625-734
KAS
1984

06993

NATIONAL TRANSPORT RESEARCH CENTRE

MANUAL
OF
RURAL ROADS DRAINAGE

NTRC-D.S-6

M. KASHIF

MAY, 1984

CONTENTS

Books and Manuals donated to National Transport Research Centre Library, Islamabad, Pakistan.

(1) Manuals - Corps of Engineers
Backwater Curves in River Channels

(2) Design of Stable Channels with Flexible Linings
U.S. Department of Transportation

(3) Hydraulic of Bridge Waterways U.S. Department of Transportation

(4) Debris - Control Structures
U.S. Department of Transportation

(5) Hydraulic Design of Energy Dissipators for Culverts and Channels U.S. Department of Transportation

(6) Hydraulic Design of Improved Inlets for Culverts

(7) Urban Hydrology for Small Watershed TR 55 SOIL Conservation Service U.S. Department of Agriculture.

(8) Capacity Chart for the Hydraulic Design of Highway Culverts.

c.c. National Talent Pool of Pakistan,
House No. 5, Street No. 19, F-7/2, Islamabad.
Pakistan.

Research and Drainage Studies are needed in the following areas listed below :

(1) To develop Rainfall-Intensity-Duration-Frequency Curves for all regions of Pakistan

(2) Small Watersheds study located in different regions of Pakistan to determine Time of Concentration

(3) Velocity and Erosion Study for bare earth channels, to determine the value of Manning 'n'

(4) Study of Commercially available drainage pipes in Pakistan, to determine Manning 'n'

(5) Structural adequacy of commercially available drainage pipes in Pakistan

(6) Methods and procedures for Maintenance of Highway Channels

(7) Economics of Drainage Channels

(8) Construction Methods and Procedures for satisfactory Highway Drainage Structures

(9) Study of Materials available in Pakistan for Channel Protection

(10) Training programme for Drainage Engineers

SECTION - I

INTRODUCTION

1.1 PURPOSE AND SCOPE

Providing adequate drainage is essential to the highway. Economical drainage design is achieved through doing an adequate job at the lowest cost.

The lowest cost and adequate drainage maintains proper balance between first cost, flood damage and maintenance cost.

This manual presents discussion of the theoretical principals, the basic data and procedures for estimating run-off, determining the size of roadside drainage channels and Highway Culverts.

This manual is prepared primarily for guidance and instructions of engineers actively engaged in Highway design. The design procedure discussed in this manual are only tools to aid in solving the surface drainage problem. The drainage problems of each section of highway is individual and for solution requires adequate field data and an engineer experienced in highway drainage.

1.2 HYDROLOGY AND RUNOFF

1.2.1 Hydrology and Hydrologic Cycle

a. Defination: The science that deals with the processes governing the depletion and replenishment of the water resources of the land areas of the earth.

b. Objective of Engineering Hydrology to predict the amount of water and/or the rate of flow of water which will be found at any given time and location (with a certain probability)

1.2.2 Hydrologic Cycle

a. Basic equation of hydrology is the water balance or mass balance.

$$P(t) - G(t) - E(t) - T(t) = \Delta S(t)$$

Where P = precipitation
R = runoff measured in the streams
G = ground water flow out of the watershed
E = evaporation from watershed
ΔS = storage in soil moisture, groundwater, lakes swamps etc.

All terms are function of time and are averaged spetially over the water shed area.

b. During a short storm event of duration t, E and T can be neglected G is assumed to be part of R Hence

$$1/t \int_0^t P(t)dt - \frac{1}{t} \int_0^t R(t)dt = \frac{1}{t} \int_0^t S(t)dt$$

or simply $P-R = \Delta S$

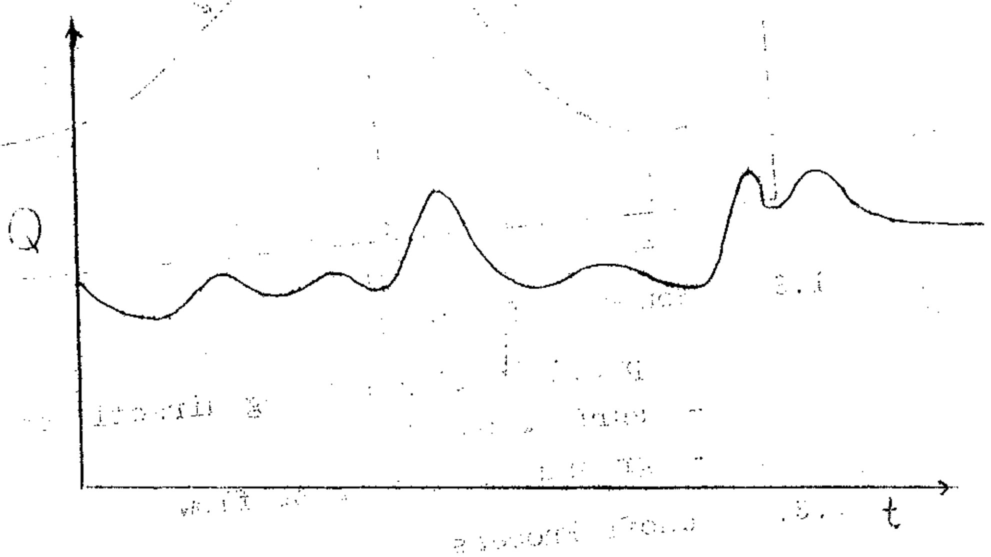
where each term is the total volume during t time. The term S is referred to as the 'losses'

c. Over an annual cycle, the change in storage can often be assumed negligible. With the inclusion of G in R the annual evapatranspiration is :

$$ET = P - R$$

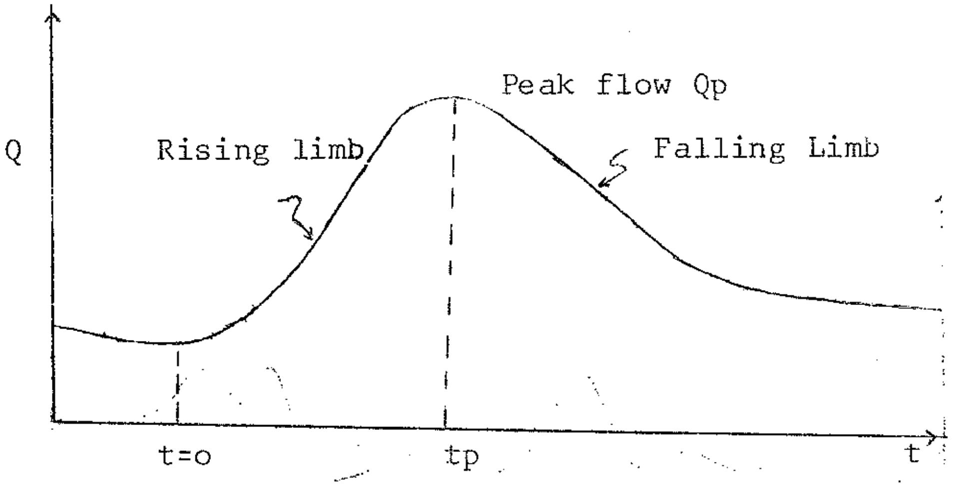
1.3 RUNOFF

1.3.1 The hydrograph : a graphical representation of flow at a channel cross-section as function of time



Q = cfs
cfs/Mi^2
acre-ft/day
inches/day
1 acre-ft. = 13560 ft^3
1 cfs $\approx$ 2 acre-ft/day

1.3.2 Storm hydrograph : The hydrograph due to an isolated storm event.

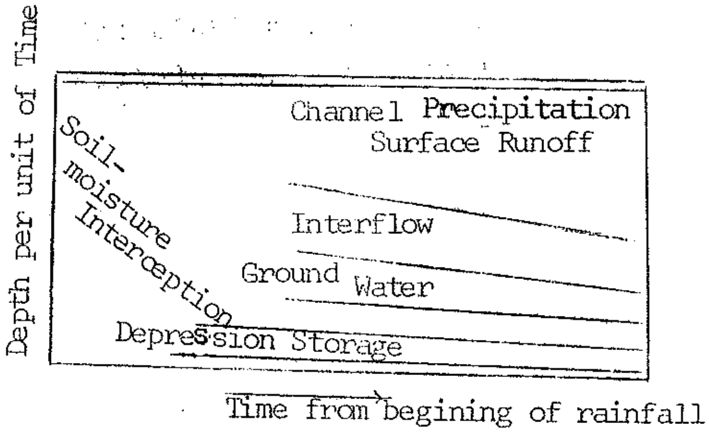


1.3.3 Sources of runoff

- precipitation falling directly on stream
- surface runoff
- ground water or base flow

1.3.4 Runoff Process

When rainfall event begins water is 'lost' to interception, depression, and soil moisture storage. Eventually, water appears in the stream channel from ground water and surface runoff. This diagram shows a time history of allocation of precipitation to various storages and to runoff.



