Chapter 19*

DRAINAGE

19.1 INTRODUCTION

Drainage is the single most important factor in the design of infrastructures in mountainous areas. Infrastructures, such as roads, involve design of surface drains, sub-surface drains, drainage crossings, and erosion control measures.

Considerable damage to road pavements, retaining walls, and the surrounding hill slopes occurs from concentrated runoffs from drainage structures which are not properly designed according to hydrologic and hydraulic considerations. Designs of side drains and culverts, which are normally numerous along a road, based on guesswork, do save time and effort in the design process but, in many instances, create problems that subsequently involve more time, efforts, and resources.

This chapter, therefore, presents brief theoretical concepts on hydrology and hydraulics, followed by some simple methods for designing culverts. Practical aspects of designs for side drains and gully control are briefly dealt with in Chapter 24, Section 24. 5.

19.2 HYDROLOGY

The branch of hydrology which is of particular concern to road engineers deals with the following:

- frequency, intensity, and duration of rainfall,
- runoff peaks and their frequencies, and
- distribution of precipitation throughout the seasons, influencing the moisture under the road pavement, and growth of vegetation for erosion control.

19.2.1 *Intensity, Frequency, and Duration of Rainfall*

The intensity of rainfall is the rate of rainfall, usually measured in millimeters per hour. However sometimes it is measured as total precipitation in millimeters for a certain time period, e.g., total millimeters of rainfall in 24 hours.

Rainfall frequency is a term used to denote the probability that a rainfall event of T-yrs' recurrence interval will be equalled or exceeded in any one year. It is derived by:

The Weibull Formula is used to calculate frequency of rainfall from the available rainfall data.

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$$F = \frac{1}{T}.$$

$$F = (\frac{m}{n+1}) \times 100 \qquad (19.1)$$

where,

F = percentage of years during which the precipitation is equalled or exceeded, the precipitation of order number m, and
n = total number of precipitation values.

The recurrence interval (T) is given by,

$$T = \frac{1}{F} = \frac{n+1}{m} \quad . \qquad (19.2)$$

First, the available rainfall data are tabulated and ranked according to order of magnitude (Table 19.1). The return periods, T, are calculated by using the above formula. The data are then plotted on a semilog paper with 24 hour maximum rainfall on a normal scale at Y axis and return period on a logarithmic scale at X axis or the inverse of return period $(\frac{1}{T})$ in the upper x-axis from right to left as shown in Figure 19.1. Rainfall intensity for the desired return period can be read out or extrapolated from this graph. Alternatively, the frequency of rainfall, that is the probability that rainfall of the given intensity will be equalled or exceeded in any one year, is obtained from the graph by reading 1/T in the upper x-axis.

Table 19.1: Frequency analysis of 24 hours rainfall (Nuwakot)

Year	Precipitation, mm	Rank, m	Return Period (Year), T
1972	98	6	2.67
1973	135	2	8.00
1974	75	12	1.33
1975	80	10	1.60
1976	93	8	2.00
1977	69	13	1.23
1978	94	7	2.29
1979	178	1	16.00
1980	69	13	1.23
1981	60	14	1.14
1982	120	5	3.20
1983	77	11	1.45
1984	85	9	1.78
1985	128	4	4.00
1986	132	3	5.33

Source: Environmental Impact Study of the Mahendra Raj Marg - Gaighat Road and Trisuli - Somdang Road, Department of Roads, Nepal, 1991.

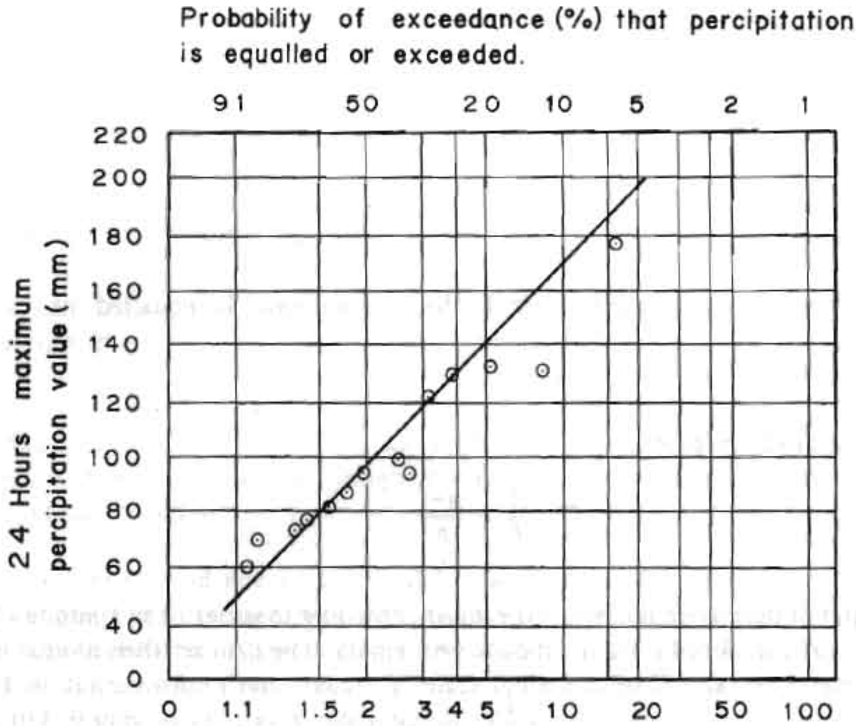


Fig. 19.1 Frequency of rainfall

Frequency analysis of rainfall data can be carried out by two methods.

(a) The annual series, for which only the maximum rainfall intensity of each year is plotted and other rainfall data of the year are ignored.

(b) The partial series, for which all the high rainfall intensities are considered regardless of the number occurring within a particular year.

The difference between two methods is insignificant for return periods greater than 10 years. For less than a 10 year return period, the partial series is generally used.

19.2.2 *Design Flood and Its Frequency*

The first step in designing drainage facilities is to estimate the quantity of water likely to be drained. Drainage facilities should have sufficient capacity to carry off safely not only peak runoffs, which occur frequently, say several times a year, but also larger runoffs, occurring less frequently, say on average once in 10 or more years. For a major highway linking major economic centres, where disruption of traffic, caused by damage to or washout of culverts, may not be acceptable, a peak runoff that recurs less frequently needs to be considered. In contrast, for a rural highway, where some minor traffic disturbances can be tolerated, a peak runoff that recurs frequently might be sufficient.

However, where serious damage would result from erosion caused by the inadequate capacity of drainage facilities, and where large drainage facility costs are involved, the less frequent peak runoffs would have to be used.

It is not practicable to design for a maximum probable flood to cater for the worst possible flood, as the capital costs increase rapidly with the increment of the peak runoff. In order to economise on construction costs, frequency of flood is selected for longer or shorter return periods, depending upon the importance of the structure. The design flood is a flood corresponding to the selected return period.

The **California Culvert Practice**, 1953, recommends a 10 year flood for the culvert to just pass the flood without static head at entrance and a 100 year flood for design of culvert and appurtenances to avoid serious damage from head and velocity.

Regarding the design of road surface drainage, the U.S. Bureau of Public Roads insists that all drainage facilities, other than culverts and bridges, be designed for a 10 year flood.

It should be kept in mind that a 25 year flood does not in any way mean that the flood will occur once in 25 years, rather there is only a 64 per cent probability that it will occur within a 25 year period. However, this flood has a 1 in 25 or 4 per cent chance of occurrence in any one year. The following example illustrates this point.

Let return period of a flood	T	= 25 yrs
Probability of exceeding	P	= 1/T
		= 0.04.
Probability that the flood will not occur in 25 years		$= (1-P)^{25}$
		= 0.36.
Probability that the flood will occur in 25 years		$= 1 - (1-P)^{25}$
		= 1 - 0.36
		= 0.64
		= 64 per cent.

19.2.3 ***Method of Runoff Prediction***

Current methods for estimating peak runoffs are discussed below.

(a) *Rational Formula*

This method is applicable to small catchments (1 to 2 sq.km) only. The peak runoff is given by:

$$Q = \frac{C I_c A}{360} \tag{19.3}$$

where,

- Q = Peak runoff in m^3/sec
- I_c = critical intensity in mm/hr, corresponding to time of concentration of catchment,
- A = catchment area in hectares, and
- C = dimensionless constant, the runoff coefficient (see Table 19.2).

The peak runoff frequency is assumed to be identical to the rainfall intensity frequency.

The critical intensity of rainfall I_c is determined as follows:

$$I_c = \frac{P\ (T+1)}{T\ (t_c+1)} \tag{19.4}$$

where,

- I_c = critical intensity of rainfall corresponding to time of concentration, in mm per hour
- P = precipitation of a storm in mm,
- T = duration of storm in hours, and
- t_c = time of concentration in hrs.

The time of concentration (t_c) is given by the formula below (The Indian Roads Congress, 1986):

$$t_c = [0.87\ \frac{L^3}{H}]^{0.385} \tag{19.5}$$

where,

- t_c = concentration time duration of a storm corresponding to the maximum rate of runoff (in hrs),
- L = length of the watercourse from the farthest point in the catchment to the outlet, (in km), and
- H = height difference between the farthest point and the outlet,m.

Table 19.2 Maximum values of runoff coefficient C for various soil covers

Steep, bare rock, also city pavements	0.90
Rocks steep, wooded	0.80
Plateau, lightly covered	0.70
Clayey soils, stiff and bare	0.60
Clayey soils, lightly covered	0.50
Loam, lightly cultivated or covered	0.40
Loam, predominantly cultivated	0.30
Sandy soil, light growth	0.20
Sandy soil, covered, heavy brush	0.10

(b) *U.S. Soil Conservation Service Curve Number Method*

The U.S. Soil Conservation Service has developed a method of estimating runoff from small watersheds of up to about eight hundred hectares based on soil and vegetative covers and moisture levels.

The runoff Q is expressed by the following formula:

$$Q = \frac{(P-I_a)^2}{(P+4I_a)} \qquad (19.6)$$

where,

Q	=	runoff in mm,
I_a	=	initial abstraction or loss due to infiltration, interception, and surface storage, and
P	=	design rainfall in mm.

The initial loss (I_a) is defined by a curve number (CN):

I_a = 0.2 S where, S = potential infiltration:

$$I_a = 5.08\ (\frac{1000}{CN} - 10)\ mm\ . \qquad (19.7)$$

The curve number (CN) represents the hydrological soil group, the land use type, and the antecedent moisture conditions. The range of C values for normal conditions is presented in Table 19.3.

The CN values obtained from Table 19.3 are for average moisture conditions and should be modified, if necessary, for relatively wet or dry conditions. The antecedent moisture condition levels and CN conversion for other moisture conditions can be taken from Tables 19.4 and 19.5.

