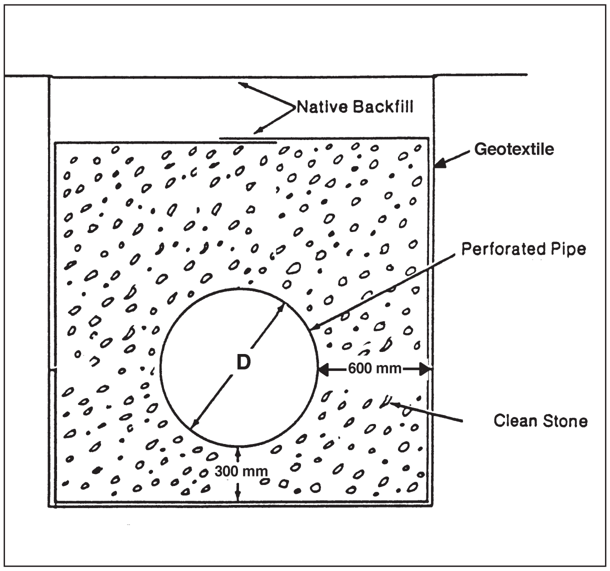


Figure 4.2: CSP Infiltration Trench

The addition of the perforated pipe in the infiltration trench will distribute storm water along the entire trench length, thus providing water quicker and more uniform access to the trench walls. It also allows for the collection of sediment before it can enter the aggregate backfill.

Since trenches may be placed in narrow bands and in complex alignments, they are particularly suited for use in road rights-of-way, parking lots, easements, or any area with limited space. A major concern in the design and the construction of infiltration trenches is the prevention of excessive silt from entering the aggregate backfill, thus clogging the system.

The use of catch basins with deep sumps, sediment traps, filtration manholes, synthetic filter cloths on the outside or inside of the pipe, and the installation of filter bags in catch basins has proven effective.

Design Infiltration Rate

The most critical design aspect of infiltration systems involves the determination of an accurate design infiltration rate. Infiltration rate tests are required in order to provide an estimate of the potential outflow rates for proposed infiltration facilities. The maximum infiltration rates determined through these tests should be used. Minimum rates are provided as a guide in Table 4.4 for given soil classifications. The design infiltration rate should be determined by infiltration testing, soil logs, and other testing and analysis that the geotechnical engineer deems necessary.

Table 4.4: Infiltration Rates for Various Soil Types

Soil Texture	SCS Soil Group	Minimum Infiltration Rate (mm/hr)
Sand	A	210
Loamy Sand	A	61
Sandy Loam	B	26
Loam	B	13
Silt Loam	B	7
Sandy Clay	C	4
Clay Loam	C	2
Silty Clay Loam	D	1
Sandy Clay	D	1
Silty Clay	D	1
Clay	D	0.5

The actual infiltration rate of an infiltration system can be influenced by geometric hydraulic constraints. A shallow water table or impervious layer will reduce the effective infiltration rate of a large facility, but may not be reflected in a small scale test. These effects should be accounted for in the recommended design infiltration rate and in any other design recommendations which may be appropriate. The influence of the site geology on the effective infiltration rate could be analyzed by using groundwater modeling software or other analytical techniques. Selecting a suitable method of accounting for the groundwater and geological conditions at a given site is the responsibility of the designer. The geotechnical engineer should work closely with the design engineer to ensure that appropriate design parameters are used.

Field Tests

Field tests may be carried out using various methods, including auger holes, sample trenches, test pits, or well-pumping tests. The method chosen will depend on the type of facility to be designed and the site parameters. Factors influencing the choice of field tests include the presence of underground utilities, the number of test sites required, requirements for maintenance of the vehicular and/or pedestrian traffic, the type of equipment available to perform the test excavation, and the types of soils involved.

<u>Percolation</u>

Percolation tests are simplified tests used primarily for the design of septic disposal fields. Tests are normally conducted by measuring the drop of water in a given time period in a test hole after a presoaking period. Results are normally reported in millimeters of water elevation drop per minute or hour, or the number of minutes it takes for the water elevation to fall a given distance.

While these tests are sometimes used to determine infiltration design rates for stormwater management, they are limited in that they measure infiltration over a small area and can produce a wide variety of results depending on the specific location of the test hole. In addition, the nature of the test procedures can produce varied results.

Double Ring Infiltrometer

Infiltration can be measured directly using the ASTM D3385 procedure known as the double ring infiltrometer test. This method is more accurate than the percolation test and is more commonly used for infiltration facilities. The procedure uses a given test geometry and head to measure an infiltration rate within a water filled inner ring, while an outer water filled ring is used to minimize boundary effects.

Laboratory Tests

Infiltration rates determined by field tests can be variable and are sensitive to conditions under which a test is conducted, so some engineers and agencies prefer that permeability, or hydraulic conductivity, be used instead. This can be determined by laboratory tests. Two tests commonly used are the constant head test for coarse-grained soil, and the falling head test for fine-grained soils. Other laboratory methods for determining permeability include sieve analysis and hydrometer tests.

Darcy's law may be used to estimate the coefficient of permeability. If a constant head is maintained during the laboratory test, then:

$$K = Q / (A\, i)$$

Where:
Q = the rate of flow (m^3/s)
A = cross sectional area of soil through which flow takes place (m^2)
K = coefficient of permeability (m/s)
i = slope of the hydraulic gradient (m/m)

In the falling head laboratory test, the head drops from the initial test point to the final test point as shown in Figure 4.3.

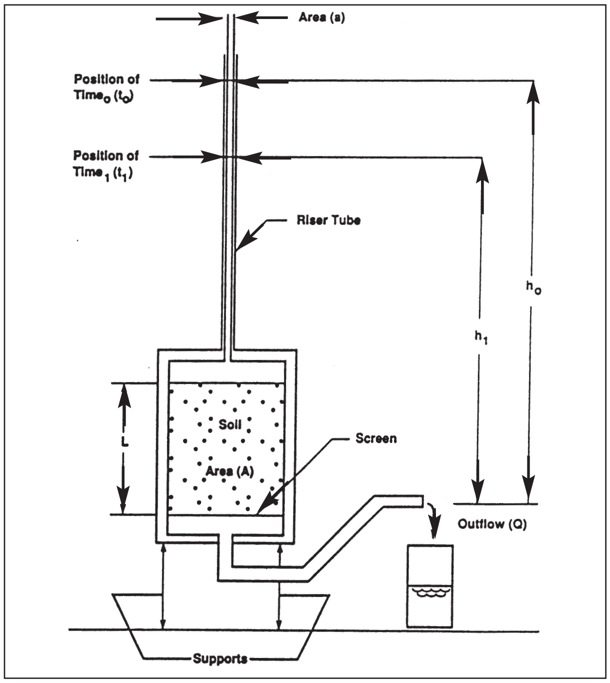


Figure 4.3: Falling Head Lab Test

The following equation may be used to establish the coefficient of permeability:

$$K = 2.3\, a\, L \log(h_0 / h_1) / (A\, d_t)$$

Where:
K = coefficient of permeability (m/s)
a = riser tube cross sectional area (mm^2)
L = length of the soil specimen (m)
h_0 = initial head (mm)
h_1 = final head (mm)
A = soil specimen cross sectional area (mm^2)
d_t = time interval (s)
= (t_1 - t_0)

Laboratory test specimens are mixtures of disturbed materials. The tests may therefore give permeabilities higher or lower than in situ materials. A factor of safety of 2 is commonly used to account for possible differences between laboratory and in situ values.

Indirect Methods

Indirect methods are the least accurate methods and are used when field or laboratory percolation tests have not been performed. They may be used for preliminary design purposes, however, further field permeability testing should be conducted before final design.

CHAPTER FOUR

The simplest of these methods is the use of SCS type soil classification maps. Since the maps only give a general idea of the basic soil types occurring in various areas, the soil classification should be verified by field investigation. Such maps will indicate in general the expected drainage characteristics of the soil classified as good, moderate, or poor drainage. This information may aid the designer in preliminary infiltration drainage feasibility studies.

Maximum Infiltration Rate

The following procedures are taken from the King County, Washington, Stormwater Management Manual. The maximum infiltration rate test can be used to estimate the maximum sub-surface vertical infiltration rate of the soil below a proposed infiltration system. The test is designed to simulate the physical process that will occur during design storm event conditions, therefore, a saturation period is required to approximate the soil moisture conditions that would occur during a major storm event. The procedure is as follows:

(1) Excavations shall be made to the bottom elevation of the proposed infiltration facility. The maximum infiltration rate of the underlying soil shall be determined using either the EPA falling head percolation test procedure (Design manual - Onsite Wastewater Treatment and Disposal Systems, EPA, 1980) or the double ring infiltrometer test (ASTM D3385).