1.4 DEFINITIONS

1.4.1 Interception : moisture held by trees, shrubs, grass

Buildings : Never reaches ground

Depression Storage : moisture held in innumerable small' and large depression. It evaporates or infiltrates but never becomes surface runoff.

1.4.2 Infiltration : moisture passing the air-soil interface the maximum rate of infiltration is the 'infiltration capacity'.

soil moisture : Water held by capillary action in surface layers of soil maximum capacity is field capacity. Field moisture deficit (FMD) is a quantity of water necessary to bring soil to field capacity.

1.4.3 Interflow : Poorly defined. Generaly, water reaching the stream channel from non-saturated soil layers. Shorter travel time than ground water

1.5 STORM RUNOFF

The storm runoff which must be carried by the roadside drainage channels and culverts is the residual of the precipitation after losses. The rate of water loss depends upon the amount of the precipitation and the rate at which it falls (intensity), upon temperature, and upon the characteristics of the land surface. Not only does the rate of runoff vary with permeability of the land surface and the vegetal cover, but it varies from time to time for the same surface depending upon the antecedent conditions. It would be inpracticable, even if the data were available, to determine for each channel drainage area the frequency of recurrence of the numerous factors which effect the rainfall-runoff relation and thus compute the magnitude of the run-off for a given frequency rainfall.

1.6 FACTORS EFFECTING STORM RUNOFF

a. Types of precipitation, rain or snow
b. Rainfall intensity
c. Rainfall duration
d. Distribution of rainfall on basin
e. Storm direction
f. Antecendent precipitation and soil moisture
g. Land Use
h. Soil type
i. Basin Shape
j. Slope of channels
k. Slope of Valley Walls
l. Drainage Density
m. Artificial storage

SECTION - II

FREQUENCY ANALYSIS RISK DESIGN STORM

2.1 BASIS FOR DESIGN

First step in estimating design flood is choice of design frequency. This choice is based on :

a. Economic analysis
   - risk of damage due to surcharging
   - cost of reducing surcharge damage

b. Legal responsibilities

c. Risk to human life

d. Intangibles and uncertainties such as
   - inconvenience Caused by exceedance of design
   - future development

e. other factors affecting choice are :

   Ability to provide future relief for damage

2.2 DESIGN STORM - DESIGN FLOOD

2.2.1 Design frequency leads to design flood on a stream with a gaging station

2.2.2 Design frequency leads to a design storm where rainfall data exists

2.2.3 A runoff estimation technique transforms a design storm to a design flood

2.2.4 A runoff estimation technique transforms a design storm to a design flood.

CHOICE OF

Design Frequency

↓

Design Storm

↓

Design Flood

↓

Hydraulic Design

2.3 FREQUENCY ANALYSIS

Probability

2.3.1 Definition

$$P_i = \lim_{N \to \infty} \frac{n}{N}$$

Where P = probability of getting an outcome of kind i in any trial

n_i = number of outcomes of kind i

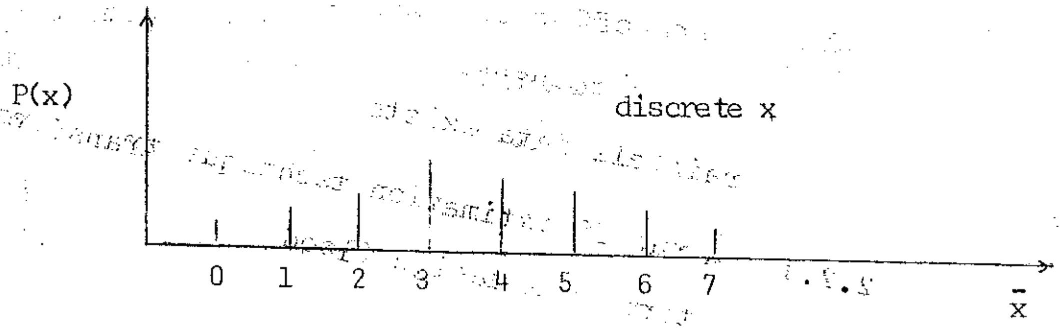
N = total number of trials

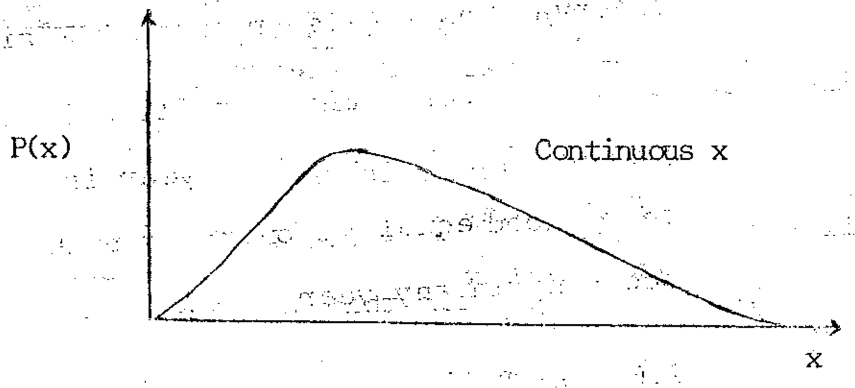
Note : $\sum P_i = 1$

2.3.2 Probability distribution

a. PDF - probability density function

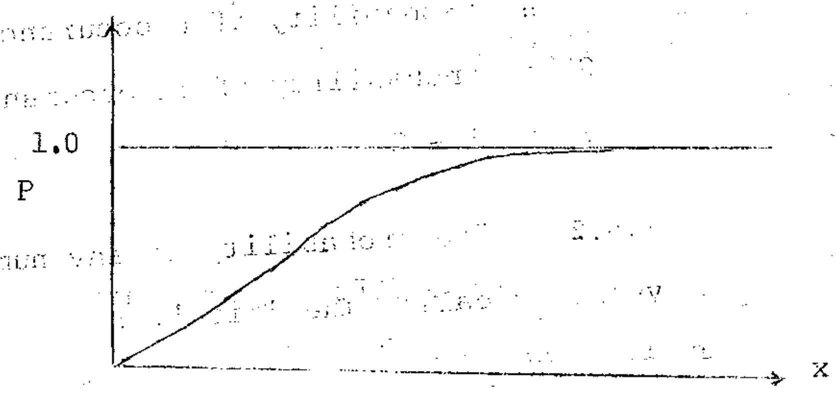
b. A histrogram of frequencies





2.4 CDF - CUMULATIVE DISTRIBUTION FUNCTION

$$P\ (x \leq s_0) = \int_{-\infty}^{x_0} P(x) \quad dx$$



2.5 RETURN PERIOD OR RECURRANVE IN-TERVAL

An alternative way to express a non-exceedance or exceedance probability

$$T = \frac{1}{1-P} = \frac{1}{G}$$

The return period is the average number of observations equals or exceeds a given value of x

e.g. a ten year flood is one whose non-exceedance probability in any given year is 0.90. The probability of a flood equal to or greater than this magnitude occuring in any year is 0.10.

2.6 RISK

2.6.1 The binomical probability Law of Bernoulli trials.

$$P(r) = \frac{n!}{r!\ (n-r)!} \quad G^{r} \quad p^{n-r}$$

= probability of r occurances in a trials where

G = probability of an occurance in any one trial and

P = 1 - G

2.6.2 The probability of any number occurances in n years is called the "risk", R

$$R = 1 - (0)$$

$$= (1) + (2) + (3) + \ldots\ldots\ldots\ldots\ldots$$

$$= 1 - p^{n}$$

2.6.3. TABLE: 11-6 IN LINSLEY, KOHLER, AND PAULHUS:

Return period required for a given risk and given project life.