The runoff can be directly read off from Figure 19.2, once the appropriate curve number and the design storm rainfall have been chosen.

c) *California Culvert Practice for Estimating Design Discharge*

Figure 19.3 is a nomograph for estimating the design discharge. The following example illustrates the use of this chart:

Given:

- o distance of the culvert from the critical point (furthest point in the watershed). L = 3 miles,
- o elevation difference between farthest points in the catchment and the culvert site, M = 900 ft.,
- o area of catchment A, = 2 square miles,
- o one hour rainfall,
  (precipitation in 60 minutes over a 100 yr. return period) P_{60} = 1.9, inches and
- o K for deciduous timberland = 60.

Table 19.3 Hydrological soil groups and on values for different types of land use

(for antecedent moisture condition II and Ia = 0.25)

Type of Soil	Infiltration Rate	Class
Deep sandy	High	A
Shallow sandy and medium textured	Moderate	B
Shallow with medium heavy texture	Slow	C
Clay and shallow/hard pans	Very slow	D

Land Cover	Land Use Type	Surface Runoff	Hydrological Soil Class			
			A	B	C	D
Fallow		Rapid	77	86	91	94
Row crops e.g., maize	Sloping cultivated land	Rapid	72	81	88	91
	Terraced land	Slow	67	78	85	89
Broadcast crops, e.g,	Sloping cultivated land	Rapid	65	76	84	88
upland Rice	Terraced land	Slow	63	75	83	87
Pasture or range	Heavily grazed, no plant cover	Rapid	68	79	86	89
	Moderately grazed, more than 50% plant cover	Moderate	49	69	79	84
	more than 75% plant cover	Lightly grazed, Slow	39	61	74	80
Forest	grazed, some litter present	Moderate	36	60	73	79
	litter and shrubs cover the soil	Slow	25	55	70	77
Dirt roads		Rapid	72	82	87	89
Roads, hard surface		Rapid	74	84	90	92

Source : Adapted from the U.S. Soil Conservation Service (3)

Table 19.4 Runoff curve number (CN), conversions, and constants

CN for Condition II (1)	CN for AMC I (2)	CN for AMC III (3)	S Values in** (4)	Curve starts where P = (in) (5)
100	100	100	0.000	0.0
98	94	99	0.204	0.04
96	89	99	0.417	0.08
94	85	98	0.638	0.13
92	81	97	0.870	0.17
90	78	96	1.11	0.22
88	75	95	1.36	0.27
86	72	94	1.63	0.33
84	68	93	1.90	0.38
82	66	92	2.20	0.44
80	63	91	2.50	0.50
78	60	90	2.82	0.56
76	58	89	3.16	0.63
74	55	88	3.51	0.70
72	53	86	3.89	0.78
70	51	85	4.28	0.86
68	48	84	4.70	0.94
66	46	82	5.15	1.03
64	44	81	5.62	1.12
62	42	79	6.13	1.23
60	40	78	6.67	1.33
58	38	76	7.24	1.45
56	36	75	7.86	1.57
54	34	73	8.52	1.70
52	32	71	9.23	1.85
50	31	70	10.00	2.00
48	29	68	10.8	2.16
46	27	66	11.7	2.34
44	25	64	12.7	2.54
42	24	62	13.8	2.76
40	22	60	15.0	3.00
38	21	58	16.3	3.26
36	19	56	17.8	3.56
34	18	54	19.4	3.88
32	16	52	21.2	4.24
30	15	50	23.3	4.66
25	12	43	30.0	6.00
20	9	37	40.0	8.00
15	6	30	56.7	11.34
10	4	22	90.0	18.00
5	2	13	190.0	38.00
0	0	0	Infinity	Infinity

Source : U.S. Soil Conservation Service, 1957

** S values for CN in column 1

Table 19.5 Rainfall limits for establishing antecedent moisture conditions

Antecedent moisture conditions class	5-day total antecedent rainfall mm	
	Dormat season	Growing season
I	Less than 12	Less than 35
II	12 to 28	35 to 54
III	Over 28	Over 54

Source: U.S. Soil Conservation Service 1957

To find:

o Design discharge for a 100 year return period:

from the nomograph (Figure 19.3)

o time of concentration, T = 41 min,
o for T = 41 min, and P_{60} = 1.9 inches, critical intensity, i = 2.2 inches per hour,
o for i = 2.2 in/hr and A = 2 sq.m total precipitation for a flood of 100 year return period, P = 2800 cu ft per sec, and
o for k = 60, and P = 2800 ft/sec, design discharge = 1700 cu ft per sec for 100 year flood.

This nomograph is convenient provided the one hour rainfall records are available. However, the one-hour precipitation for the desired return period in the design can be obtained from Equation 19.4 also.

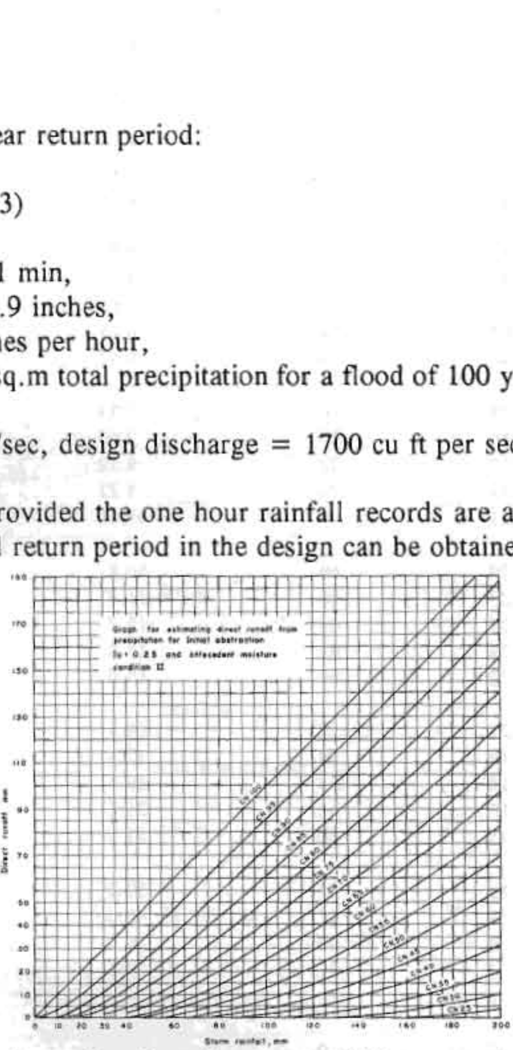


Fig. 19.2 Graph for estimating direct runoff from precipitation for initial abstraction I_a = 0.25 and antecedent moistore condition II

19.3 HYDRAULICS

19.3.1 *Hydraulics of Drainage Channels (Adapted from Compendium 5, Roadside Drainage, Transportation Research Board, 1979)*

Most highway drainage facilities such as roadside and catch drains, chutes, and culverts, flowing partially full, are designed according to principles of flow in open channels.

Flow in open channels is classified as steady and unsteady. Unsteady flow occurs when the quantity of water, cross sections of flow, and the slope of the carrying channels are changing. However, for simplicity of hydraulic calculations, flows in road drainage channels are treated as if occurring under steady conditions.

Steady flow can either be uniform or non-uniform (varied).

a) *Uniform Flow*

Uniform flow will take place when the cross section, roughness, and slope of the channel remain constant over the stretch under consideration. The errors involved in assuming uniform flow in drainage channels are relatively small compared to errors in establishing design peak flows, hence drainage channels with constant cross-section, roughness, and slope are often designed as uniform flow channels.

The most widely used equation for uniform flow is the following Manning Equation:

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{19.8}$$

where,

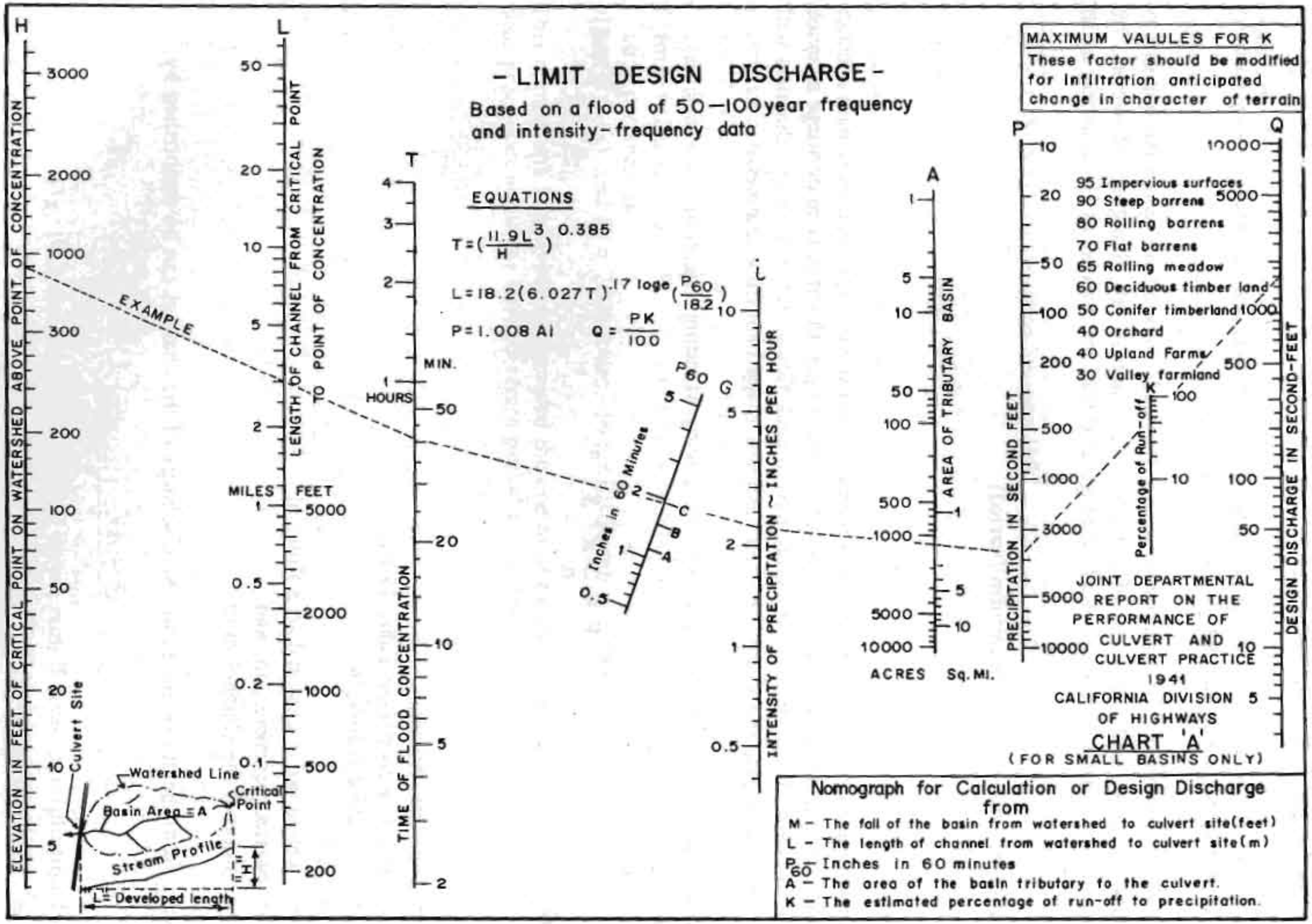
V	=	velocity, m/sec,
n	=	rugosity coeff (see Table 19.6),
R	=	hydraulic radius, m,
R	=	A/P,
A	=	wetted cross sectional area flow, m^2
P	=	wetted perimeter, m, and
S	=	slope of the channel m/m.