(2) The test hole or apparatus is filled with water and maintained at depths above the test elevation for a period of not less than 4 hours (the saturation period).

(3) Following the saturation period, the rate shall be determined in accordance with the specified test procedures, with a head of 150 mm of water.

(4) The engineer shall perform sufficient tests to determine a representative infiltration rate for the site, but at least three tests shall be performed for each proposed tank or pond site, and at least 5 tests per hectare and a minimum of 4 tests shall be performed for each closed depression. The measured rate shall have the following factor of safety applied: for the EPA method, F.S. = 2.0; for ASTM D3385, F.S. = 1.75.

(5) A minimum of two soils logs, extending a minimum of 1.5 m in depth below the bottom of each proposed tank and a minimum of two soils logs, extending a minimum of 1.5 m in depth below the bottom of each proposed pond shall be obtained for each 1000 m^2 (plan view area) of proposed pond infiltration surface area. The logs shall describe the S.C. S. series of the soil, the textural class of the soil horizon through the depth of the log, and note any evidence of high groundwater level, such as mottling.

Pretreatment for Infiltration Systems

Pretreatment is required for infiltration systems to remove high sediment loads that can lead to clogging of the backfill material. The same types of pretreatment systems used for sand filters may be applied to infiltration systems. These include:

- Sedimentation Basins
- Vegetative Filter Strips
- Grass Swales
- Catch Basins with Sumps
- Water Quality Structures (Oil/Grit Separators)

Design Procedure and Factors

The required depth of the infiltration system can be estimated based on the following equation:

$$\mathbf{d = f_c\, T_s}$$

Where:
d = system depth (m)
f_c = soil infiltration rate (m/s)
T_s = detention time (s)

The detention time is typically 24 to 72 hours with 72 hours being the most common time used. The objective is to have most of the runoff recharged back to the ground before the next storm event occurs.

A minimum of 1.2 m of clearance is usually required between the bottom of the infiltration basin or trench and the bedrock or seasonal high water table.

The required storage volume in the infiltration system is equal to the WQV. Most jurisdictions allow credit for storage provided in the voids of the stone backfill. Therefore, the total storage is computed as the volume of the pipe plus the backfill volume times the void ratio. The backfill material should have a D_{50} between 40 and 80 mm (unless otherwise specified) which gives a void ratio between 0.3 and 0.4.

Subsurface disposal techniques have various applications that will result in both environmental and economic benefits. In designing any subsurface disposal system, it should be realized that for many applications the rate of runoff is considerably greater than the rate of infiltration. Some form of detention will therefore be required for most subsurface disposal facilities.

Modifications can be made to existing systems to take advantage of the infiltration capacity of the soil.

Pollutant Removal

Infiltration systems remove fine and soluble particulate pollutants from runoff by slowly infiltrating the water back into the soil. Removal mechanisms include adsorption, filtering, and microbial decomposition in the upper soil and backfill material. Larger sediments are removed in the pretreatment process. Very little monitoring data is available for the performance of infiltration systems. Table 4.5 provides projected removal efficiencies that are most commonly used. These removal efficiencies are dependent on the detention time.

Table 4.5: Estimated Pollutant Removal Rates for Infiltration Systems

Pollutant	Removal Rate %
Sediment	60 - 100
Total Phosphorus	40 - 80
Total Nitrogen	40 - 80
Trace Metals	40 - 100
BOD	70 - 100
Bacteria	60 - 100

Inspection and Maintenance

Infiltration systems generally do not require a great deal of maintenance but should be inspected during the first few months of operation to ensure that they are functioning properly. After that, the systems may be inspected on an annual basis unless the performance dictates otherwise. If pretreatment devices are used, they should be inspected and maintained properly.

Chapter Five: Structural Design

After the detention system has been hydraulically designed to meet the required storage volume, its structural design must be considered. Specifically, the corrugation profile and the steel thickness must be determined, so that the final installation will have strength and stiffness to adequately resist the live and dead loads present. The tables subsequently presented in this chapter simplify this process. The following discussion of loads and design considerations provides a background for the tables.

LOADS

Underground conduits are subject to two principal kinds of loads:

(1) dead loads developed by the embankment or trench backfill, as well as stationary (permanent) superimposed uniform or concentrated surface loads; and

(2) live loads moving (temporary) loads, including an allowance for impact.

DEAD LOAD

The dead load is considered to be the soil prism over the pipe. The unit pressure of this prism acting on the horizontal plane at the top of the pipe is given by:

$$DL = w\,H$$

Where:
w = unit weight of soil (kN/m^3)
H = height of fill over pipe (m)
DL = dead load pressure (kPa)

LIVE LOAD

Live loads are greatest when the height of cover over the top of the pipe is small and decrease as the fill height increases. Standard highway loads are referred to as CSA-S6 CS-600 and AASHTO H-25 live loads, and standard railroad loads are referred to as AREA E-80 live loads. Table 5.1 gives the live load pressure, LL, on the pipe for CS-600, H-25, and E-80 live loads for various heights of cover.

It should be noted that the tire prints of CS-600 and H-25 are similar. There are minor differences in the impact factor and method of pressure distribution through the soil.

Table 5.1: Highway and Railway Live Load Pressures

Highway loading[1]			Railway E-80 loading[1]	
Depth of cover (m)	Load (kPa) CS-600[2]	Load (kPa) H-25[3]	Depth of cover (m)	Load (kPa)
0.3	138	126		
0.5	71	63		
0.75	39	34		
1	26	22	1	147
1.25	19	16	1.2	133
1.5	14	12	1.5	115
1.75	11	10	2	91
2	9	8	3	53
2.25	8	7	4	34
2.5	6	6	6	15
2.75	5	5	8	7
			9	5

Notes: 1. Neglect live load when less than 5kPa.
2. Load distribution through soil according to CSA-S6.
3. Load distribution through fill according to AASHTO methods.

DESIGN STEPS

The structural design of underground detention systems consists of the following steps:

1. Select the backfill density to be required or expected.
2. Calculate the design pressure.
3. Compute the compression in the pipe wall.
4. Calculate the allowable compressive stress.
5. Determine the wall thickness required.
6. Check minimum handling stiffness.
7. Check seam strength requirements.

BACKFILL DENSITY

Select a percent compaction of pipe backfill for design. The value chosen should reflect the importance and size of the structure, and the quality of backfill that reasonably can be expected. The recommended value for routine use is 90%. However, for more important structures in higher fill situations, select higher quality backfill and specify the same for construction. This can increase the allowable fill height and/or save on metal thickness, although stormwater detention systems are usually under low cover.

DESIGN PRESSURE

When the height of cover is equal to or greater than the span or diameter of the structure, the total load (total load is the sum of the dead and live loads) can be reduced by a factor of K which is a function of soil density and which accounts for soil arching over the pipe.

- 85% Standard Proctor Density, K = 0.86
- 90% Standard Proctor Density, K = 0.75
- 95% Standard Proctor Density, K = 0.65

A conservative design uses the K value for a Standard Proctor Density of 85%.

If the height of cover is less than one pipe diameter, which is often the case with underground detention systems, the total load is assumed to act on the pipe.

$$\mathbf{P_v = K\,(DL + LL),\ when\ H \geq S}$$

$$\mathbf{P_v = (DL + LL),\ when\ H < S}$$

Where:
P_v = design pressure (kPa)
K = load factor
DL = dead load pressure (kPa)
LL = live load pressure (kPa)
H = height of cover (m)
S = diameter or span (m)

RING COMPRESSION

With the inherent flexibility of corrugated steel pipe, the vertical load on the top of the pipe causes a slight downward deflection of the top of the pipe and a resulting outward deflection of the sides of the pipe. The sides of the pipe push against the compacted side fill material to mobilize passive earth pressure. Thus, the pipe is loaded by radial pressure. For round pipes, the pressure around the periphery tends to be approximately equal, particularly at deep fill heights.

The radial pressure develops a compressive thrust in the pipe wall, and the pipe must have sufficient structural strength to resist this load. Accordingly, the stress in the pipe wall may be determined and compared to recognized allowable values to prevent yielding and buckling. Such allowable values have been derived from destructive tests done in extensive research programs, against which a safety factor of 2.0 is applied.