	LIFE OF PROJECT		
RISK	10 YEARS	25 YEARS	100 YEARS
0.01	910	2440	9100
0.1	95	238	940
0.5	15	37	145
0.99	2.7	6	22

Example :

If we are willing to bear a 10% risk of failure over 50 years project life. What return period event must we design for ?

$$R = 1 - P^{50}$$

$$0.1 = 1 - P^{50}$$

$$P^{50} = 0.90$$

$$P = (0.99)^{0.02} = 0.997895$$

$$T = \frac{1}{1-P} = 475 \text{ Years}$$

SECTION - III

ROAD - SIDE DRAINAGE CHANNEL

3.1 Roadside Drainage Channels Perform the vital function of diverting or removing surface water from the highway right-of-way.

They should provide the most efficient disposal system consistent with cost, importance of road and economy of maintenance. One standard channel will rarely provide the most satisfactory drainage for all sections of highway, although it might be adequate for most locations.

This manual discusses flow in roadside drainage channels, estimation of peak discharges from small areas, prevention of channel erosion, and present methods for the design of drainage channels required to remove runoff from the area immediately adjacent to the highway.

3.2 FACTORS IN DESIGN

The primary purpose of roadside drainage channels is to prevent surface runoff from reaching the roadway and to efficiently remove the rainfall or surface water that reaches the roadway. To achieve this purpose, the drainage channels should have adequate capacity for the peak rates of runoff that recur with a frequency depending upon the class of road and the risk involved. On less important roads where little damage would result from overtopping of the drainage channel and where traffic would suffer only minor inconvenience, a peak runoff that recurs frequently might be satisfactory. On major highways or on minor highways where serious erosion damage would result from overtopping the drainage channels, a less frequent peak runoff might be used as the design runoff.

It is recommended that all drainage facilities other than culverts and bridges be designed to keep the travelleways useable during storms at least as great as that for 10-year frequency.

CAPACITY OF CHANNEL

3.3 The capacity of a drainage channel carrying uniform flow depends upon its shape, size, slope and roughness. For a given channel, the capacity becomes greater when the grade or depth of flow is increased. The channel capacity decreases as the channel surface becomes rougher. For example the stone gutter or channel of the same size, shape and slope has only half the capacity of the concrete gutter of channel.

Some times rough channels have advantage on steep slope to keep the velocities from becoming too high.

The most efficient shape of channel is that of semicircle, but hydaulic efficiency is not the sole criterion.

In addition to performing its hydraulic function, the drainage channel should be economical to construct, and require little maintenance during the life of the roadway.

3.4 Highway drainage channels may be classified according to function as: gutters, chutes, roadway channels, toe-of-slope channels, intercepting channels, median swales, and channel changes.

3.4.1 Gutter: Gutters are the channels at the edges of the pavement the shoulder formed by a curb or by a shallow depression. Gutters are invariably paved with concrete, brick, stone blocks, or some other structural material.

Gutters are generally used in lieu of other type channels for urban highway drainage and are sometimes used in rural areas, particularly in the areas of poor soil stability and for special drainage problems such as traffic interchanges and underpasses.

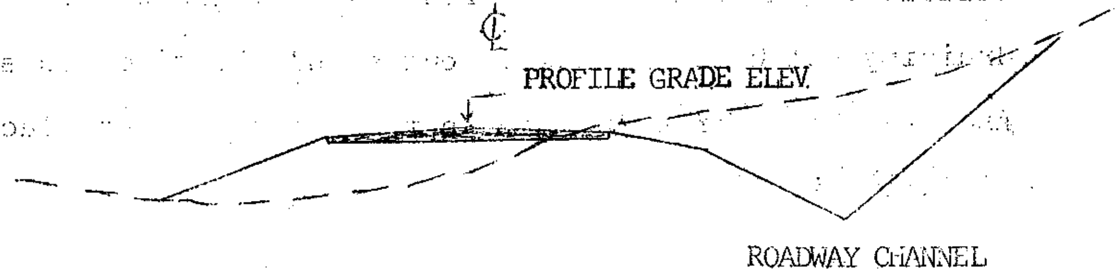
In area where vegetation cover cannot be used to prevent erosion damage to high fills, shoulder should be designed to serve as a gutter with a curb constructed at the outer edge to confine water to the shoulder.

The water collected in the gutter may be discharged down the slope through the chutes. The curb may be made of earth, bitiminous material, portland cement, concrete, or cut stone.

3.4.2 Chutes: Chutes use in this manual, are steeply inclined open or closed channels, which convey the collected water to a lower level. The chutes may be used to convey water down cut or fill slopes.

Open chutes can be metalled or be paved with portland cement concrete, bitiminous material, stone, or sod, depending upon the volume and velocity of the water to be remained. On long slopes, closed (pipe) chutes are generaly preferable to open chutes because in an open chute the high velocity water is likely to jump out of the channels, erode the slope, and destroy the chute.

3.4.3. Roadway Channels: Roadway Channels are the channels provided in the cut section to remove the runoff from rainfalling on the roadway and an the cut slopes.



These channels are sometimes called gutters when paved.

A well-designed roadway channel removes storm water from the cut areas with the lowest overall cost, including cost of maintenance, and with the least hazard to the traffic.

The channel should also be pleasing in appearance. The roadway channels built in earth should have side slopes 4 horizontal to 1 vertical or flatter where terain permits.

The depth of channel should be sufficient to remove the water without saturating the pavement sub-grade.

3.4.4. Toe-of-slope Channels : Toe-of-slope channels are located at or near the toe of a fill when it is necessary to convey water collected by the roadway channel to the point of disposal. On the down hill side of the highway, this channel can often be laid on a mild slope and the lower end flared to spread the water over the hillside. In arid and semiarid regions, the water draining out of the roadway cut should be diverted away from the fill for enough so that it does not come back to the highway.

3.4.5 Intercepting Channels: Intercepting channels are located on the natural ground near the top edge of a cut slope or along the edge of the right-of-way, to intercept the runoff from a hillside before it reaches the roadway. Intercepting the surface flow reduces erosion of cut slopes, lessens silt deposition and infiltration in the roadbed area, and decreases the likelihood of flooding the highway in severe storms.

Intercepting surface water is particularly important in arid and semiarid regions. Intercepting dikes may be built well back from the top of the cut slope and generally on a flat grade until water can be spread or emptied into a natural watercourse.

An intercepting channel constructed by furnishing a dike with borrow material is superior to an excavated channel because the later destroys the natural ground cover and is more likely to erode.

Care should be taken to avoid ponding water at the tops of slopes subject to sliding. In slide areas, storm water should be intercepted and removed as rapidly as practicable and section of the crossing highly permeable soil might require lining with impermeable material.

3.4.6 Median Swales: Median swales are the shallow depressed areas at or near the centre of medians used to drain the median areas and portion of the roadway.

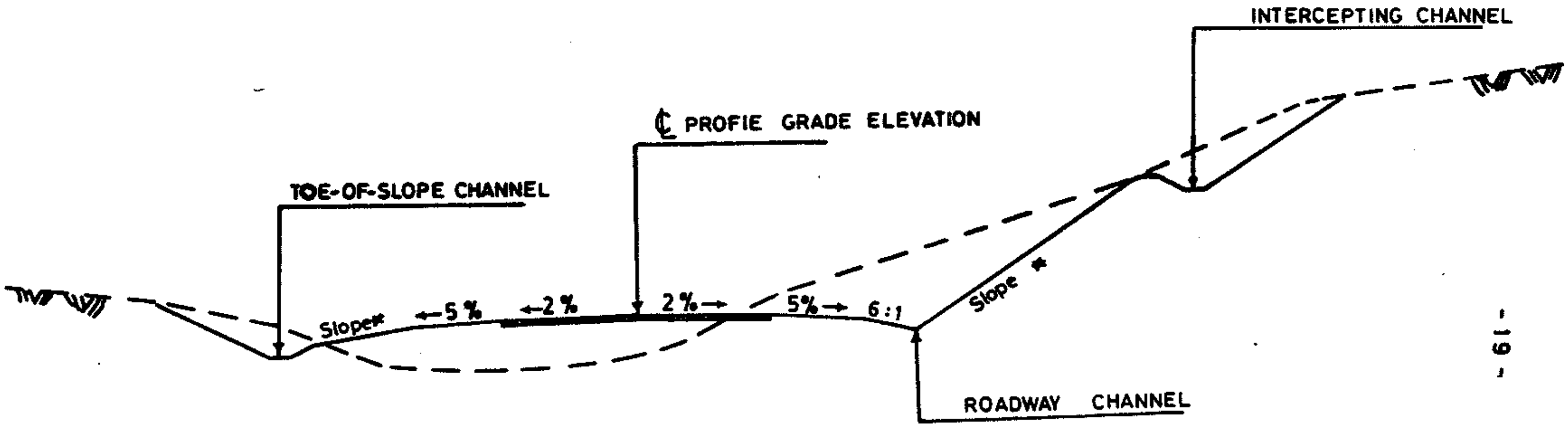
The depressed area or swale is sloped longnitudinally for drainage, and at intervals the water is intercepted by inlets and discharged from the roadway.