After finding velocity from the above equation, the discharge of the channel can be established by:

$$Q = AV$$

where,

Q	=	flow, m^3/sec,
A	=	cross-sectional area of flow, m^2, and
V	=	velocity of flow, m/sec.



Source: Adapted from California Culvert Practice 1957

Fig. 19.3 Nomograph for calculation of design discharge

Table 19.6 Manning roughness coefficient

Closed Conduits	
Concrete Pipe	0.011 - 0.018
Corrugated metal pipe	0.024
Cast iron pipe	0.013
Brick	0.014 - 0.017
Cement rubble masonry with natural floor	0.019 - 0.023
Open Channels	
Earthen, clean, recently completed	0.016 - 0.018
Earthen with short grass and weeds	0.022 - 0.027
Gravelly soil, clean, uniform	0.022 - 0.025
Earthen fairly uniform tides, clean cobble bottom	0.030 - 0.040
Concrete formed no finish	0.013 - 0.017
Concrete bottom, dressed stone sides	0.015 - 0.017
Cement rubble masonry	0.030 - 0.025
Brick	0.014 - 0.017
Mountain stream, no vegetation in channel, steep banks, trees and brush along bank submerged at high stage	
Bottom of gravel, cobbles, few boulders	0.04 - 0.05
Bottom of cobbles, with large boulders	0.05 - 0.07

Source : Compendium 5, Road Drainage, Transportation Research Board, 1979

Table 19.7 Typical safe velocities for different materials

S.No.	Bed Material	Safe velocity (m/s)
1.	Loose clay or fine sand	up to0.5
2.	Coarse sand	0.5 - 1.0
3.	Fine gravel, sandy or stiff clay	1.0 - 1.5
4.	Coarse gravel, rocky soil	1.5 - 2.5
5.	Boulders, rock	2.5 - 5.0

Source : Sharma 1985.

b) *Non-Uniform or Varied Flow****

Varied steady flow occurs when the quantity of water remains constant, but the depth of the flow, velocity, or cross-section changes from section to section. The relation of all cross-sections will be:

$$Q = A_1V_1 = A_2V_2 = A_nV_n \ . \tag{19.9}$$

Equation 19.9 is sometimes called the Equation of Continuity.

Velocity of uniform flow in open channels can be computed by the Manning Equation, using the slope of the channel bed as the slope of the energy line but non-uniform steady flow computations require other methods.

The hydraulic design engineer needs a knowledge of varied flow in order to determine the behaviour of the flowing water when changes in channel resistance, size, shape, or slope occur. A discussion of varied flow properly begins with a discussion of the energy of the flowing water.

c) *Energy of flow*

Water flowing in an open channel possesses energy of two kinds - **potential energy** and **kinetic energy**. The potential energy is due to the position of the water above a specific datum and kinetic energy is due to the velocity of the flowing water. In channel problems, energy is conveniently expressed in terms of **head**. Thus, a column of water 20 feet high has a potential (static) head of 20 feet with respect to the bottom of the column. Flowing water has both **potential head** and **velocity head**, the velocity head being equal to :

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

where,

V = the mean velocity in feet per second, and
g = acceleration of gravity or 32.2 feet per second^2.

A useful hydraulic concept of the energy of flowing water within one vertical cross-section of the channel is that of **specific head** (also called **specific energy**).

$$Specific\ head\ (H_o) = d + \frac{V^2}{2g} \tag{19.10}$$

*** This section up to and including the sub-heading section entitled, The Froude Number, is extracted from Text 1 of Compendium 5 of the Transportation Research Board of the National Academy of Sciences, pp 16-20, Washington D.C. 1978.

If the potential head is related to a datum (Figure 19.5) at or below the bed of the channel at the outlet, energy can be expressed in terms of total head. If Z is the elevation of the channel bottom, total head at any section is:

$$Total\ head\ (H) = d+\frac{V^2}{2g}+Z\ . \qquad (19.11)$$

The energy losses due to friction, channel contractions, changes in alignment, and other factors are termed head losses (h_L). The law of conservation of energy, (Bernoulli's Theorem), states that the total head at any section is equal to the total head at any section downstream plus intervening head losses, or, for the channel in Figure 19.5, is equal to the total head at Section 2, plus head loss between Sections 1 and 2, or:

$$d_1 + \frac{V_1^2}{2g}+Z_1 = d_2+\frac{V_2^2}{2g}+Z_2+h_L\ . \qquad (19.12)$$

In Figure 19.5, the head loss, in a channel of uniform cross-section, equals the change in Z or (Z_1-Z_2). Thus, the water surface is parallel to the streambed, and

$$d_1+\frac{V_1^2}{2g} = d_2+\frac{V_2^2}{2g}\ .$$

Thus the flow is uniform and can be computed by the Manning Equation. The head loss is:

$$(Z_1 - Z_2) = LS_o \qquad (19.13)$$

where,

L = horizontal distance between Section 1 and Section 2, and

S_o = channel slope or $\frac{Z_1 - Z_2}{L}$.

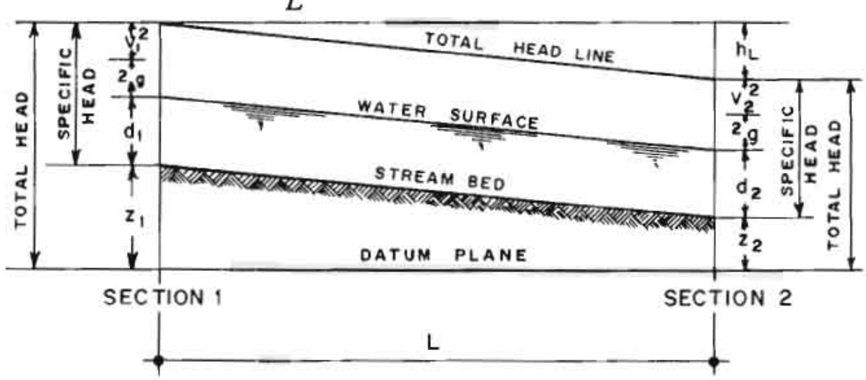


Fig. 19.4 Water-surface profiles of channel with uniform flow

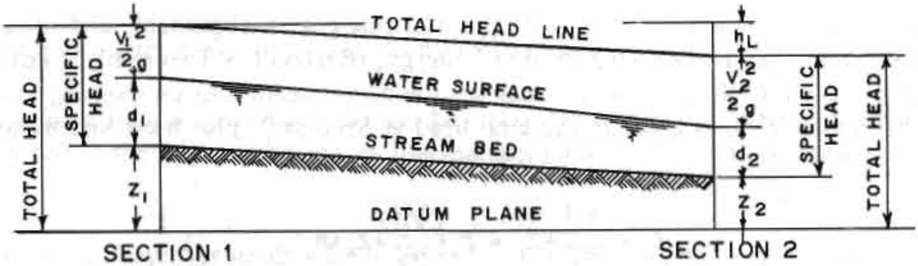


Fig. 19.5 Water-surface profile of channel with non-uniform flow

S_o in uniform flow is sometimes called the **friction slope**. For uniform flow, the Manning Equation can be computed for $S(= S_o)$.

$$S = \left(\frac{V_n}{1.49\ R^{2/3}}\right)^2 \tag{19.14}$$

When the head loss does not equal the change in Z, non-uniform flow occurs and the depth of the flow either increases or decreases in a uniform channel. In Figure 19.6, flow takes place with decreasing depth.

Between Sections 1 and 2, the velocity is increasing and the rate of energy loss is, therefore, not constant. This condition could be caused by a channel slope steeper than that needed to overcome frictional resistance or by a change in channel cross-section. Thus, the total head line (also called the energy line or energy gradient) is not a straight line. The water surface line in an open channel is sometimes called the hydraulic grade line.

d) *Critical Flow.*

With a constant discharge passing a cross-section, changing the depth of flow causes a different specific head for each depth. If specific head is plotted against depth of flow, the result is a specific head (energy) diagram (Figure 19.6).

The specific head curve is asymptotic to the line representing the energy caused by depth and the vertical line of zero depth. Examination of Figure 19.6 reveals several important facts. Starting at the upper right of the curve with a large depth and small velocity, the specific head decreases with decrease in depth, reaching a minimum value at depth d_c known as **critical depth**, sometimes called the depth of the minimum energy content. Further decrease in depth results in rapid increase in specific head. For any value of specific head, except that corresponding to **critical depth**, there are alternate depths at which the flow could occur. These alternate depths are sometimes referred to as **equal energy depths**.

When the flow occurs at depths greater than critical depth (velocity less than critical), the flow is called sub-critical or tranquil. When the flow occurs at depths less than critical depth (velocities greater than critical), the flow is called supercritical, rapid, or shooting. The change from supercritical to subcritical flow is often very abrupt, resulting in the phenomenon known as hydraulic jump. Flow at the critical depth is called **critical flow** and the velocity at critical depth is the **critical velocity**. The channel slope which produces critical depth and critical velocity for given discharge is the **critical slope**.

Critical depth for a particular discharge is dependent on channel slopes and roughness. Critical slope depends upon the channel roughness, the channel geometry, and the discharge. For a given critical depth and critical velocity, the critical slope for a particular roughness can be computed by the **Manning Equation**.

Supercritical flow is difficult to control because abrupt changes in alignment or in cross-section produce waves that travel downstream, alternating from side to side, and sometimes cause the water to overtop the channel sides. Changes in channel shape, slope, or roughness cannot be reflected upstream except for very short distances (upstream control). Supercritical flow is common in steep flumes and in mountain streams. Pulsating flow can occur at depths as great as 8 feet.

Subcritical flow is relatively easy to control. Changes in channel shape, slope, and roughness affect the flow for some distances upstream (downstream control). Subcritical flow is characteristic of the streams located in the plains and valleys, regions where stream slopes are relatively flat.

Critical depth is important in hydraulic analyses because it is always a hydraulic control. The flow must pass through critical depth in going from one type of flow to the other. Typical locations of critical depth are:

(1) at abrupt changes in slope when a flat (subcritical) slope is sharply increased to a steep (supercritical) slope;

(2) at a channel constriction such as a culvert entrance under some conditions;

(3) at the unsubmerged outlet of a culvert or flume on a subcritical slope, with discharge into a wide channel or with a free fall at the outlet; and

(4) at the crest of an overflow dam or weir.