The compressive thrust in the conduit wall, or “ring compression”, is equal to the radial pressure acting on the wall multiplied by the pipe radius. The ring compression is an axial force acting tangentially to the conduit wall. For conventional structures in which the top arc approaches a semicircle, it is convenient to substitute half the span for the wall radius. The formula for ring compression is:

$$\mathbf{C = P_v\ S / 2}$$

Where:
C = ring compression (kN/m)
P_v = design pressure (kPa)
S = diameter or span (m)

ALLOWABLE WALL STRESS

The ultimate compressive stress, f_b, for corrugated steel structures with backfill compacted to 85% Standard Proctor Density and a minimum yield strength of 230 MPa, is expressed by equations. The first is the specified minimum yield strength of the steel which represents wall crushing or yielding. The second represents the interaction of yielding and ring buckling, and the third represents ring buckling. These equations are, respectively:

$$f_b = f_y = 230 \text{ MPa, when } D / r \leq 294$$

$$f_b = 275 - (558 \times 10^{-6})(D / r)^2, \text{ when } 294 < D / r \leq 500$$

$$f_b = (34 \times 10^{6}) / (D / r)^2, \text{ when } D / r > 500$$

Where:
f_b = ultimate compressive stress (MPa)
f_y = specified minimum yield strength (MPa)
D = diameter or span (mm)
r = radius of gyration (mm) for an assumed corrugation profile and wall thickness

The radius of gyration is presented with other corrugation section properties in Table 5.2 for various corrugations and wall thicknesses.

Table 5.2: Corrugation Section Properties

Corrugation profile (mm)	SPECIFIED THICKNESS (mm)									
	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	RADIUS OF GYRATION, r (mm)									
68 x 13	4.332	4.345	4.374		4.402		4.433			
76 x 25	8.666	8.685	8.724		8.758		8.794			
125 x 26	9.277	9.287	9.308		9.326		9.345			
152 x 51				17.326		17.375		17.425	17.475	17.523
19 x 19 x 190	6.553	6.350	6.020		5.791					
	CROSS-SECTIONAL WALL AREA (mm^2/mm)									
68 x 13	1.512	1.966	2.852		3.621		4.411			
76 x 25	1.736	2.259	3.281		4.169		5.084			
125 x 26	1.549	2.014	2.923		3.711		4.521			
152 x 51				3.522		4.828		6.149	7.461	8.712
19 x 19 x 190	1.077	1.507	2.506		3.634					
	MOMENT OF INERTIA, I (mm^4/mm)									
68 x 13	28.37	37.11	54.57		319.77		86.71			
76 x 25	130.40	170.40	249.73		322.74		393.12			
125 x 26	133.30	173.72	253.24				394.84			
152 x 51				1057.25		1457.56		1867.12	2278.31	2675.11
19 x 19 x 190	46.23	60.65	90.74		121.81					

A factor of safety of 2 is applied to the ultimate compressive stress to obtain the design stress.

$$\mathbf{f_c = f_b / 2}$$

Where:
f_c = design stress (MPa)
f_b = ultimate compressive stress (MPa)

WALL THICKNESS

The required wall area is computed from the calculated ring compression in the pipe wall and the allowable stress.

$$\mathbf{A = C / f_c}$$

Where:
A = area of pipe wall (mm^2/mm)
C = ring compression (kN/m)
f_c = design stress (MPa)

Values of A are given in Table 5.2 for various corrugations and wall thicknesses.

HANDLING STIFFNESS

Minimum pipe stiffness requirements, for practical handling and installation without special considerations or attention, have been established through experience and formulation. The resultant flexibility factor, FF, limits the size of each combination of corrugation and metal thickness.

$$\mathbf{FF = D^2 / E\ I}$$

Where:
FF = flexibility factor (mm/N)
E = modulus of elasticity (MPa)
= $200 \times (10)^3$ MPa
D = diameter or span (mm)
I = moment of inertia of wall (mm^4/mm)

Recommended maximum values of FF, for various corrugations, are:

- 68 x 13 mm FF = 0.245 mm/N
- 125 x 26 mm FF = 0.188 mm/N
- 76 x 25 mm FF = 0.188 mm/N
- 152 x 51 mm FF = 0.114 mm/N

Higher values can be used with special care or where experience allows. A trench installation condition, as in sewer design, and arches on footings are two examples where flexibility limits may be exceeded. Guidance can be found in ASTM A796 "Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches and Arches for Storm and Sanitary Sewers and Other Buried Applications" and in the "Handbook of Steel Drainage and Highway Construction Products" available from the Corrugated Steel Pipe Institute.

LONGITUDINAL SEAM STRENGTH

Most pipe seams develop the full yield strength of the pipe wall. However, there are exceptions in standard pipe manufacture. Shown underlined in Tables 5.3 and 5.4 are those standard riveted and bolted seams which do not develop a strength equivalent to the yield strength. The maximum ring compression should not exceed the ultimate longitudinal seam strength divided by a factor of safety of 2.

Table 5.3: Riveted CSP - Ultimate Longitudinal Seam Strength (kN/m)

Design base steel thickness (mm)	8 mm RIVETS		10 mm RIVETS			12 mm RIVETS
	68 x 13 mm		68 x 13 mm		76 x 25 mm	76 x 25 mm
	SINGLE	DOUBLE	SINGLE	DOUBLE	DOUBLE	DOUBLE
1.6	236	274			387	
2.0	261	401			499	
2.8			341	682		769
3.5			356	712		921
4.2			372	746		1023

Table 5.4: Structural Plate CSP - Ultimate Longitudinal Seam Strength (kN/m)

Specified wall thickness (mm)	Bolts per Corrugation		
	2	3	4
3.0	745	-	-
4.0	1120	-	-
5.0	1470	1650	-
6.0	1840	2135	-
7.0	2100	2660	3200

DEPTH OF COVER

Tables for the selection of the wall thickness, depending upon the pipe diameter or span and the depth of cover requirements, are presented as Tables 5.5 through 5.10. The tables are for circular pipes, pipe-arches or arches of a particular corrugation profile. The tables include the effect of live loads that do not exceed a CS-600, H-25 or E-80 live load, as indicated.

Table 5.5: Round CSP, Depth of Cover (68 mm x 13 mm corrugation)

	Minimum cover		Maximum cover (m)				
			Specified wall thickness (mm)				
Diameter (mm)	CS-600 or H-25 (mm)	E-80 (mm)	1.6	2.0	2.8	3.5	4.2
300	300	300	70	91			
400	300	300	53	68			
500	300	300	42	54	79		
600	300	300	35	45	66		
700	300	300	30	39	57		
800	300	300	26	34	50		
900	300	300	23	30	44	56	
1000	300	300	21	27	40	50	63
1200	300	300		23	33	42	52
1400	300	500			27	35	43
1600	300	500			22	28	35
1800	300	500				22	27
2000	400	500					22

Notes:
1. Backfill around pipe must be compacted to a specified S.P.D. of 90%.
2. Use reasonable care in handling and installation.

Table 5.6: Round CSP, Depth of Cover (75 mm x 25 mm corrugation)

	Minimum cover		Maximum cover (m)				
			Specified wall thickness (mm)				
Diameter (mm)	CS-600 or H-25 (mm)	E-80 (mm)	1.6	2.0	2.8	3.5	4.2
1200	300	500	20	26	38		
1400	300	500	17	22	32	41	
1600	300	500	15	19	28	36	44
1800	300	500	13	17	25	32	39
2000	400	500	12	15	22	29	35
2200	400	700	11	14	20	26	32
2400	400	700		13	19	24	29
2700	500	700		11	16	21	25
3000	500	1000			14	17	21
3300	600	1000			12	15	18
3600	600	1000				12	15

Notes:
1. For E-80 loading the following minimum steel thicknesses apply: 2 mm for 2700 to 3000 mm diameter, and 2.8 mm for 3000 to 3600 mm in diameter.
2. Backfill around pipe must be compacted to a specified S.P.D. of 90%.
3. Use reasonable care in handling and installation.