3.5 CHANNEL CHANGES

Channel changes alter the alignment or cross-section of natural watercourses. Replacing a long channel by a shorter improved channel will increase the channel slope and usually decrease the channel roughness. Both of these changes cause an increase in the velocity of the flowing water, sometimes enough to cause damage to the highway embankment near the stream or excessive scour around the footing of the structures. Alignment and Grade; A sharp change in channel alignment presents a point of attack for flowing water, and abrupt changes in grade cause deposition of transported material when the grade is flattened or scour when grade is steepened therefore abrupt sharp changes in alignment or grade should be avoided.

A drainage channel should have a grade that produces velocities that neither erode nor cause deposition in the channel.

Special attention is required, when drainage channel discharges into natural streams. The alignment of the drainage channel should not cause eddies, which causes scoure in the natural stream and if the flow line of the drainage channel is considerably high that the natural stream, a spillway or chute should be provided to discharge water into natural stream inorder to avoide erosion in the drainage channel. From "Design of Roadside Channels" U.S. Department of Transportation Washington D.C.



TYPICAL SECTION
TWO LANE RURAL HIGHWAY

* SEE TABLE

FIG: 3.1

Height of cut or Fill feet	Earth slope Horizontal to Vertical		
	Flat or rolling	Moderatly steep	steep
0 - 4	6 : 1	4 : 1	4 : 1
4 - 10	4 : 1	3 : 1	2 : 1
10 - 15	3 : 1	2½ : 1	1¾ : 1
15 - 20	2 : 1	2 : 1	1½ : 1
Over 20	2 : 1	1½ : 1	1½ : 1

3.6 MAINTENANCE

Maintenance is aessential to drainage system. The drainage system should be designed, so that it is easy for maintenance department to use their hand tools and equipment to ensure that these systems always function as efficiently as when it was first constructed. Without proper maintenance the roadside channel becomes unsightly gully and some time traffic hazard.

SECTION - VI

DESIGN PROCEDURE

4.1 In this connection, each of the steps considered pertinent to the actual design of a highway drainage system will be considered in their respective order.

A typical layout plan and the drainage criteria will be used. Before any design can be undertaken, certain basic information and data must be available to develop and detail the drainage system.

4.1.1 These data should consist primarily of the following :

1. The contour map of the highway and adjacent area
2. The "drainage working drawing" showing the layout of the Highway.(See fig 4.1)
3. All rainfall data, such as frequency, intensity, and duration of storms.
4. Necessary hydraulic data, graphs and tables for the design, including standard specifications.

4.1.2 In actual design, the initial step is a comprehensive study of the topographic map that is extensive enough to include the areas surrounding the Highway, to permit identifying possible contribution surface of subsurface flow, to determine general direction of flow, and to locate natural water courses or outfalls.

4.1.3 The first step in designing a channel is to determine the quantity of the water the channel is to carry.

4.1.4. Some of the storm formulas used in past give the size of structure directly, but modern practice is to first calculate the anticipated discharge and then design the channels.

4.1.5 The Rational method is recommended in this manual for determining the design discharge for roadside drainage channels draining less than about 500 Acres.

For larger areas other methods are more applicable (Soil Conservation Method, Potter Method and Hydrograph Method).

The expected frequency of occurance of the design discharge is of a concern because economy is always factor in design. Over design and under design both involve excessive costs on a longtime basis. For example a channel designed for 1 year storm frequency would have a small cost to build, but the maintenance cost would be high, because the channel would be damaged almost every year, while on other hand the channel designed for 100 year storm would be high first, but very low maintenance cost. Engineer judgment is required to choose the frequency appropriate for particular design purpose.

4.2 STORM RUNOFF

Precipitation falling on land and water surfaces of the watershed produces runoff. Some times the surface runoff increases due to addition by subsurface flow, that flows just beneath the ground surface and reaches the stream in time to be part of the storm runoff.

The surface runoff which must be carried by the roadside channel is the residual of the preciption after losses (the extractions for interception, infilitration and depression storage). The rate of water loss depends upon the amount of the precipitation and the rate at which it falls (intensity); upon temprature and upon the characterstic of land surface.

4.3 INTENSITY

The intensity of rainfall is the rate at which the rainfalls. Intensity is usually stated in inches per hour regardless of the duration of the rainfall, although it may be stated as total rainfall in a particular period.

There are two methods available for selecting the rainfall data used in frequency analysis. These methods are the annual series and partial duration series. The annual-series analysis consideres only the maximum rainfalls of the each year (usually the calander year) and ignores the other rainfall, of the other years.

The partial-duration series analysis consider all the high rainfalls regardless of the number occurring within a particular year. When the return period (design frequency) is less than 10 years, the partial duration series is believed to be more appropriate.

To convert partial series to annual series, multiply by following factors:

T	Factor
2 Year	0.88
5 Year	0.96
10 Year	0.99

To change the frequency curves based an annual series to one based on partial-duration series, multiply the annual series values by the following factors.

T	Factor
2 Year	1.13
5 Year	1.04
10 Year	1.01

4.4 RATIONAL METHOD

Rainfall Intensity is converted into rate of storm runoff by the rational formula :

$$Q = C\,i\,A$$

Where Q = peak runoff rate, cfs (cubic feet per second) with return period T

C = runoff co-efficient

i = average rainfall intensity, in/hr with return period T.

A = drainage area in acres

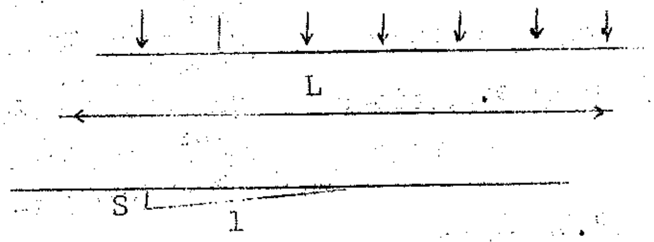
The rational equation is called rational because

$$1.008 \text{ cfs} = 1 \text{ in-acre/hr}$$

i.e. the co-efficient is almost dimensionless.

4.5 PHYSICAL BASIS OF THE EQUATION

4.5.1 Consider a constant intensity rainfall, falling uniformly on a surface.



a. After a time, the flow off the surface is exactly equal to instanteous input on to the surface, A.

b. The time to establish this equilibrium condition is called to time-to-equilibrium te

c. If the rainfall ends before te, the flow rate off the plan would be less than equilibrium value, iA.

d, Since real catchments are previous, a runoff co-efficient is applied

$$Q = C \, i \, A$$

4.6 TIME-TO-EQUILIBRIUM, te Vs TIME OF CONCENTRATION, tc

4.6.1 te : defined above as time for flow rate of the surface to equal instantaneous rate of input on to the surface.

4.6.2 tc = time-of-concentration is a ambiguous concept

a. "time required for a partical of water to move from the remotest part of the basin to the outlet" GRAY

b. "Time required for the runoff to become established and flow from the most remote part of the drainage area to the point under design" ASCE

c. Most Equations in the literature for tc were evolved by measuring te and assuming te = tc

4.7 ASSUMPTIONS OF THE RATIONAL METHOD

a. The peak rate of runoff is a direct function of the average rainfall intensity during the time of concentration.

b. The frequency of peak discharge is the same as the frequency of the average rainfall intensity.

c. The co-efficient of runoff is the same for storms of various frequencies.

4.8 VARIABLES

4.8.1 Drainage Areas, A (acres)

a. obtained from map

b. usually, rational method is applicable to catchments less than 500 acres. Larger catchments will exhibit channel storage characteristics.