Distinguishing between the different types is important in channel design, thus the location of critical depth and the determination of critical slope for a cross-section of given shape, size, and roughness becomes necessary. When flow occurs at critical depth:

$$\frac{A^2}{T} = \frac{Q^2}{g} . \tag{19.15}$$

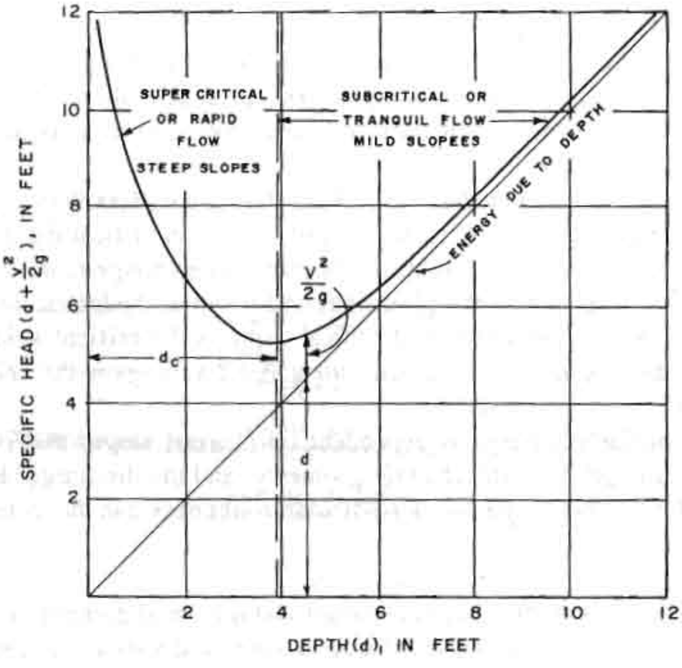


Fig. 19.6 Specific head diagram for constant Q

Critical depth (d_c) can be found from the design charts or computed for various channel cross-sections by the following equations:

Rectangular sections

$$d_c = 0.315 \sqrt[3]{(\frac{Q}{B})^2} \tag{19.16}$$

Trapezoidal sections

$$d_c = \frac{4z\ H_o - 3b + \sqrt{16z^2\ H_o^2 + 16z\ H_o b + 9b^2}}{10z} \tag{19.17}$$

The tables in King's Handbook (1976) provide a much easier solution for critical depth than Equation 19.17.

Triangular sections

$$d_c = 0.574 \sqrt[5]{(\frac{Q}{z})^2} \tag{19.18}$$

Circular section, approximate solution:

$$d_c = 0.325 \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.083D \tag{19.19}$$

accurate only when d_c/D lies between 0.3 and 0.9

where,

A = area of cross-section of flow, in square feet,
B = the width of a rectangular channel, in feet,
b = bottom width of a trapezoidal channel, in feet,
D = diameter of a circular conduit, in feet,
g = acceleration of gravity, 32.2 feet per second^2,
T = top width of water surface, in feet,
V = mean velocity of flow, in feet per second, and
x = slope of sides of a channel (horizontal to vertical).

Problems in Non-uniform Flow

Problems in non-uniform flow include computing the water surface profile, design of channel transitions, and dissipation of energy of the flowing water. A case that must be considered in the design of chutes is discussed briefly. This is a case of a sudden change in channel grade from one less than critical to one greater than critical (Figure 19.7).

The depth of flow at Section 1 can be computed using the Manning Equation. The flow at Section 2 (which for practical purposes can be assumed to occur at the change in grade) passes through critical depth (d_c). If the channel grade downstream from Section 2 is equal to the critical slope, the flow will become uniform at a depth equal to d_c. However, when a drainage channel discharges into a chute, the chute grade is steeper than critical slope and the flow is uniform and accelerating. Section 2 becomes the control section for both the flow in the channel (downstream control) and the flow in the chute (upstream control).

Knowing the specific head (H_o) in the approach channel, the capacity of the chute entrance, such as Section 2 (Figure 19.7) for a rectangular channel can be computed by a weir formula.

$$Q = 3.09\ k_e\ B\ H_o^{3/2}\ . \tag{19.20}$$

For a trapezoidal channel the capacity can be computed by the formula:

$$Q = 8.03\ k_e\ (H_o - d_c)^{\frac{1}{2}}\ (d_c)\ (b + zd_c) \tag{19.21}$$

where,

d_c = critical depth at Section 2,

H_o = specific head at Section 1,

K_e = coefficient which represents the entrance loss - varies from 1.0 for perfect entrance of smooth curves and gradual transition to 0.82 for a rectangular shaped-structure with square corners, and

Z = slope of sides of a channel (horizontal to vertical).

The critical depth, d_c, is computed by Equations (19.16 to 19.20).

The usual problem is to determine the size of chute channel required to carry a given discharge. The bottom width at the chute entrance (Section 2, Figure 9.7) can be computed by Equation (19.22) for a rectangular channel,

$$B = \frac{0.324Q}{k_e\ H_o^{3/2}} \tag{19.22}$$

and approximately by Equation (19.23) for a trapezoidal channel

$$b = \frac{0.324Q}{k_e H_o^{3/2}} - 0.7zH_o \tag{19.23}$$

where,

the symbols are the same as those in Equations 19.20 and 19.17, from which Equations 19.18 and 19.19 were derived.

The flow through the chute must satisfy Equation 19.12. If the head loss (h_L) through the chute is solely from friction, it can be expressed in terms of the hydraulic properties at each end of the chute, the roughness coefficient (n), and the length of chute (L) or:

$$h_L = \frac{n^2}{4.41}\ [(\frac{V_1}{R_1^{2/3}})^2 + (\frac{V_2}{R_2^{2/3}})^2]\ L\ . \tag{19.24}$$

If the flow is accelerating ($h_L < LS_o$), the cross-section of a large chute can be gradually reduced in order to provide a more economical section.

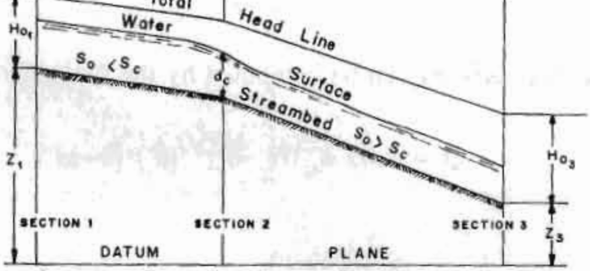


Fig. 19.7 Water-surface profile of a channel with sudden change in grade

The Froude Number

A useful parameter of flow in the Froude Number, one form of which is:

$$F = \frac{V}{\sqrt{gd_m}} = \frac{V}{V_c} \tag{19.25}$$

where,

d_m	=	mean depth of flow in feet-in the general expression any characteristic dimension of flow might be used,
g	=	acceleration of gravity = 32.2 (fps^2),
V	=	mean velocity in feet per second, and
V_c	=	critical velocity for the channel and discharge.

The Froude Number uniquely describes the flow pattern when gravity and inertia forces are the dominant factor in the flow. For example, in Figure 19.6 each point on the specific head curve has a single value of the Froude Number, although two values on the curve can be found for a particular value of specific head. The Froude number of critical flow is one; values greater than one indicate supercritical flow and values less than one indicate subcritical flow.

19.3.2 *Hydraulic Design of Culverts*

Flow through culverts occurs under two major conditions of flow (1) flow with inlet control (2) flow with outlet control. To avoid rigorous computations for determining the conditions of flow and headwater depths, the nomograph prepared by the U.S. Federal Highway Administration (Hydraulic Engineering Circular No.5) are used to calculate headwater depths for both inlet control and outlet control. Assuming a trial size culvert, the headwaters are calculated for both inlet and outlet control flow conditions. The condition at higher headwater is assumed to prevail. However, the headwater should not be greater than permitted by site and other design consideration. See 19.3.2 (c) for stepwise details for use of nomographs.

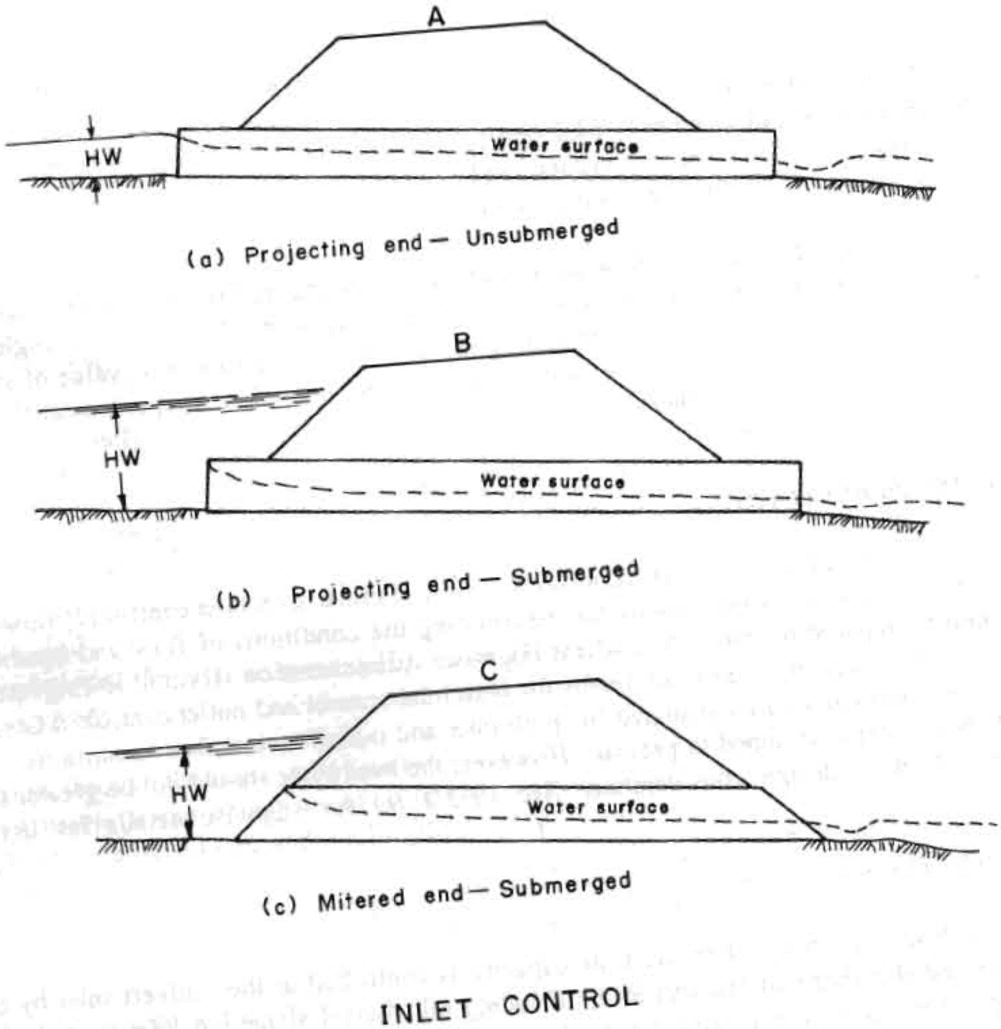
(a) *Flow with Inlet Control*

A culvert operates with inlet control when the flow capacity is controlled at the culvert inlet by the depth of headwater and the shape of the entrance, including the barrel shape. In inlet control, the roughness and length of the culvert and tailwater depth do not practically affect the culvert capacity.