Table 5.7: Round CSP, Depth of Cover (125 mm x 26 mm corrugation)

	Minimum cover		Maximum cover (m)				
			Specified wall thickness (mm)				
Diameter (mm)	CS-600 or H-25 (mm)	E-80 (mm)	1.6	2.0	2.8	3.5	4.2
1200	300	500	18	23	34		
1400	300	500	15	20	29	35	45
1600	300	500	13	18	25	31	39
1800	300	500	12	16	22	28	35
2000	400	500	11	14	20	25	31
2200	400	700	10	12	18	23	29
2400	400	700	9	11	17	21	26
2700	500	700		10	15	18	23
3000	500	1000			13	16	21
3300	600	1000			11	14	19
3600	600	1000				12	17

Notes:
1. For E-80 loading the following minimum steel thicknesses apply: 2 mm for 2700 to 3000 mm diameter, and 2.8 mm for 3000 to 3600 mm in diameter.
2. Backfill around pipe must be compacted to a specified S.P.D. of 90%.
3. Use reasonable care in handling and installation.

Table 5.8: CSP Pipe-Arch, Depth of Cover, CS-600 or H-25 Live Load (68 mm x 13 mm corrugation)

Span (mm)	Rise (mm)	Minimum specified wall thickness (mm)	Maximum depth of cover (m) over pipe for the following corner bearing pressures		
			200 kPa	300 kPa	400 kPa
450	340	1.6	6.0	8.9	11.9
560	420	1.6	5.9	8.9	11.8
680	500	1.6	5.9	8.8	11.7
800	580	1.6	5.8	8.7	11.7
910	660	1.6	5.8	8.7	11.6
1030	740	1.6	5.7	8.6	11.5
1150	820	2.0	5.7	8.5	11.3
1390	970	2.8	5.6	8.4	11.2
1630	1120	3.5	5.5	8.2	11.0
1880	1260	3.5	5.4	8.1	10.8
2130	1400	3.5	5.3	8.0	10.6

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. The remaining backfill around the pipe-arch must be compacted to a specified S.P.D. of 90%.
2. Use reasonable care in handling and installation.
3. Minimum cover is maximum of span / 6 or 450 mm for 200 kPa bearing capacity, 350 mm for 300 kPa bearing capacity, and 300 mm for 400 kPa bearing capacity.

Table 5.9: CSP Pipe-Arch, Depth of Cover, CS-600 or H-25 Live Load (76 mm x 25 mm and 125 mm x 26 mm corrugation)

Span (mm)	Rise (mm)	Minimum specified wall thickness (mm)		Maximum depth of cover (m) over pipe for the following corner bearing pressures		
		76 x 25	125 x 26	200 kPa	300 kPa	400 kPa
1330	1030	1.6	1.6	5.6	8.4	11.2
1550	1200	1.6	1.6	5.5	8.2	11.0
1780	1360	1.6	2.0	5.4	8.1	10.8
2010	1530	2.0	2.0	5.3	8.0	10.6

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. The remaining backfill around the pipe-arch must be compacted to a specified S.P.D. of 90%.
2. Use reasonable care in handling and installation.
3. Minimum cover is maximum of span / 6 or 450 mm for 200 kPa bearing capacity, 350 mm for 300 kPa bearing capacity, and 300 mm for 400 kPa bearing capacity.

Table 5.10: CSP Pipe-Arch, Minimum Wall Thickness, E-80 Live Load (68 mm x 13 mm corrugation)

Span (mm)	Rise (mm)	Minimum specified wall thickness (mm) for the following depth of cover range			
		0.6 to 0.9 m	0.9 to 1.5 m	1.5 to 2.4 m	2.4 to 4.6 m
560	420	2.8	2.0	2.0	2.0
680	500	2.8	2.8	2.0	2.0
800	580	3.5	2.8	2.8	2.8
910	660		3.5	2.8	2.8
1030	740		3.5	3.5	2.8
1150	820			3.5	3.5
1390	970				3.5

Notes: 1. Required corner bearing capacity at maximum cover is 300 kPa. Remaining fill to be compacted to S.P.D. of 90%.
2. Use reasonable care in handling and installation.

CHAPTER FIVE

cspi

Chapter Six: Construction & Installation

SUBSURFACE SOIL INFORMATION

Information regarding subsurface soil conditions is often included as a part of the construction plans and specifications. This information is used to facilitate the design of the project, and also to aid the contractor in planning his construction procedure. Often, soil information that is adequate for design does not contain sufficient detail to meet the needs of the contractor. For this reason, it may be advantageous to obtain additional subsurface information. This may be accomplished through tests performed by the contractor or by engaging the services of a geotechnical engineer.

The purpose of a subsurface soils investigation is to determine:

- The types of soils that will be encountered in the construction area.
- The presence of rock.
- The thickness of various strata.
- The behavior of soils during and after excavation.
- The presence of ground water and the elevation of the ground water table.

FOUNDATION PREPARATION

Foundation requirements should be detailed on the drawings. However, field conditions often vary and differences may only be discovered during excavation. Special attention and design changes may be required as a result. Any alterations should first be approved by the engineer.

Although corrugated steel drainage structures can experience some differential settlement without disjointing or breaking, they should be placed on a firm, uniform foundation for best performance and long service life. All pipe systems must be installed with the area under the haunches (that portion of the pipe cross-section between the maximum horizontal dimension and the top of the bedding) well compacted and all voids filled.

For corrugated steel pipe, the most popular method of preparing the foundation and installing the pipe is by excavation to a flat surface, setting the pipe directly on the foundation, and then carefully tamping backfill under the haunches of the pipe. Proper backfill density can be achieved under the haunches by compacting the soil with a wooden pole, “2x4”, or small pneumatic tampers to eliminate all voids under the structure. See Figure 6.1 for typical foundations and placement of backfill under the haunches.

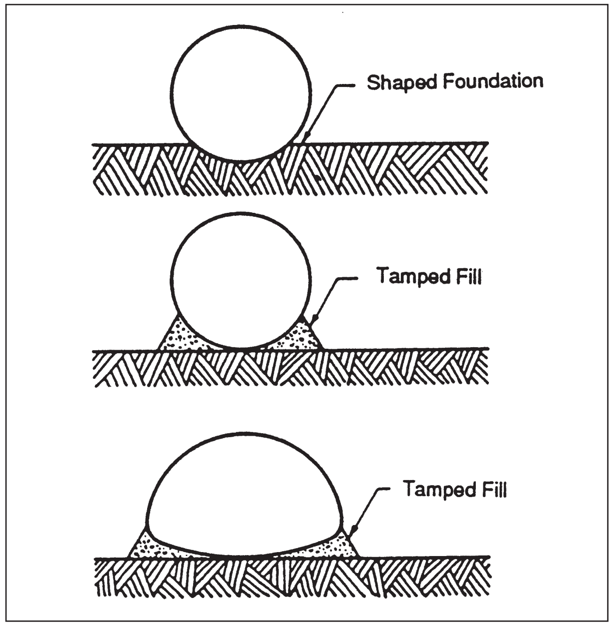


Figure 6.1: CSP Installation

This “flat foundation” technique works well except for larger relatively flat bottom radius structures such as pipe arches and horizontal ellipses. The shaped or vee foundation techniques work well for these structures, as shown in Figures 6.1 and 6.2.

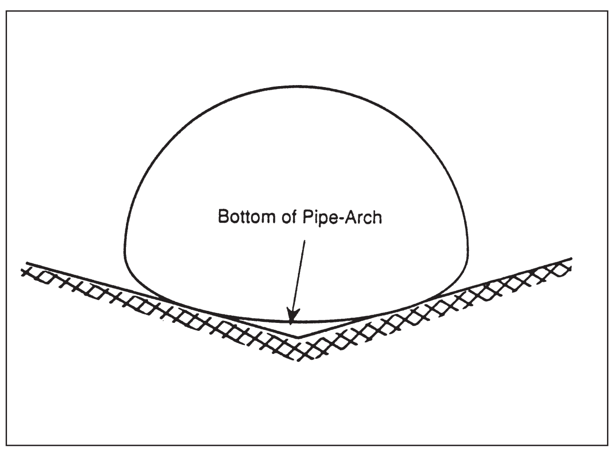


Figure 6.2: Shaped Vee Foundation

All pipe must be placed on stable earth or a fine granular foundation. Never install them on sod, frozen earth, highly plastic clay, corrosive material, organic matter or on a bed that contains boulders or rock larger than 75 mm in size. When poor foundations with low bearing strength are encountered, investigate the possibility of a change in pipe location. Otherwise, it may be necessary to stabilize the poor foundation by methods such as that described below.

A pipe performs better when all voids surrounding it, including the corrugations, are filled with backfill. While it is important that the foundation be such that it can support the pipe and loads, a 150 mm to 200 mm thick layer of loose bedding material provides a means for the bottom corrugations to be filled and for some advantageous settlement of the structure. This loose bedding can be either natural foundation material, if it is a quality material, or imported granular.