4.8.2 Rainfall Intensity (in/hr)

a. rainfall intensity is the average intensity over a duration equal to tc.

b. obtained from an "intensity-duration-frequency" curve for chosen frequency. see fig. (4.1)

4.8.3 Time-to-equilibrium

When considering a catchment, the tc is the time for overland flow plus channel time

a. IZARD

$$tc \quad (min) = \frac{41b \quad L^{1/3}}{(Ci) \quad 2/3}$$

Where $b = \frac{0.000d + C}{S \quad 1/3}$

and i = rainfall intensity (in/hr)

S = Slope

L = Length of overland flow (ft)

C = Runoff Co-efficient

c = ratardance co-efficient

SURFACE TYPE	C
Smooth asphalt	0.007
Concrete paving	0.012
Tar and gravel paving	0.017
Closely clipped sod	0.046
Dense bluegrass turf	0.060

b. Kirpich's equation

$$tc\ (min) = 0.0078\ L^{0.77}\ \ S^{0.385}$$

When notation is the same as shown previously

c. Kerby's equation

$$tc\ (min) = (2/3\ LK/s^{\frac{1}{2}})^{0.467}$$

Where K is a retardance co-efficient.

TYPE OF SURFACE	K
Impervious surfaces	0.02
Bare packed soil, smooth	0.10
Bare surfaces, moderately rough	0.20
Pasture or average grass	0.40
Timber Land, Deciduous trees, deeplitter	0.80
Timber Land, Conifers	0.80
Dense grass	0.80

d. Kinematic wave-a fluid mechnical analysis of overland flow led Ragan and Duru (1972) to publish the nomograph. see fig (4.1)

$$tc\ (min) = \frac{0.93\ L^{.6}\ \ n^{.6}}{{}^{.4}\ \ S^{.3}}$$

4.9 RUNOFF CO-EFFICIENT

Traditionally thought of as a constant, dependent on soil type and land use. Where the drainage area is composed of several types of ground cover, the runoff co-efficient should be weighted. Several tables for 'C' are shown on page 36,37 and 38.

To find weighted co-efficient 'C'.

For example :

Given : Asphalt pavement draining towards drainage channel (Gravel) shoulder 8' wide and 500 long.

Side slopes 15 Wide and 500 long

Pasture 300 Wide and 600 long

Area Sq.Ft	Type of Surface	C	CA
6000	Asphalt Pavement	0.9	5400
4000	Shoulder (Gravel)	0.7	2800
7500	Side Slopes	0.5	3750
150000	Pasture	0.3	45000
167500			56950

Weight $C = \frac{56,960}{167,560} = 0.34$

For use in rational formula, the CA product (56,960) can be converted to acre and multiplied by without computing the weighted 'C'.

4.10 TIME OF CONCENTRATION

Extreme precision is not warranted in determining time of concentration for the design of drainage channels of rural highways. Time of concentration can be obtained by using any of the formulas given in section 4.8.3 or using nomographs developed by p.z. Kirpich Fig. 4.3.

4.11 RAINFALL INTENSITY

Rainfall intensity-frequency data are taken from weather Bureau Atlas.

A chart such as that of fig (4.4) should be constructed for project location for the frequencies to be used for the project design.

For the use in the rational method, the values of total rainfall are converted to rainfall intensity by dividing the rainfall frequency Atals map value by the duration expressed in hours.

These maps should be constructed for Pakistan's different regions using the rainfall data obtained from Metrological Department of Pakistan.

See example Rainfall intensity-duration-frequency curve developed for Washington D.C. Fig 4.4

4.12 DRAINAGE AREA

The drainage area, in acres, contributing to the point for which channel capacity is to be determined, can be measured on a topographic map or determined in the field by estimation, pacing, or a survey comparable in accuracy to the stadia-compass traverse.

Computing the design discharge:

The design discharge is computed by using the rational equation $Q = C i A$.

A typical Layout of Drainage Plan is Shown in Fig (4.1) Area (A_1) consists of overland flow draining towards intercepting channel,

Area (A_2) consists of pavement shoulder and side slopes.

Area (A_3) and (A_4) consist of pavement, shoulder, side slopes and overland flow, therefore weighted co-efficient of runoff 'C' should be computed for these areas.

Then Compute $Q = C i A$ in cfs

TO FIND DISCHARGE $Q = CiA$

Step : 1 Measure the areas in acres A_1 see fig 4.1

Step : 2 Choose appropriate Runoff-Co-efficient 'C' and find weighted 'C' for area A_1

Step : 3 Find Time of concentration T_c (From formula or Nomograph)

Step : 4 Find intensity (i_{10}) (From Intensity-duration-Frequency curve for Location)