Various inlet flow conditions are shown in Figures 19.8 (a) to 19.8 (c).

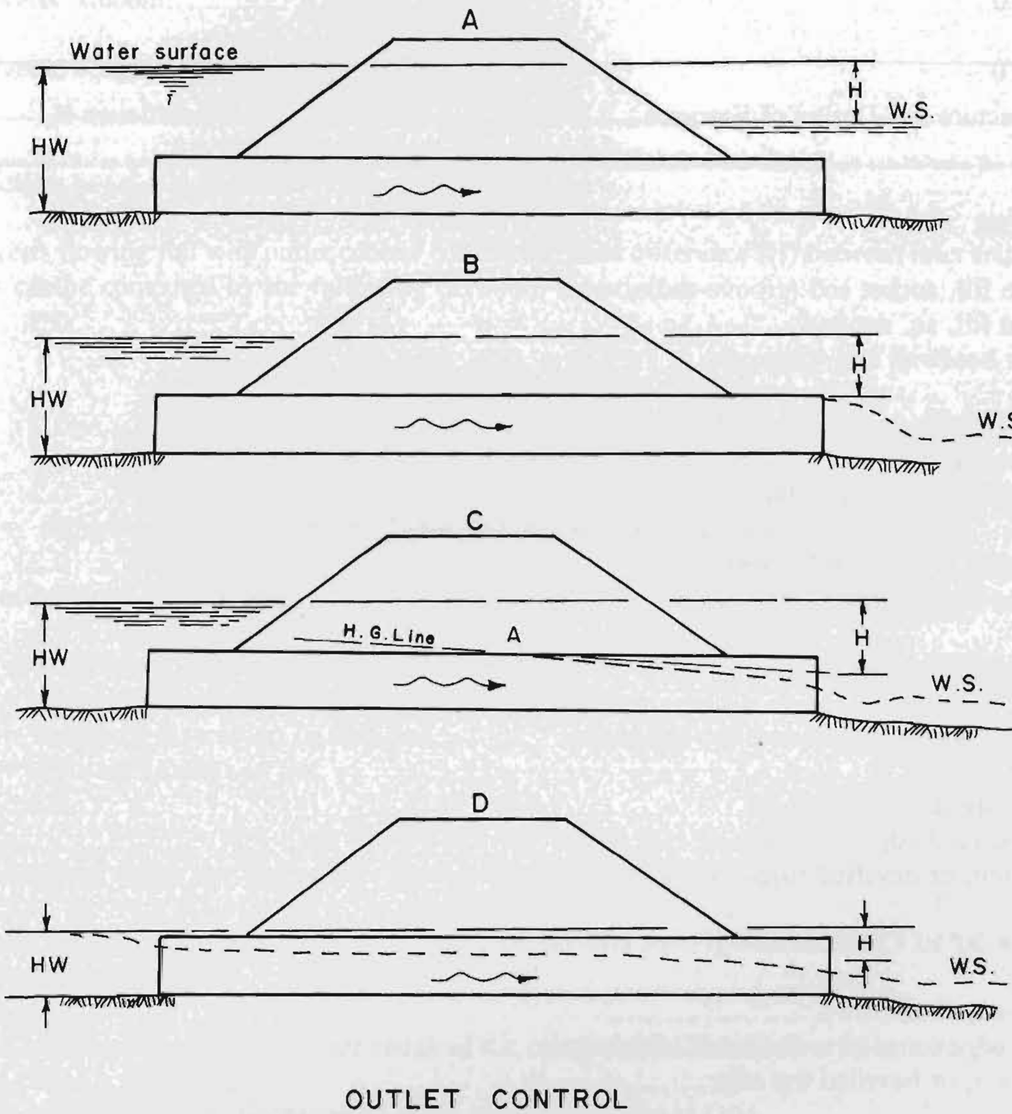
(b) *Flow with Outlet Control*

Flow in culverts flowing with outlet control can be full or partially full for part of the barrel length or for the full length. Various types of outlet control flows are shown in Figure 19.9(a-d). In a culvert operating with outlet control, flow capacity is affected by tail water elevation, the length, roughness, and the slope of the culvert in addition to inlet conditions. However, outlet conditions are the determining factors controlling flow capacity.



Source: U.S. Federal Highway Administration 1977

Fig. 19.8: Culverts with inlet control



Source : U.S. Federal Highway Administration 1977

Fig. 19.9: Culverts with outlet control

Table 19.8 Entrance loss coefficients

Outlet control, full or partly full

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

Type of Structure and Design of Entrance	Coefficient K_e
<u>Pipe, Concrete</u>	
Project from fill, socket end (groove-end)...	0.2
Project from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)...........	0.2
Square-edge................................	0.5
Rounded (radius = 1/12D)..................	0.2
Bevelled edges, 33.7° or 45° bevels...........	0.2
Side-or slope-tapered inlet..................	0.2
<u>Box, Reinforced Concrete</u>	
Headwall parallel to embankment (no wingfalls)	
Square-edged on 3 edges....................	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or bevelled edges on 3 sides.....	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown......................	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or bevelled top edge............	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown......................	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown......................	0.7
Side or slope-tapered inlet....................	0.2

Source: Transportation Research Board Commission on Sociotechnical Systems (TRBCSS) 1978

Table 19.9 Value of n for commonly used culvert materials

Concrete pipes................................	0.012
Box culverts, smooth..........................	0.012
Box culverts, rough, with sediment deposits...	0.016

The Equation 19.26 can be readily solved for H by the use of the nomograph for full flow. The nomograph can be used for other values on n also by modifying the culvert length.

For culverts flowing full with outlet control conditions, head difference (H) between inlet and outlet water surfaces can be computed by the following Bernoulli Equation:

$$H = [1 + K_e + \frac{29n^2L}{R^{1.33}}] \frac{V^2}{2g} \quad (19.26)$$

where,

H = difference of levels between headwater and tail water surfaces in feet,

$\frac{V^2}{2_g}$ = velocity head in feet,

K_e = coefficient of entrance loss (see Table 19.7),

n = Manning roughness coefficient (see Table 19.8),

L = length of the culvert in feet, and

R = hydraulic radius in feet.

(c) *Stepwise Detail for Use of the Nomograph to Design a Culvert*

Figures 19.10 and 19.11 illustrate flow through pipe culverts and Figures 19.12 to 19.15 are nomographs for determining culvert size.

1. The following design data are required for computation of the culvert size with the nomograph:

   a. design flood or discharge, Q_n of n years return period (cfs),

   b. length of the culvert, L (feet),

   c. slope of the culvert, S, (ft per ft),

   d. allowable headwater depth, HW, from culvert invert to the permissible water surface elevation at the entrance (feet), and

   e. mean and maximum velocities in natural drains.

2. Culvert type, including culvert material, shape, and entrance type, is selected. The approximate size of the culvert is determined by the equation:

$$\frac{Q}{10} = A \ .$$

Alternatively, the inlet control nomograph can be used assuming an approximate

$$\frac{HW}{D} = 1.5 \text{ using } Q_n.$$

3. For the trial size culvert, headwater depths are determined for inlet control and outlet control conditions of flow as described below.

<u>For Inlet Control Flow,</u>

a) Find HW/D for the trial size culvert from the nomograph and establish HW by multiplying obtained HW/D by trial size D.

b) If HW is more or less than permissible, another trial size is assumed until HW is acceptable for inlet control.

<u>For Outlet Control Flow</u>

a) Assume the depth of tailwater TW, in feet, above the invert at the outlet for the design flood in the outlet channel.

An approximate depth of flow in a natural stream at the outlet can be made by calculating with the Manning Equation, if the channel at the outlet has a uniform cross-section, slope, and roughness. However, most outlet channels are wider and steeper than the culvert; tailwater depth being less than critical depth does not affect the flow and therefore, channel depth calculations are not required.

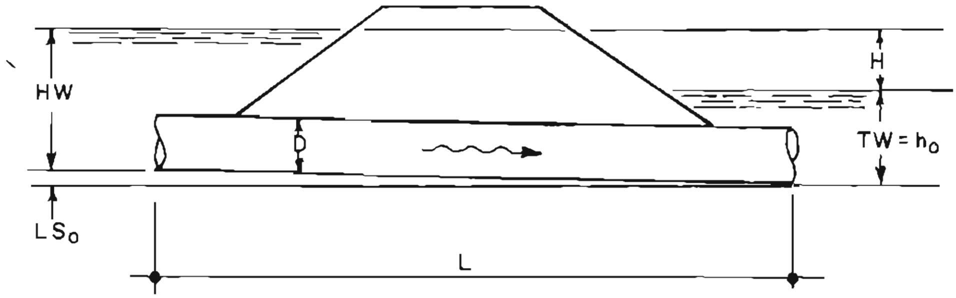
b) Establish headwater, HW, by the following equation:

$$HW = H + h_o - LS_o \tag{19.27}$$

where,

- H = headloss from the nomograph (Figures 19.14 and 19.15) for outlet control (ft),
- S_o = slope of culvert invert (ft per ft), and
- L = length of culvert (ft).

However, if the tailwater, TW, elevation is equal to or greater than the top of the culvert at the outlet; h_o = TW - (tailwater depth [feet]).



Source : U.S. Federal Highway Administration 1977

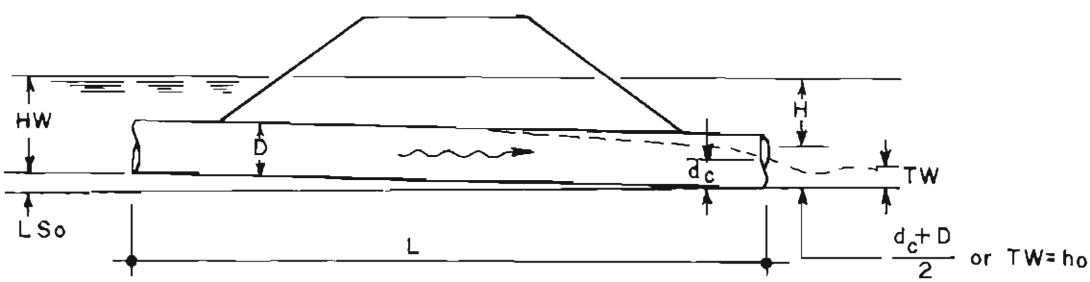
Fig. 19.10 Flow through pipe culvert

If tailwater elevation is lower than the crown of the culvert at the outlet:

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

where,

d_c = critical depth in feet, and
D = height of culvert in feet.



Source : U.S. Federal Highway Administration 1977

Fig. 19.11 Flow through pipe culvert

4. Compare the headwaters, HW, obtained for inlet control and outlet control flows. The higher headwater governs and indicates flow control for the selected size.

5. If outlet control governs and gives headwater, larger then permissible headwater, select a larger sized culvert. Only outlet control calculations need to revised, as inlet control has been satisfied already by the previous smaller sized culvert.