Care must be taken to prevent water flow along the outside of the pipe. When granular materials have been used for bedding, the ends of the fill should be sealed against infiltration. This can be done by providing a well compacted clay seal or by adding some type of end treatment such as an end section or a cut- off wall.

FOUNDATION PROBLEMS

If poor or non-uniform foundations are encountered, they must be improved to assure satisfactory results. The critical factor is to achieve uniformity along the pipe with a tendency for the foundation to yield under the pipe in relation to alongside the pipe.

Variable Support Foundations

When the excavated grade line reveals both soft and hard material, the foundation must be improved to make it as uniform as possible. Hard material can be excavated below grade and replaced with softer material. Alternatively, it may be more economical to excavate the entire foundation slightly below grade and replace it with suitable, uniform bedding material. In any event, any abrupt changes from hard to soft foundation must be avoided.

Soft Foundations

When soft, unstable material is encountered at the foundation level, it must be excavated below grade and backfilled to grade with sand, gravel, crushed stone or other suitable material. The zone of select material must be adequate to support the pipe and backfill. When unexpected materials are encountered during excavation, consult the engineer.

Pockets of Unstable Soil

If unstable foundation material is in small pockets, it is best to excavate all of the poor foundation and replace it with suitable bedding material. Frequently, a relatively thin mat of granular material will provide satisfactory support, but it may be necessary to replace very soft foundations to a depth great enough to support not only the pipe, but also the heavier backfill placed beside it.

Swampy Foundations

Corrugated steel pipe must not be placed in direct contact with pipe bents or concrete cradles that are installed to help provide the foundation. Such supports, if used, should be built with a flat top and covered with an earth cushion. In this way the flexible structure can develop side support without concentrating the load at any point.

Improved Foundations

Whenever a foundation is stabilized by using a coarse granular material, consideration of the bedding and backfill material becomes even more important. Fine materials can migrate into coarser material, so geotextile separators are recommended to prevent this migration.

Rock Foundations

Rock encountered in the foundation must be removed and replaced with bedding material to provide an adequate bedding thickness underneath the bottom of the structure. The amount of rock removal, as shown in Figure 6.3, depends on the amount of cover over the pipe. A general rule-of-thumb is to provide 50 mm per metre of cover, with a minimum of 150 mm and a maximum of 600 mm.

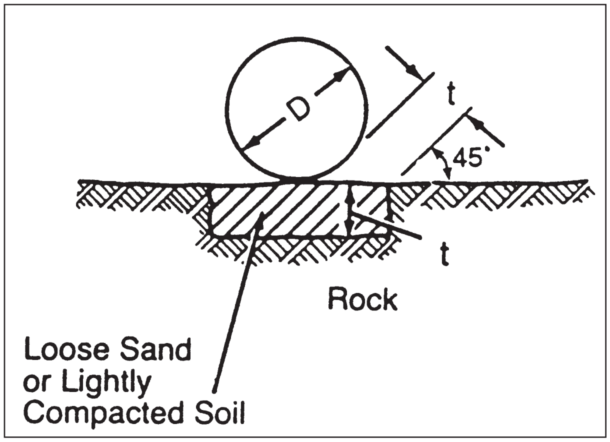


Figure 6.3: Rock Foundation Installation

CHAPTER SIX

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SETTLEMENT UNDER HIGH FILL LOADS

Cambering the center part of the foundation will compensate for unequal settlement under the weight of heavy embankments. This assures proper grade after settlement and prevents the structure from sagging in the middle as the foundation consolidates.

Generally, an adequate grade can be achieved by excavating the foundation for the upstream half of the pipe on an almost flat grade and the downstream half on a steeper than required grade. In this method, the inlet and outlet ends of the pipe are usually set at their desired elevations. It is important that the middle of the pipe not be set above the inlet end so that adequate grade is achieved even if the pipe does not settle as much as anticipated.

If camber is considered necessary based on foundation soil conditions and the final grade of the pipe is critical, the amount of camber should be determined by a qualified geotechnical engineer.

If the pipe is resting on cushioned rock or other adequate strength foundation, camber is not usually necessary as settlement will be minor.

CSP ASSEMBLY

UNLOADING AND HANDLING

PIPE MUST NEVER BE DUMPED DIRECTLY FROM A TRUCK BED ONTO THE GROUND WHILE UNLOADING.

Although corrugated steel drainage structures withstand normal handling, they should be handled with reasonable care. Dragging the pipe at any time may damage the coating. Also, avoid striking rocks or hard objects when lowering pipe into trenches.

Since corrugated steel pipes are relatively light weight, they can be handled with light equipment. The use of slings is recommended to properly and safely handle the pipe.

COUPLERS

The usual method of joining two or more lengths of pipe is by steel band couplers. The couplers are placed to overlap each pipe section equally so that they engage the ends of each pipe. The corrugations or indentations on the band must fit into the corrugations of each pipe. Some helical pipe is supplied with rerolled ends to accept annularly corrugated bands. Tightening of bolts or the installation of wedges draws the band tightly around the pipe, providing an integral and continuous structure.

One piece couplers are used for most installations of smaller sizes of pipe. Two- and three-piece couplers are used on larger diameter pipe and when installation conditions are difficult.

Corrugated or semi-corrugated couplers are used on levees, aerial sewers, steep slopes and similar installations where couplers that provide a more positive connection are essential.

Typical couplers are illustrated in Figure 6.4. Specially fabricated bolted, welded or riveted connectors can be supplied for special or unusual conditions.

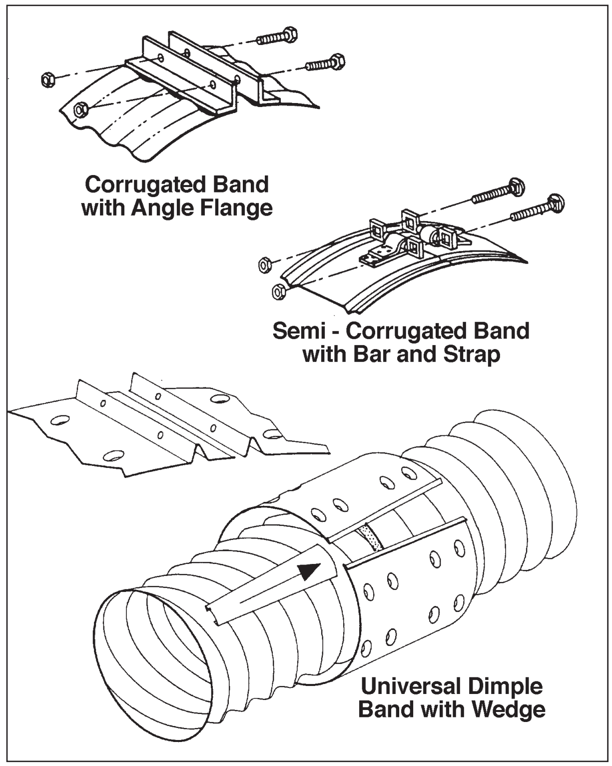

Figure 6.4: CSP Couplers

For installations requiring more watertight connections, pipe is provided with ends match marked and the couplers are provided with gaskets. If the pipe ends have been match marked, the pipes must be installed in the proper sequence.

INSTALLING COUPLERS

During the construction of a corrugated steel pipe system, care must be given to the assembly of joints to control both infiltration and exfiltration. Both processes will have an effect upon backfill materials since soil particle migration can occur as a result of water flow. This is particularly true when fine grained soils (fine sands and silts) are present in the backfill material. When necessary, a gasket, a geotextile wrap, or both can be used to control infiltration of fines.

Couplers are put into position at the end of one section of pipe with the band open to receive the next section. The next pipe section is positioned so that there is a space between pipe ends as dictated by the coupling system. After checking to see that connecting parts of both band and pipe sections match, and that the interior of the bands and exterior of the pipe are free of soil and stones, bolts are inserted and tightened.

To speed the coupling operation, a special cinching device can be used to draw the ends of the band together. The advantage of these devices is that they permit faster hand-tightening of the bolts, so that a wrench is required only for final tightening.

On large diameter pipe and asphalt coated pipe, tightening of the bolts will not assure a tight joint as a result of the friction between the coupler and the pipe. In these types of installations, tap the band with a mallet to loosen it as the bolts are tightened.

The wrench used to tighten coupling band bolts may need to have a deep socket. A ratchet wrench should be used for greater installation speed.