Step : 5 Compute $Q_1 = Ci_{10} A_1$

Similarily Q_2' Q_3 and Q_4 can be computed

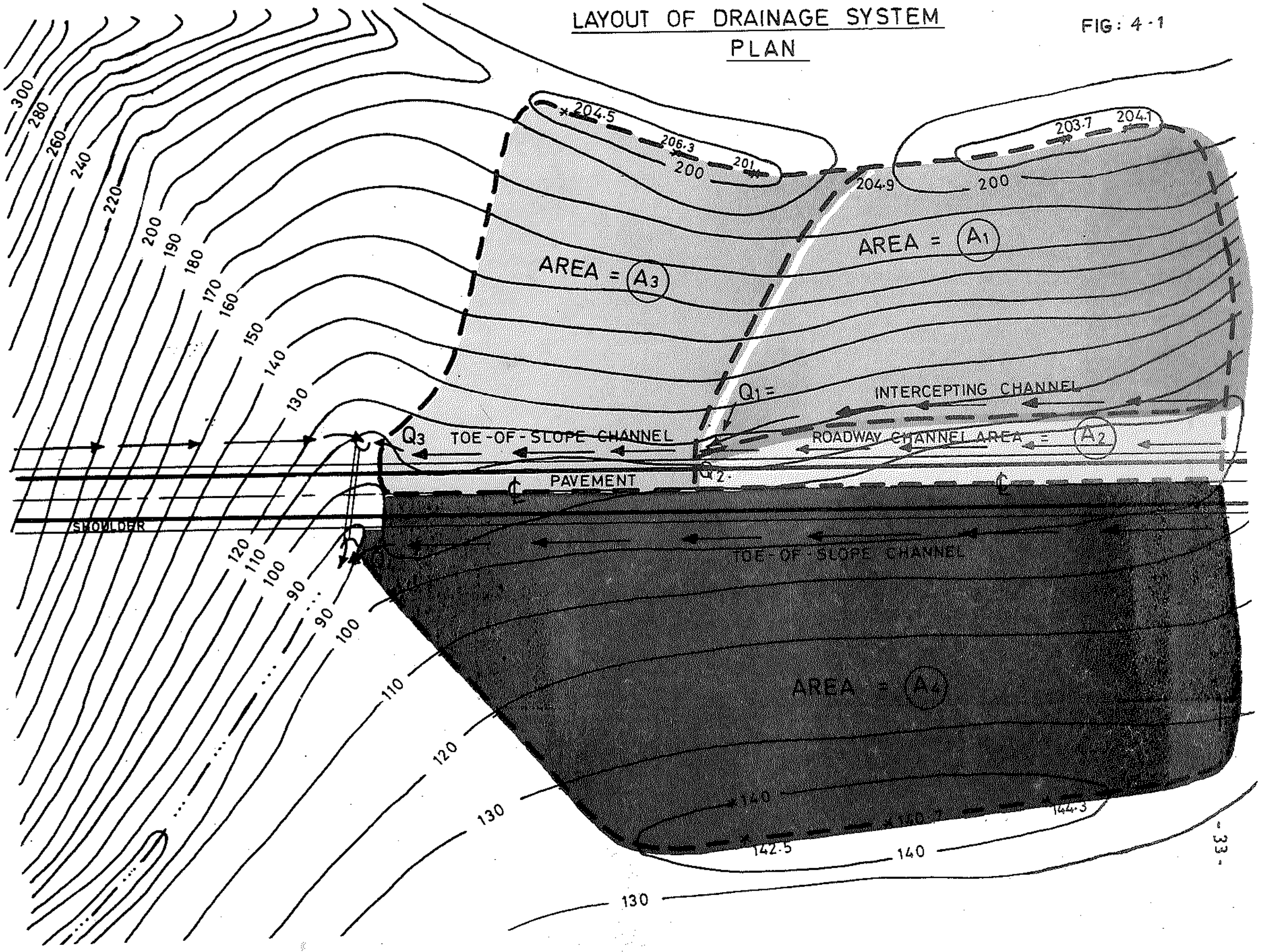
4.13 FOR CHANNEL DESIGNS

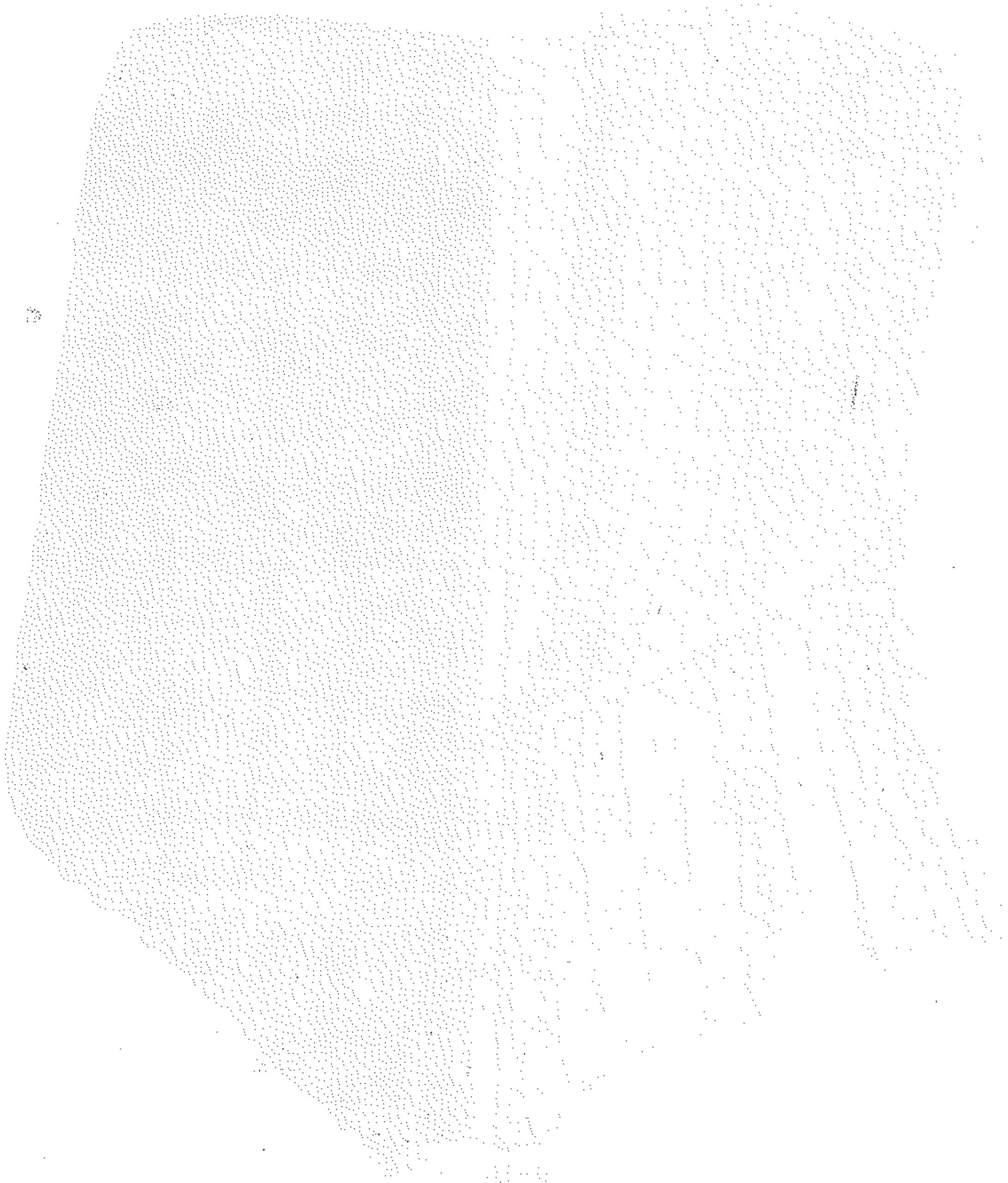
Use minimum time of concentration = 5 min

Frequency = 10 years

LAYOUT OF DRAINAGE SYSTEM
PLAN

FIG: 4·1





$t_c = 0.93 \frac{L^{.6} n^{.6}}{I^{.4} S^{.3}}$

Example
L = 400ft
n = 0 015
I = 5·5 in/hr
S = 0 01
l_c = 5 5min

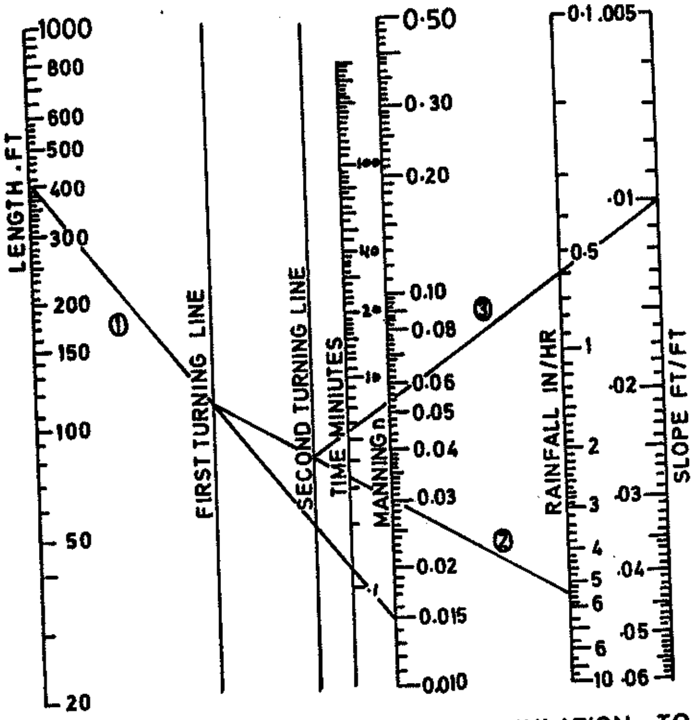
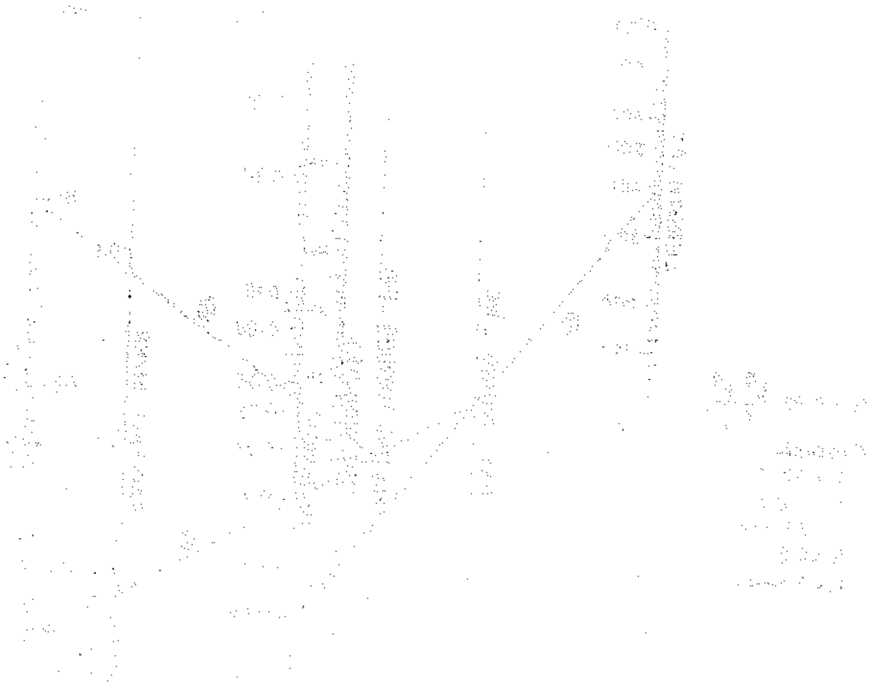
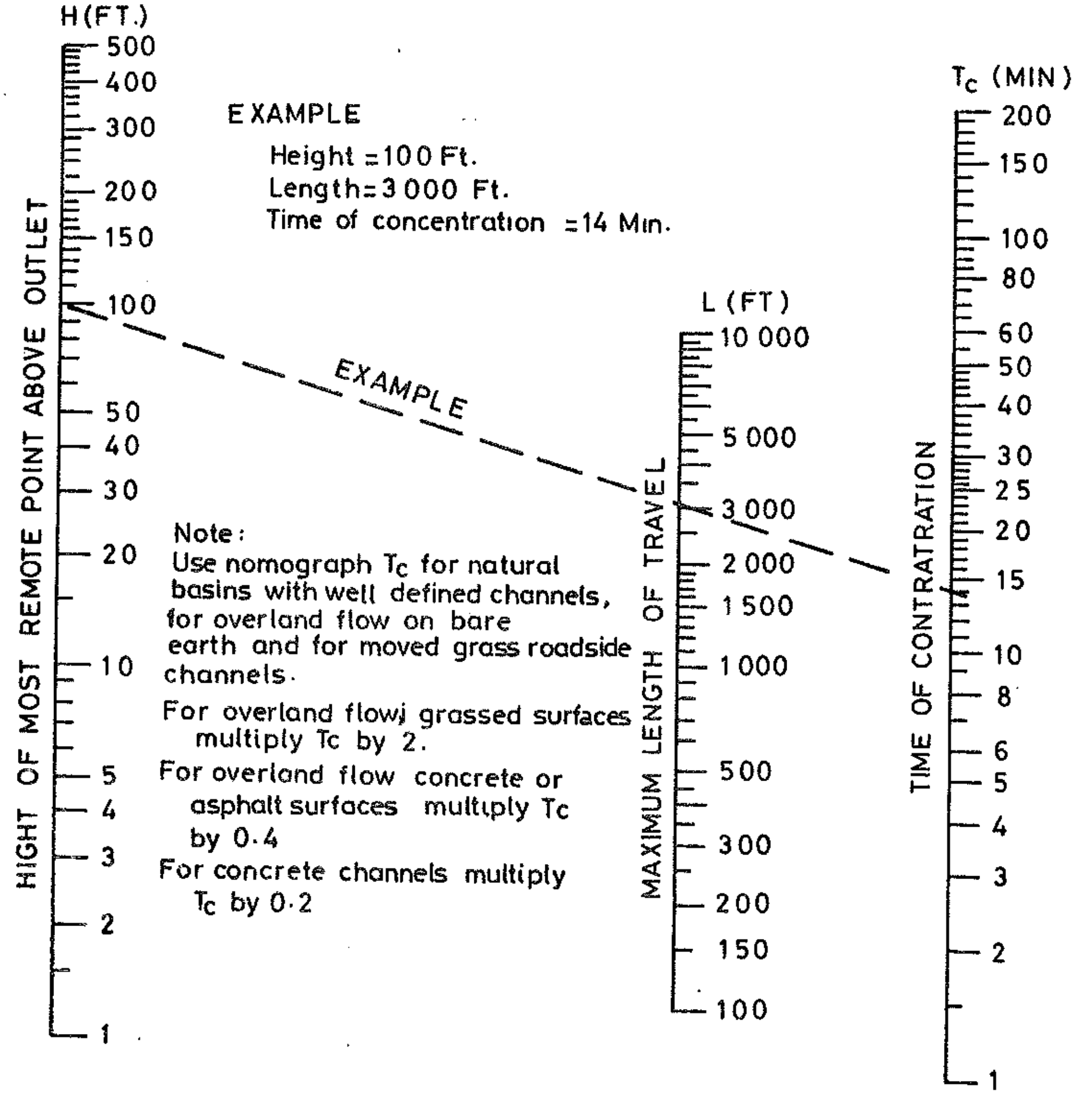