6. Compute outlet velocities for the finally selected culvert size, to determine the channel protection needs.

a) If outlet control governs, then outlet velocity $V = \frac{Qn}{A_o}$, where A_o is the area of flow in the culvert barrel at the outlet:

where,

A_o = the area corresponding to d_c or TW, if d_c or TW is lower than the culvert barrel top at the outlet. A_o is always less than the total cross-sectional area of the culvert barrel.

b) If inlet control governs, outlet velocity can be taken to be equal to mean velocity in open channel flow, computed with the help of the Manning Equation.

For n or K_e values different from those shown in the nomograph, the following procedure is applied.

For the n of the nomograph and a K_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the k_e values.

For a different roughness coefficient n_1 from that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula:

$$L_1 = L \left[\frac{n_1}{n}\right]^2 . \qquad (19.28)$$

c) Examples on the use of the nomograph (Figures 19.12 to Figure 19.15) for hydraulic design of culverts:

given,

- o 25 year return period flood, Q = 70 cu ft/sec,
- o stream-bed slope, S = 75 %,
- o culvert type = concrete pipe,
- o length of pipe = 30ft,
- o road formation width = 23ft,

to find:

- o culvert diameter
- o culvert slope

design steps:

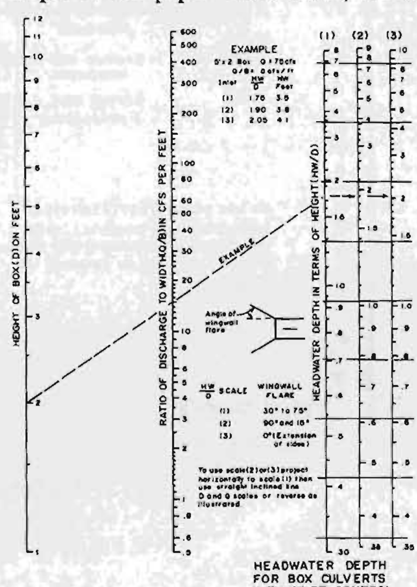
o assume slope of culvert	=	70%,
o assume size of culvert	=	3ft diameter, and
o assume allowable headwater	=	5ft.

<u>Checking for Inlet Control</u>

o Approximate size of pipe required:
= Q/10 = 70/10 = 7 sq ft. = one 3ft diameter pipe

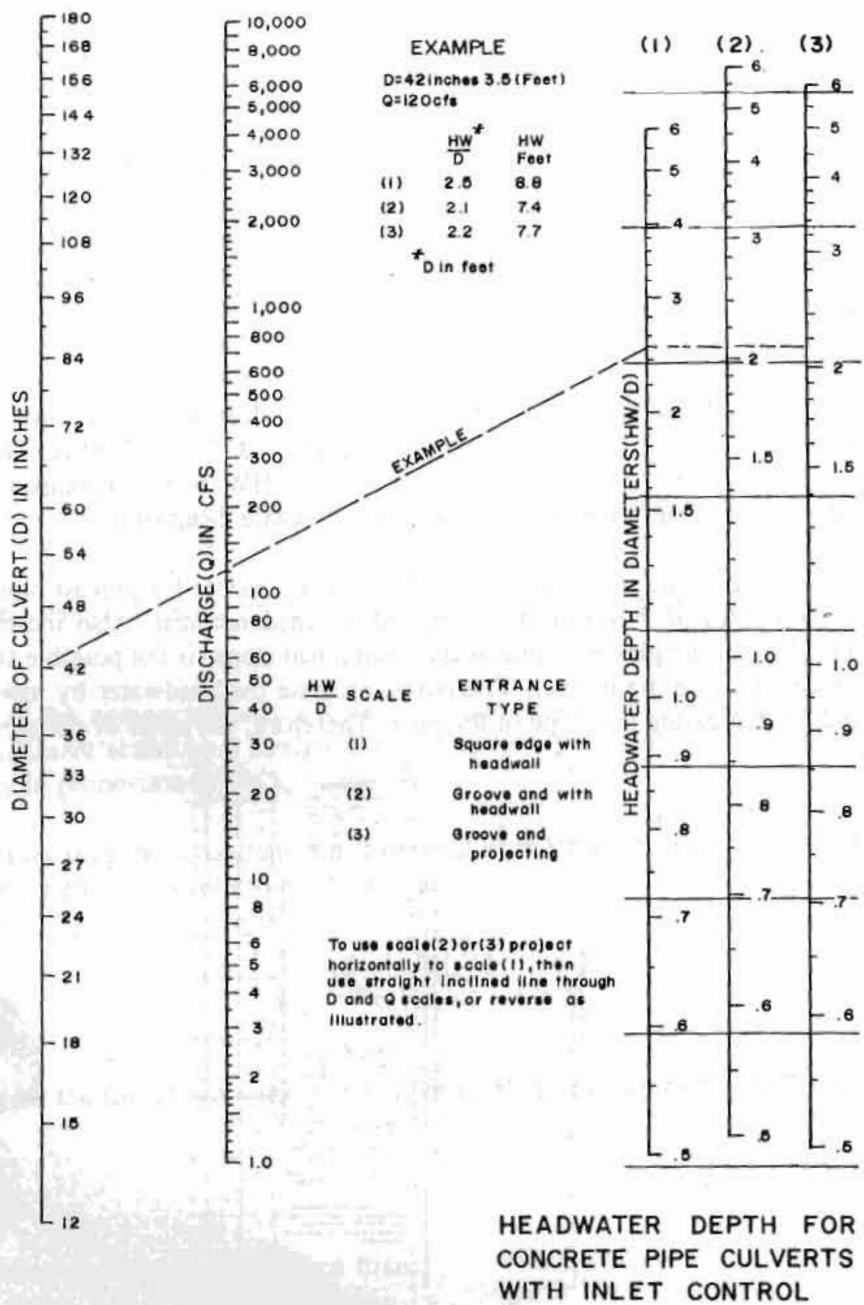
o Using HW/D = 1.5 and Q = 70, from Figure 19.13, diameter of the pipe = 40" > 3′. Try HW/D = 2, from Figure 19.13, diameter of the pipe = 36" = 3′ O.K. Therefore, HW = Dx2 = 3 x 2 = 6ft. For inlet control flow, since allowable HW = 5ft, increase the size of the pipe or increase the height of the parapet to increase the allowable headwater.

o Increasing the size of the slope by a few inches is not possible for precast pipes 3 feet in diameter. Providing two pipes of 3 feet in diameter will be uneconomical. Also increasing the headwater, keeping the slope of the pipe the same as the natural bed slope, is not possible since this will increase the road level and side drain level. Therefore, increase the headwater by lowering the inlet of the culvert and by decreasing the slope of the pipe. Therefore, the slope of the pipe = 70 - 4.5 = 65.5 per cent.



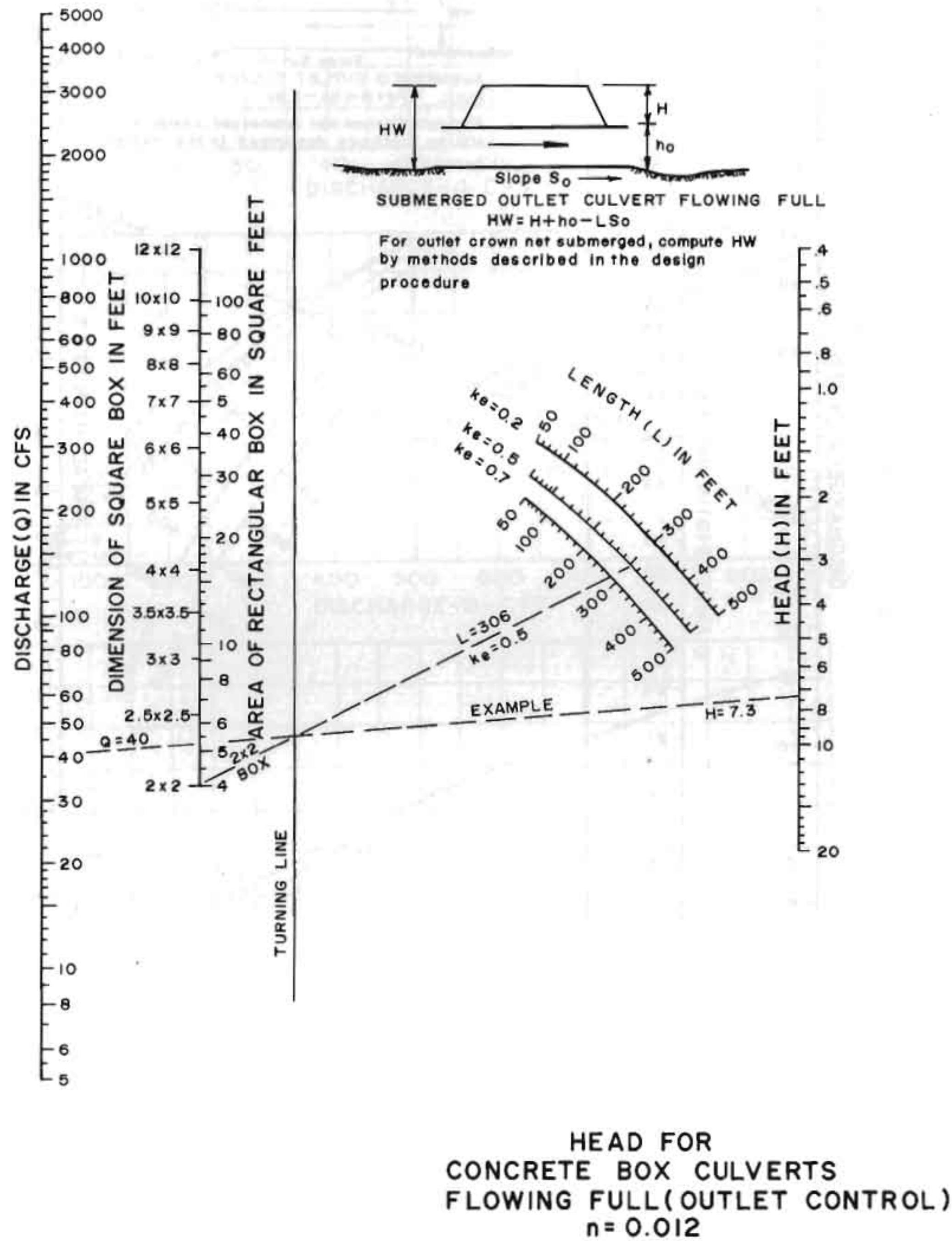
Source : U.S. Federal Highway Administration 1977