GASKETS

Couplers can be furnished with gaskets as described below and as shown in Figure 6.5. Gaskets are typically used to provide improved water tightness.

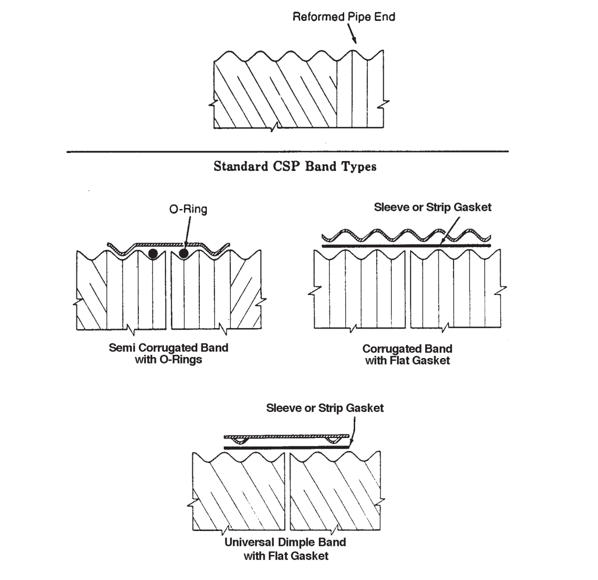

Figure 6.5: CSP Gaskets

Butyl rubber, neoprene, or closed cell rubber or rubber sponge gaskets are the basic types of materials used to provide;

(1) "O" ring gaskets which are recessed in an annular corrugation at the end of each pipe and confined by the coupling band,

(2) flat sleeve or strip gaskets placed on the end of each pipe and confined by the coupling band, or

(3) a single flat sleeve or strip gasket placed over the ends of both pipes and confined by the coupling band.

For tight fitting field installed gaskets, the tension in the gasket can be equalized by pulling the gasket away from the pipe, at several locations around the periphery, and letting go or "snapping" it.

On asphalt coated pipe it may be necessary to clean the corrugation into which the gasket must fit to allow it to properly seat.

The alignment and positioning of the pipe sections is extremely important when gaskets are used.

A lubricant must be applied to the gasket for proper installation as recommended by the manufacturer. The band should be tapped with a mallet to loosen it and let it slide against the lubricated gasket during bolt tightening.

When a leakage test is required for final acceptance, the contractor should conduct his own test after a few joints are assembled as a check of his assembly methods.

ASPHALT COATED PIPE

Although asphalt coated corrugated steel pipe is installed in the same manner as galvanized pipe, special attention should be given to attaching the couplers.

Contacting surfaces of the band and pipe need to be lubricated. This allows the band to easily slip around the pipe so that it can be properly tightened. Lubrication is especially required when surfaces are cold. In addition, tapping the bands with a mallet during tightening will loosen the band and let the lubricated surfaces slide over one another.

ALUMINIZED STEEL TYPE 2 PIPE

Corrugated steel pipe manufactured with Aluminized Steel Type 2 shall be installed in the same manner as galvanized corrugated steel pipe. Couplers shall be made of the same material type as the pipe.

STRUCTURAL PLATE CSP ASSEMBLY

Structural plate corrugated steel pipe is an assembly of corrugated plates that have been curved to a radius required by the shape of structure. The 152 mm x 51 mm corrugations are perpendicular to the axis of the structure. The plates are supplied in various lengths and widths so that they can be assembled into various cross sectional shapes and so that a longitudinal seam stagger can be used to ensure there are no four plate laps. Assembly of the plates is done through the use of 3/4" galvanized ASTM A 449 bolts and nuts.

Each structure is supplied with a detailed assembly drawing showing the plate and bolting arrangement.

Each plate is identified by a series of numbers stamped into the crest of an end corrugation near the middle of the plate. Additional symbols are used on cut end plates corresponding to details on the assembly drawings.

Bolting plates at the seams is best done by first placing bolts near the middle of the plates. Aligning bolt holes is easier when the bolts are left loose. Structural plate pipes should be assembled with as few bolts as possible until all plates are in place.

It is recommended that the plate assembly begin at the downstream (outlet) end of the structure. The plate stamp is located at the downstream end of the plate. On longitudinal seams, it is important that plates be assembled with the "valley" bolt closest to the visible plate edge. This rule of thumb applies regardless of the point of view; both inside the structure and outside.

Structural plate corrugated steel pipes are usually assembled in three stages; bottom, sides, and top. Bottom plates are set on the bedding beginning at the downstream end. Each successive bottom plate, or row of plates, is set in place so that it laps one corrugation of the preceding plate. The bolt holes are aligned and bolts are inserted and tightened as successive plates are added.

After a number of bottom plates have been assembled, aligned and bolts tightened, side plates are attached to each side of the bottom plates, again starting at the downstream end. It is important to have correct orientation of the longitudinal seams, as previously discussed.

Structural plate pipes should be assembled with as few bolts as possible until all plates are in place. The recommended progression for bolting is to insert three or four loose bolts near the centre of the plate along the longitudinal and circumferential seams. Long service bolts are usually supplied by the manufacturer to temporarily draw the plates together until the other bolts can be inserted.

After the side plates have been assembled, top plates are set in place beginning at the upstream end and bolted into place as described above.

The remaining bolts can be loosely inserted after three or four rings have been assembled, always working from the centre of a seam toward the corner of the plate. Alignment of bolt holes by the use of a bar is done more easily when the bolts are loose.

Tighten nuts progressively and uniformly, starting at one end of the structure, after all plates have been assembled. A balanced progression of tightening is necessary to prevent spiraling of the structure. The recommended range for bolt torque is between 200 N.m and 350 N.m. DO NOT OVER TORQUE.

BACKFILLING

The load carrying capacity of any type of pipe is largely dependent upon proper backfilling. Corrugated steel pipes make use of side support provided by the backfill as they deflect under load. Therefore, to obtain maximum strength and prevent washing out and settlement, it is necessary that the backfill be comprised of satisfactory material, properly placed, carefully compacted and protected.

BACKFILL MATERIAL

The use of select, drainable materials as backfill is preferred as they result in superior pipe performance. However, many local fill materials or natural in-situ materials may be satisfactory if carefully placed and compacted. Consult the design engineer or a geotechnical engineer for proper backfill selection. Fill material must be free from sod, frozen clumps, highly plastic clay, corrosive material, organic matter and boulders or rock larger than 75 mm in size.

PLACEMENT AND COMPACTION OF BACKFILL

Enough emphasis cannot be placed on the necessity of adequate compaction of backfill. **Faulty compaction and the use of improper backfill material are the leading causes of trouble with flexible and rigid pipe installations, more than all other factors combined!**

For trench installations, backfill must follow as closely behind the excavation and assembly stages as possible. Embankment installations are typically backfilled after the entire structure, or a major portion of it, is assembled. Unless the embankment and backfill materials are placed and compacted simultaneously, one of them must be benched so the other can be compacted against it.

The backfill should, as previously described, be carefully compacted under the haunches. Special care should be taken in doing this for pipe-arches because of the significant pressures that backfill under the haunches may be subject to.

Continue placing the backfill equally on both sides of the pipe in 150 mm to 200 mm layers thoroughly compacting each layer to 90% Standard Proctor Density or to a density specified by the engineer. Such layers must extend to each side of the structure to the limits shown on the plans, to the side of a trench, or to the natural ground line.

One problem encountered in backfilling is the tendency for installation crews to dump backfill material in piles around the pipe. Such piles of material are seldom spread to give a maximum lift depth of 150 mm to 200 mm. If the placement crew works too fast, the compaction crew has difficulty adequately compacting the first lift before more backfill is placed in the trench.

The backfill immediately next to the pipe must be compacted by hand-operated methods, although heavy compaction equipment may be used in most embankment installations as long as it remains at least 1 metre from the edge of the pipe. Changes in dimension or plumb of the structure, as visually noted especially if a shape monitoring program is used, warn that heavy machines must work further away.

Spread and compact backfill material beside the pipe with equipment running parallel to, not at right angles to, the structure. Backfill must be placed so that the fill depth is maintained approximately equal on each side of the structure at all times. The maximum difference in elevation shall be 400 mm. Uneven placement can cause peaking or rolling of the structure, and must be avoided.

Once the backfill reaches the 3/4 rise elevation of the structure, light compaction equipment should be used running perpendicular to the structure. The same techniques of spreading shallow layers and compacting thoroughly must be followed as the fill covers the pipe. For the initial layers over the pipe, light hand or walk-behind compaction equipment is still necessary. This procedure should be followed until backfill material has been placed and compacted over the structure up to the minimum cover elevation.