FIG:4.2-NOMOGRAP USING KINEMATIC WAVE FORMULATION TO OETERMINI TIME OF CONCENTRATION FOR OVERLAND FLOW



[illegible]



EXAMPLE

Height =100 Ft.
Length=3 000 Ft.
Time of concentration =14 Min.

Note:

Use nomograph T_c for natural basins with well defined channels, for overland flow on bare earth and for moved grass roadside channels.

For overland flow, grassed surfaces multiply Tc by 2.

For overland flow concrete or asphalt surfaces multiply Tc by 0.4

For concrete channels multiply T_c by 0.2

Based onstudy by P.Z. Kirpich,
Civil Engineering VOL 10, No 6, June 1940, p.362

FIG: 4.3:Time of concentration of small drainage basins.

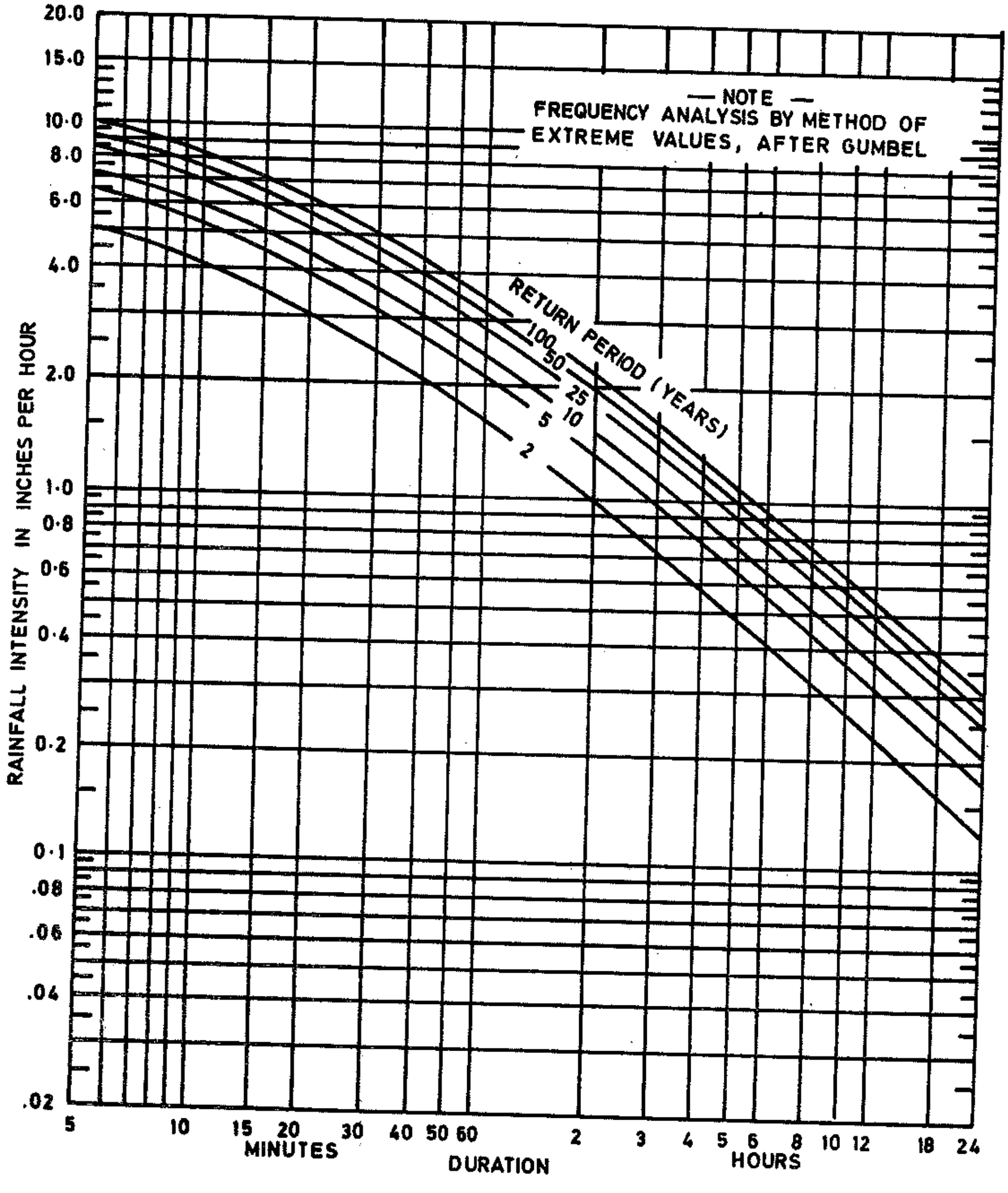


Figure 4.A Rainfall intensity duration frequency curve for Wash. D C., 1896-97, 1899-1953. (U.S. Weather Bureau.)