Fig. 19.12 Hydraulic chart



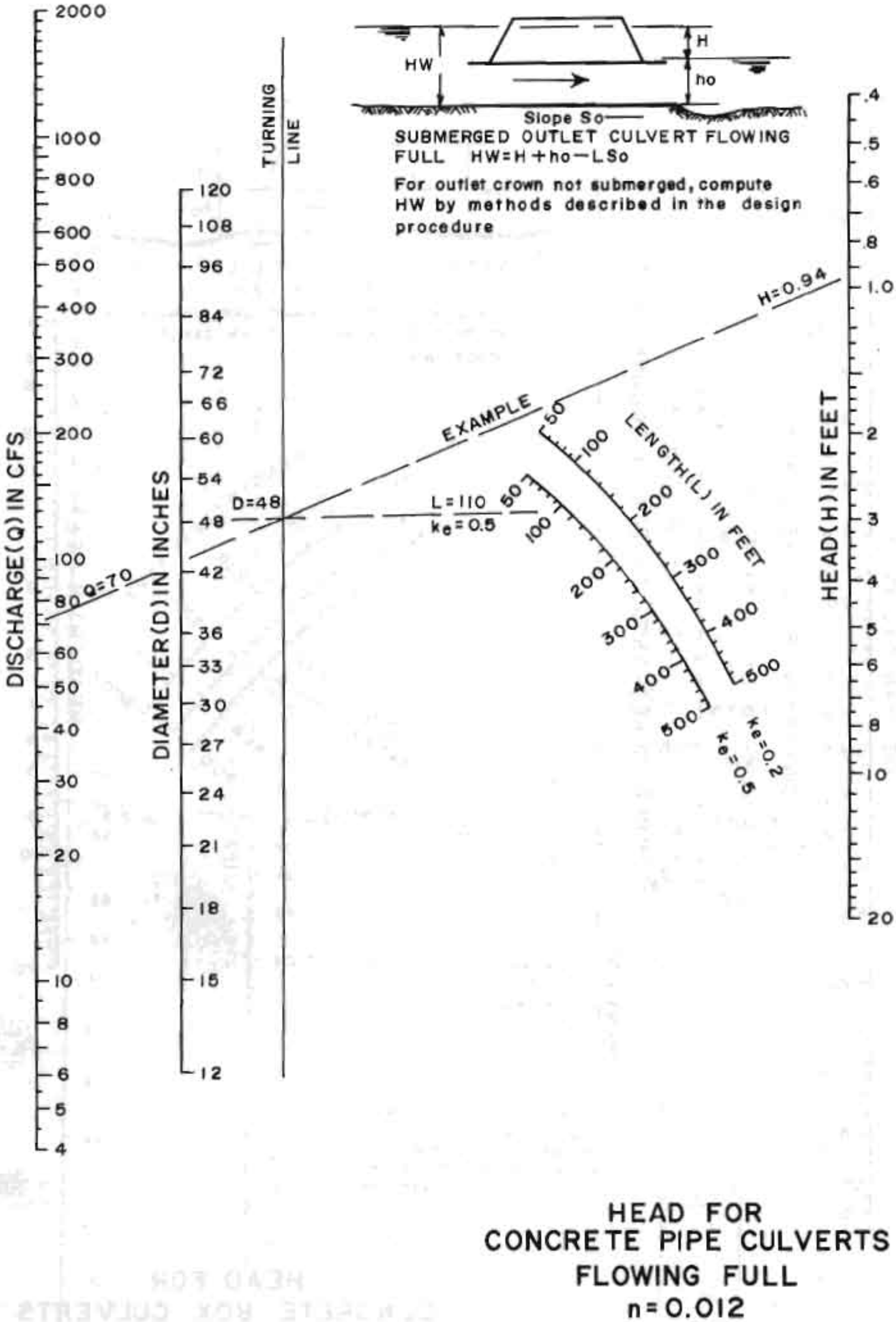
Source : U.S. Federal Highway Administration 1977

Fig. 19.13 Hydraulic chart



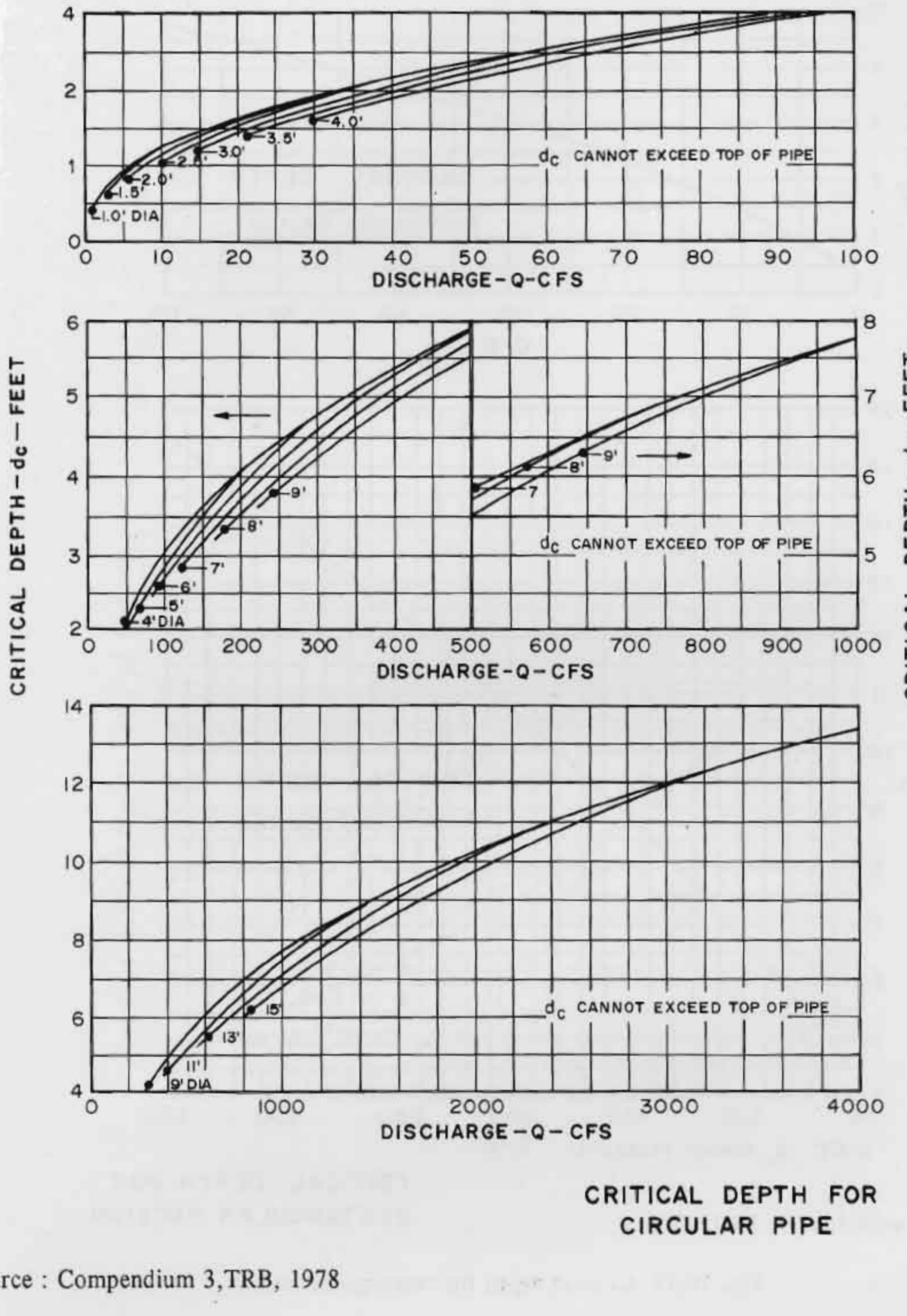
Source : U.S. Federal Highway Administration 1977

Fig. 19.14 Hydraulic chart



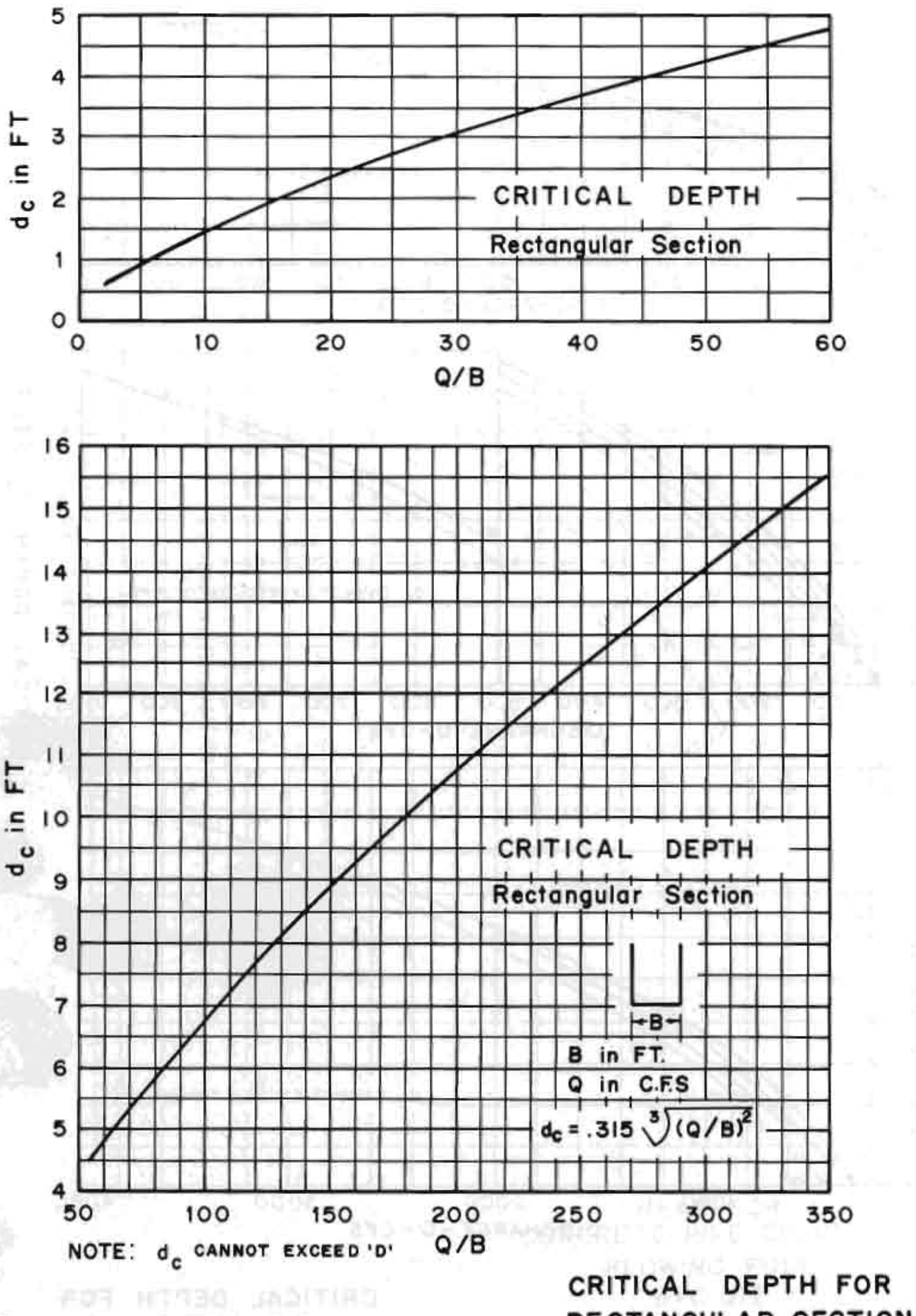
Source : U.S. Federal Highway Administration, 1977