After backfilling to an elevation over the top of the pipe which is the greater of 300 mm or 1/6th the diameter or span, and the soil-steel structure is "locked into place", further filling to grade may continue using procedures applicable to embankment construction.

The use of water flooding or jetting should be limited to compacting soils which are sufficiently permeable to dispose of the excess water and should not be used with cohesive soils. The subgrade must also be permeable and able to dispose of the water.

Slurry backfill can provide a viable alternative to soil backfill, particularly where the native soil is not suitable or installation speed is critical. Typical specifications describe a slurry with 40 kg of cement per cubic metre, 10 mm maximum aggregate size, and a 130 mm maximum slump, to achieve a minimum compressive strength of 690 kPa. The slurry backfill can be placed directly in the trench around the pipe without vibrating. Care must be taken to raise the level of the slurry on either side of the pipe at about the same rate. Also, it is important to estimate and control the pipe uplift to avoid damage. Uplift can be controlled by limiting the rate of placement and by placing weights, such as sand bags, internally or externally along the pipe. Further information may be found in "CSP Structure Backfill Alternatives," National Corrugated Steel Pipe Association, August, 1987, which is available through the Corrugated Steel Pipe Institute.

For multiple installations, sufficient space between the pipes must be allowed for compaction equipment to operate properly.

The bedding and backfill operation should be conducted entirely in the dry if at all possible, but with enough moisture to meet compaction requirements. Large pipes have been pre-assembled and rolled or lifted into the stream bed "in the wet" where it is not possible to build a coffer-dam and divert the stream. Such conditions make it very difficult to ensure good base preparation and proper backfill. Strength consideration must be made by the designer in these cases, and expert advice should be obtained on backfill procedures.

If backfill material is properly selected, well placed and then adequately compacted, there is little danger of anything going wrong with the installation.

Backfill must be placed and fully compacted to the minimum cover elevation over the structure before the pipe is subjected to highway or light construction loads. **When construction equipment that exceeds legal highway loads will cross the pipe, extra compacted fill, beyond that required for minimum or planned cover, is required. See the following discussion on Construction Loads.**

MULTIPLE STRUCTURE INSTALLATIONS

Backfill must be balanced across all pipes at all times. Placement may require a hoe, stonebucket, conveyor or other device to ensure that the backfill elevation is roughly the same on all sides of all the pipes. The design should have provided adequate room between the structures to operate the equipment required for proper compaction of the backfill. Flowable fills or slurry backfill, requiring no compaction effort, can be used in cases with minimal spacing between the pipes.

Whether the structure is large or small, keep in mind that the requirements of economical equipment should also be considered in determining spacing between the structures. For example, with larger structural plate pipes it may be desirable to utilize larger mobile equipment for compaction between the structures. The space between pipes should allow for the efficient selection and operation of compaction equipment.

FINAL BACKFILLING

Once the envelope of backfill material is placed and compacted around and over the pipe, the remainder of the fill should be placed and compacted to prevent settlement at the surface. The backfill material and compaction level specified is selected to prevent surface subsidence and to protect the pavement.

When sheet piling has been used to support the trench walls, fill and compact the voids left when it is withdrawn or cut it off at an elevation above the top of the pipe.

Final backfill is compacted by conventional methods. It has fewer restrictions on materials and layer thickness than the backfill in the envelope around and immediately above the pipe.

STRUCTURE PROTECTION

Often, construction loads exceed the finished design loads for the structure. Additionally, during the various phases of assembly and backfill, the structure typically is more vulnerable to loads and hydraulic forces because its backfill is not complete. The corrugated steel structure must be properly protected.

COMPACTION EQUIPMENT

Hand Compaction

For compacting the area under the haunches of a structure, a “2x4” pole is usually required. Hand tampers should weigh not less than 10 kilograms and have a tamping face not larger than 150 mm x 150 mm.

Mechanical Compactors

Most types of power tampers are satisfactory in all except the most confined areas. However, they must be used carefully and completely over the entire area of each layer to obtain the desired compaction. Avoid striking the structure with power tamping tools.

Roller Compactors

Where space permits, sheepsfoot (recommended for clays and silts only), rubber tired and other types of rollers - with the exception of smooth rollers - can be used to compact backfill. But the fill adjacent to the structure should be tamped with hand or hand-held power equipment.

Vibrating Compactors

Vibrating compactors can be used effectively on all types of backfill except heavy clays or other plastic soils. Small walk behind equipment is especially suited to trench installations.

Hydraulic Compaction

The use of water flooding and/or jetting for compacting backfill around the pipe is limited to compacting clean, granular soils. To be effective, the foundation below the pipe must be sufficiently permeable to carry the water down and away quickly. Backfill around and immediately above the pipe must be placed and compacted in individual lifts of 150 mm to 200 mm. Water jetting is best used only for the haunch areas, while other types of compaction equipment are suitable for the remainder of the backfill.

CONSTRUCTION LOADS

Frequently, it is necessary for heavy construction equipment to travel over installed corrugated steel structures during completion of grading, paving or other site work. Heavy construction equipment can impose concentrated loads far in excess of those the structure is designed to carry.

Adequate protection of the corrugated steel structure may require more than finished design fill. The amount of additional fill required depends on the equipment axle loads as well as the frequency of use.

Table 6.1 provides the minimum construction use cover for typical structure sizes and axle loads. While providing extra cover is a simple way to protect the structure, it must be maintained so that rutting and surface grading do not reduce its effect. A minimum crossing width of 7.5 metres is recommended for typical equipment.

Table 6.1: Minimum Cover (m) For Construction Loads (tonnes)

Pipe Span	Axle Load (tonnes)			
(mm)	8-23	23-34	34-50	50-68
300-1000	0.6	0.8	0.9	0.9
1200-1800	0.9	0.9	1.0	1.2
2000-3000	0.9	1.0	1.2	1.2
3200-3600	1.0	1.2	1.4	1.4

Minimum cover may vary, depending on local conditions. The contractor must provide the additional cover required to avoid damage to the pipe. Minimum cover is measured from the top of the pipe to the top of the maintained construction roadway surface. In unpaved situation, the surface must be maintained.

Temporary dead loads such as storage piles and crane placements must be evaluated as the structural capacity, loading balance, backfill support, adequate foundation strength and other factors may dictate.

Chapter Seven: Durability of Detention Systems

Corrugated steel pipe has been used successfully since the late 1800's for culvert and storm sewer applications throughout North America. While it continues to provide long service life in these applications, covering a wide variety of soil and water conditions, detention systems may last even longer.

As discussed below, durability of corrugated steel pipe is related to a combination of corrosion and abrasion of the invert. While corrosion can take place inside a detention structure, there is an absence of abrasion. This is due to the fact that there is very little velocity or bedload.

Since the initial applications before the turn of the century, an estimated 50,000 installations have been the subject of critical investigative research to establish durability guidelines. The behavior of both the soil side and the effluent side of the pipe have been studied extensively. These studies have shown that corrugated steel pipe generally provides outstanding durability with regard to soil side effects, and that virtually any required service life can be attained by selecting appropriate coatings and/or pavings for the waterside invert.

Of course, all pipe materials show some deterioration with time, and such effects vary with site conditions. To aid the engineer in evaluating site conditions and selecting the appropriate corrugated steel pipe system, the main factors affecting durability are described below.

FACTORS AFFECTING CSP DURABILITY

DURABILITY IN SOIL

The durability of metal pipe in soil is a function of several interacting parameters including soil resistivity, acidity (pH), moisture content, soluble salts and oxygen content (aeration).

However, all of the corrosion processes involve the flow of electrical current from one location to another (a corrosion cell). Thus, the higher the resistivity, the greater the durability.

It is important to note that the construction specifications for most detention systems require the placement of a select granular backfill material. Therefore, aggressive in-situ soils are removed and replaced with desirable neutral soils with high resistivities.

Most soils fall in a pH range of 6 to 8, which is favorable to durability. Soils with lower pH values (acidic soils), which are usually found in areas of high rainfall, tend to be more corrosive. Granular soils that drain rapidly enhance durability. Conversely, soils with a moisture content above 20 percent tend to be corrosive. High clay-content soils tend to hold water longer and therefore are more corrosive than well drained soils.

Soil moisture may also contain various dissolved solids from the soil itself. This can contribute to corrosion by lowering the resistivity. Conversely, many soil chemicals form insoluble carbonates or hydroxides on buried galvanized surfaces. This can reduce soil side corrosion. High levels of chlorides and sulfates will make a soil more aggressive.

DURABILITY IN WATER

Some have suggested that there is little difference in the durability of steel in still natural waters in the pH range of 4.5 to 9.5, because the corrosion products maintain a pH of 9.5 at the steel surface. While moving water may remove these products and increases the level of dissolved gases, this is generally not an issue with detention systems.

Increasing levels of dissolved oxygen and carbon dioxide can accelerate corrosion. The most important effect of carbon dioxide in water relates to its interference with the formation of the protective calcium carbonate films that frequently develop on galvanized pipe surfaces, particularly in hard waters.

Dissolved salts can increase durability by decreasing oxygen solubility, but can increase corrosion if they ionize and decrease resistivity.

RESISTANCE TO ABRASION

While abrasion can become significant where flow velocities are high in culverts and storm sewers, it is not a factor in detention applications.

STUDIES

Like most studies, the State of California study is based on actual field conditions of culverts and storm sewers. California surveyed the condition of corrugated steel pipe at hundreds of locations and developed a method to estimate life based on pH and resistivity. Investigations in Florida, Idaho, Georgia, and Nebraska showed that the California method was too conservative compared to their actual service experience.

The results of the various investigations illustrate the variety of conditions that can be found throughout North America, and emphasize the need to use local information when available. Nevertheless, the California method appears to be the most reasonable basis available for general use.

The California study included the combined effects of soil corrosion, water corrosion, and abrasion on the durability of corrugated steel pipe culverts that had not received special maintenance treatment. The pipe invert, which can easily be paved to extend life, was found to be the critical area.

The predictive method developed depended on whether the pH exceeded 7.3.

Where the pH was consistently less than 7.3, the study was based on pipes in high mountainous regions with the potential for significant abrasion. Also, at least 70 percent of the pipes were expected to last longer than indicated by the chart.

Where the pH was greater than 7.3, the study was based on pipes in the semi-arid and desert areas in the southern part of California. Durability under those conditions, which was generally excellent, would be dominated by soil side corrosion because the average rainfall was less than 250 mm per year and the flow through the invert was only a few times per year.

DURABILITY GUIDELINES

GALVANIZED CSP

The original California method, referred to previously, was based on life to first perforation of an unmaintained culvert. However, the consequences of small perforations in a storm sewer or detention system are usually minimal.

Therefore, the curves on the chart were converted by R. F. Stratfull to "average service life" curves, using data developed on weight loss and pitting of bare steel samples by the NIST (National Institute of Standards and Technology, formerly the National Bureau of Standards). Figure 7.1 shows the resulting chart for estimating the average invert service life when designing corrugated steel pipe culverts and storm sewers. Use of this chart for estimating service life of detention systems is a conservative approach.

This chart is often used to determine the average service life for steel structural plate applications as well. Structural plate steel thickness is often greater than 4.2 mm, so using this chart for that product may be overly conservative.

ALUMINIZED STEEL TYPE 2 CSP

Aluminized Steel Type 2 was developed in 1939 as a product intended to provide superior corrosion resistance. It combines the strength of a steel substrate with the corrosion resistance of aluminum. It is typically used in environments where galvanized steel has not performed well.

The aluminum layer spontaneously forms an aluminum oxide passive film. This film has a high resistance to major environmental factors influencing corrosion behavior in waters and soils. Corrosion due to dissolved oxygen and carbon dioxide, and erosion due to high velocity waters are the common influential factors in a pipe waterside environment.

Corrosion of the aluminum is typically by pitting. Pitting in the aluminum layer is arrested at the thick aluminum-iron intermetallic alloy layer. At this layer, pits grow horizontally rather than vertically. The hard alloy layer resists both corrosion and abrasion, as well as erosion corrosion.

Aluminized Steel Type 2 can provide a service life of between 1.3 times the service life for galvanized (derived from the AISI chart; Figure 7.1) and 75 years (possibly more), depending on specific environmental conditions.

CSP WITH PROTECTIVE COATING OR PAVING

The use of protective coatings or pavements will add additional service life to detention facilities. The difference between these two systems is that a pavement totally fills the invert corrugations while a coating is thinner and is intended to provide an additional layer of protection. The most commonly used material for protective coatings and paving is bituminous.

DESIGN LIFE

Many agencies do not have specific design life requirements. A specified design life of 25 or 50 years may be typical, with longer design lives used in some rare cases. The average service life for a detention system may not need to meet the project design life. For example, pavements and structures like bridge decks are often replaced or rehabilitated several times during the life of the project.

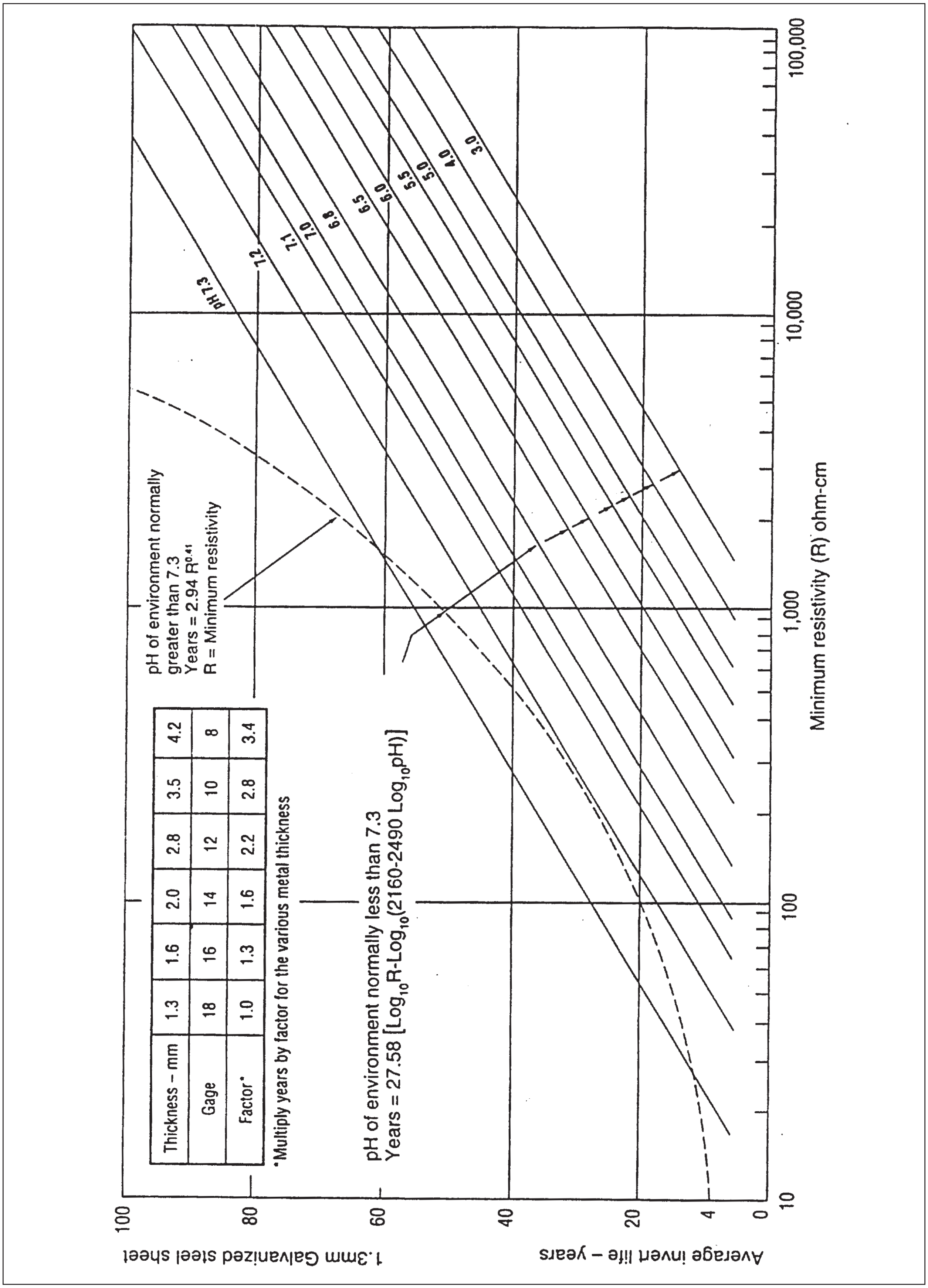

Figure 7.1: Average Service Life Prediction for CSP

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