Fig. 19.15 Hydraulic chart



Source : Compendium 3,TRB, 1978

Fig. 19.16 Critical depth for circular pipe



Source : Compendium 3, TRB, 1978

Fig. 19.17 Critical depth for rectangular section

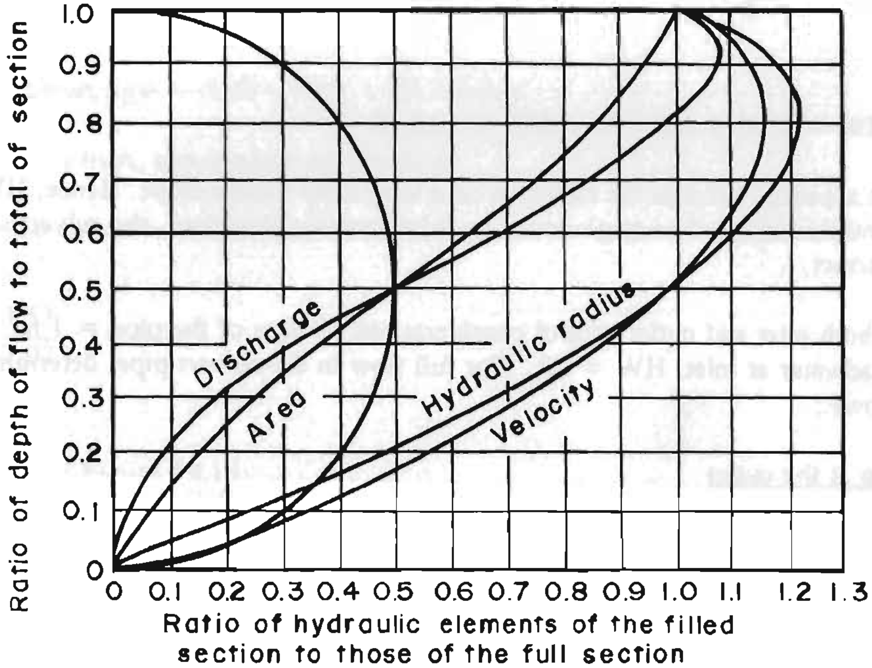


Fig. 19.18 Hydraulic elements of a circular pipe

Checking for Outlet Control

From the outlet control nomograph (Figure 19.14) head loss H = 2.6 ft for 50 ft culvert, and K_e = 0.5 from Table 19.7. Hence for a 30 ft culvert:

head loss H = 30/50 x 2.6 = 1.56 ft, and

headwater for outlet control HW = H + h_o - Ls_o.

Because of the steep slope, assume that the tailwater depth below the pipe top at the outlet is half the pipe diameter:

if TW = 1/2 D then h_o = d_c + D/2. (for d_c values, see Figures 19.16 and 19.17)

Find d_c from Figure 19.16:

$$h_o = \frac{2.6 + 3}{2} = 2.8 \text{ ft.}$$

Hence,

$$HW = 1.56 + 2.8 - 30 \times .655 = -15.29.$$

The negative sign appeared because the culvert is on a very steep torrent slope. Hence, HW inlet control > HW outlet control, and the flow is governed by inlet control. Therefore, the culvert size selected by inlet control is correct.

Therefore, from both inlet and outlet control considerations the size of the pipe = 1 no. 3ft dia.; slope = 65.5% and headwater at inlet, HW = 6ft. For full flow in the culvert pipe, determine velocity and discharge as follows :

Check for erosion at the outlet

velocity at full flow

$$V_F = \frac{1.49}{h} R^2/3\ S^1/3$$

$$= \frac{1.49}{.012} (\frac{3}{4})^2/3\ (.655)^1/2$$

$$= 124.16 \times 0.8247 \times 0.8093$$

$$= 82.8\ ft/\text{sec},$$

full area of culvert pipe, $A_F = \frac{\pi d^2}{4} = \frac{\pi\ 3^2}{4} = 7.07 sq.ft,$

full flow,

$Q_F = V.A = 82.8 \times 7.07 = 585$ cfs,

$$\frac{Q \text{ of culvert flow}}{QF} = 70/585 = 0.1197,$$

from Figure 19.18

$d/df = 0.22$, and

depth of flow in culvert pipe for design flow, $d = 0.22 \times 3'\,0" =$ **0.66′** .

From Figure 19.18,

$V/V_f = 0.6$

hence,

velocity in the culvert pipe = $0.60 \times 82.8 = 49.7$ ft/sec.

The velocity is very high, hence adopt milder slope.

Now, limiting the culvert velocity to 20ft/sec for $Q = 70$ cfs, required area = 3.53 sq ft

if,

$$A/A_f = \frac{3.53}{7.07} = 0.5,$$

then,

$d/df = 0.5$ (see Figure 19.18)

and for $d/d_f = 0.5$; $R/R_f = 1.0$.

Therefore,

$R = 1.0 \times 3/4 = 0.75$ ft,

therefore,

$P = A/R = 3.53/0.75 = 4.71$ ft.

now,

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$S = \sqrt{\frac{V.n}{1.49\ R^{2/3}}} = \sqrt{\frac{20 \times 0.012}{1.49 \times 0.75^{2/3}}} = 0.4419,$$

therefore, adopt a culvert pipe slope of 44 per cent.

The inlet control flow is not affected by change in slope. The change in outlet control L_{80} value from the original 19.65 to 13.2 after the change in slope also does not affect the governing inlet flow control. Because of the pipe outlet above the gully bed and the milder slope of pipe, scour protection would be needed at the outlet. The protection measures can be based on the following calculations.

The energy of water will be dissipated by the water falling freely out of the culvert pipe into a water pool lined with at least d_{50} size riprap obtained in the following calculations. Calculate maximum scour depth and riprap gradation required by the following formulas for cohesionless soil:

1) $$\frac{D_{sm}}{D_o} = 0.80 \left(\frac{Q}{D_o^{5/2}}\right)^{0.375} t^{0.10}$$

$$D_{sm} = 0.80 \left(\frac{70}{3^{5/2}}\right)^{0.375} (60)^{0.10} \times 3$$
$$= 6.34\ ft$$

where,

D_{sm}	=	maximum scour depth in ft,
D_o	=	diameter of culvert in ft,
Q	=	flow in cfs, and
t	=	duration of flow in minutes.

The x,y coordinates of the flow trajectory can be approximately calculated as follows :

$$X = V\sqrt{\frac{2y}{g}}$$

where,

V	=	velocity at pipe outlet (ft/sec),
x,y	=	trajectory coordinates (ft), and
g	=	32.2 ft/sec^2.

x (ft)	8.5	12	15.7	19
y (ft)	3	6	10	15

For half full flow at outlet and elevation difference of about 6 ft between the top of the flow at the outlet and the ground level below the outlet, the distance (x) requiring protection is 12 ft.

The protection measures can be gabion or masonry cascade or a heap of flow stable, large boulders.

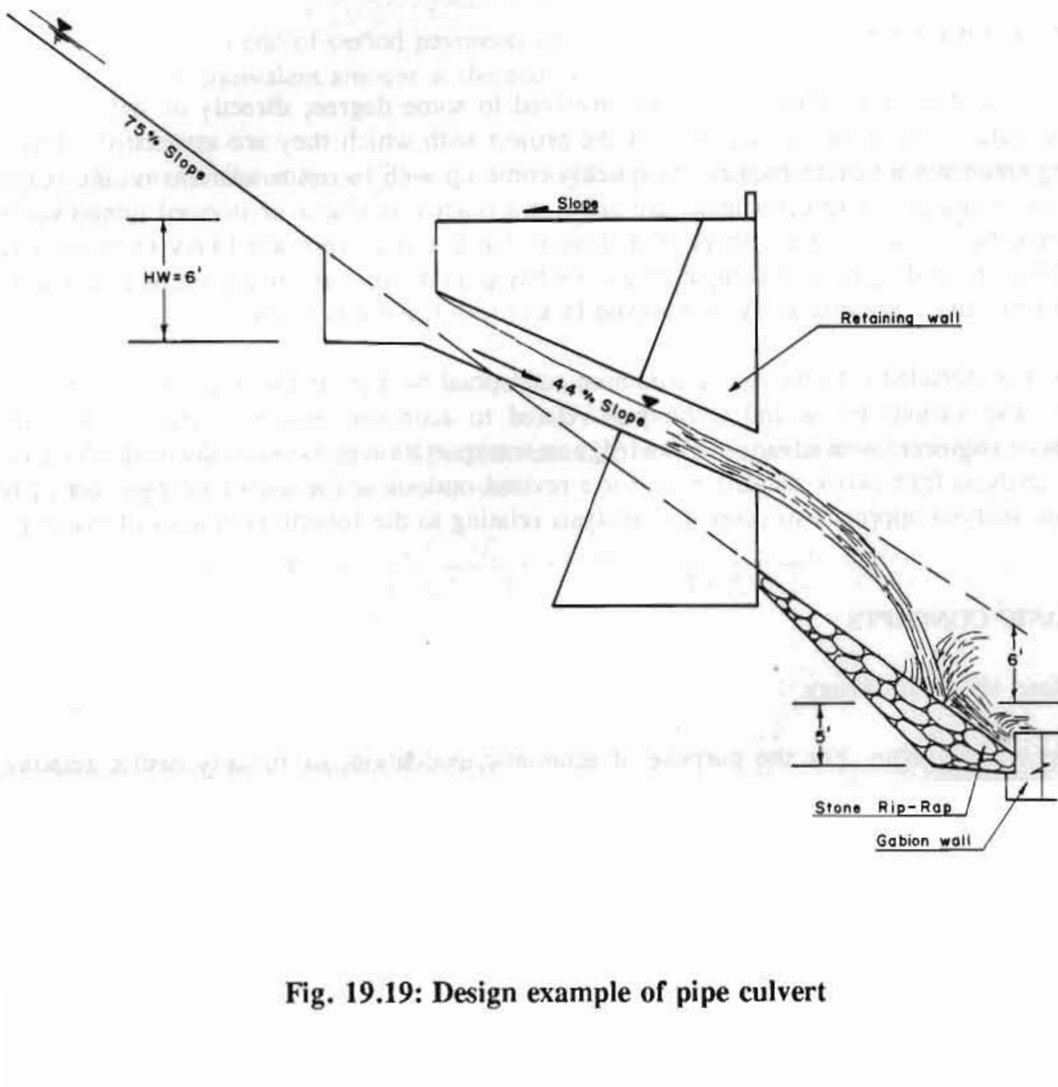


Fig. 19.19: Design example of pipe culvert