From ASCE M&R No. 37

Description of Area	Runoff Cefficients
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	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, heavy 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

From Chow (1964)

Table 14-1. Values of Runoff Coefficient C

Type of drainage area	Runoff co-efficient C
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average 2-7 %	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi units, detached	0.40 - 0.60
Multi units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.35
Railroad yard areas	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs	0.75 - 0.95

Table 1. Values of Runoff Coefficients (C) for Use in the Rational Method

Type of surface	Runoff Coefficient (C)
Concrete of sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadways or shoulders	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed areas (2:1)	0.5 - 0.7
Turf meadows	0.1 - 0.4
Forested areas	0.1 - 0.3
Cultivated fields	0.2 - 0.4
Urban Areas[2]	
Flat residential, with about 30 percent of area impervious	0.4
Flat residential, with about 60 percent of area impervious	0.55
Moderately steep residential, with about 50 percent of area impervious	0.65
Moderately steep built up area, with about 70 percent of area impervious	0.80
Flat commercial, with about 90 percent of area impervious	0.80

SECTION - V

HYDRAULICS OF DRAINAGE CHANNEL

5.1 Flow in open channels is classified as steady or unsteady. Although the flow in most channels during the storm is unsteady flow, but assuming the peak flow as steady flow greatly simplies the design of drainage channels.

Steady flow occurs when the quantity of water passing and section of the steam is constant steady flow is further classified as uniform if velocity and depth of flow are constant, and nonuniform or varied if velocity and depth of flow changes from section to section.

5.2 UNIFORM FLOW

For uniform flow grade must be constant and all cross-sections of flow must be identical in form, roughness, and area, necessitating a constant mean velocity. Under uniform flow conditions, the depth (dn) and the mean velocity (Vn) for a particular discharge are said to be normal. Under these conditions the water surface is parallel to stream bed.

5.3 MANNING'S EQUATION

Manning's equation may be used if the channel slope, roughness and cross-section are constant for some distance.

5.3.1 Manning equation: In 1890, Manning presented evidence that the chezy C varied with the hydraulic radius raised to one-sixth power,

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

In which n is the Manning Roughness Co-efficient.

5.3.2 MANNING FORMULA

In English Unit $Q = \frac{1.49}{n} AR^{2/3} S^{\frac{1}{2}}$

Q = (cfs)

$Q = AV$

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

V = mean velocity in feet/second

n = manning co-efficient of channel roughness

R = hydraulic radius in feet

S = Slope, in feet per foot

R, the hydraulic radius, is a shape factor that depends only upon the channel dimensions and the depth of the flow.

It is computed by the equation

$$R = A/WP$$

where A = cross-section area of the flowing water in square feet taken at right angles to the direction of flow. WP = wetted perimeter of the length, in feet, or the wetted contact between a stream of water and its containing channel, measured in a plane at right angles to the direction of flow.

5.3.3 In SI Units $Q\ (m^3/S) = 1/n\ AR^{2/3}\ S^{\frac{1}{2}}$

5.3.4 Nomographs such as that shown in fig 5.1 provide a graphical solution of manning equation

Example: The drainage channel, of straight alignment and uniform cross-section in earth, bottom width 2ft., side slope 1:1 and depth 1ft.

Nomograph for solution of manning Equation

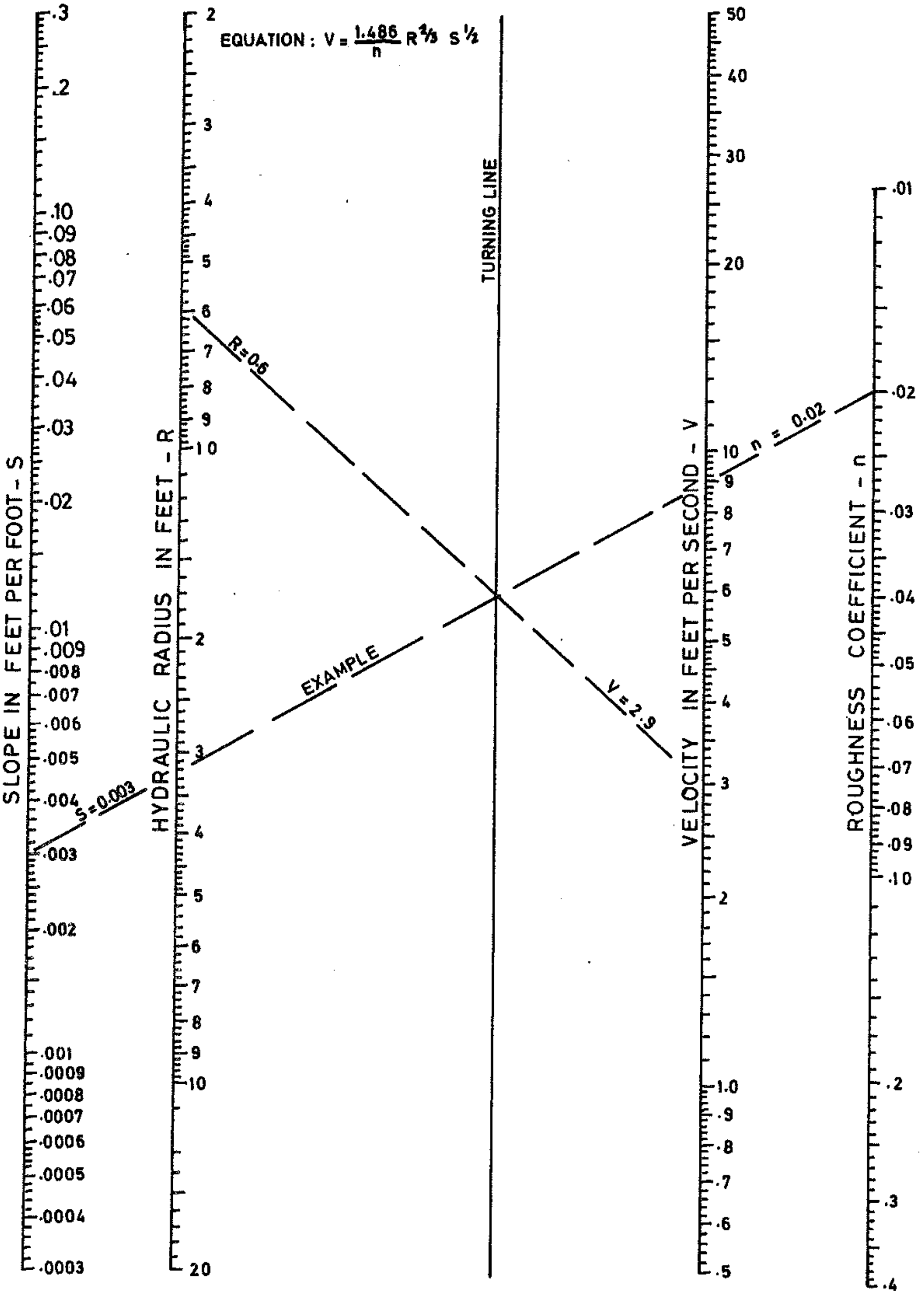
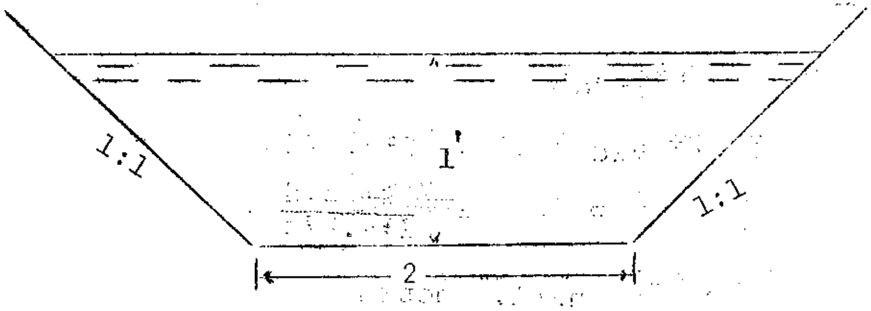


FIG = 5·1

To find velocity and discharge manning n for earth channel n = 0.02.



Area = 2+1 = 3ft

WP = 2+(2x1.41) = 4.83

A/WP = $\frac{3}{4.83}$ = 0.62

Using the nomograph fig 5.1

and values of S = 0.003, n = 0.02, Velocity = 2.9 f.p.s

The discharge Q = AV

= 3.0 x 2.9 = 8.7 cfs

5.4 SIZE OF CHANNEL

The common problem in channel design is to find the size of channel required to carry the design discharge on the available slope and to compute the velocity in the channel in order to determine what protection is needed to prevent channel erosion, Channel charts are available to solve these problems. If the charts are not available, to solve this problem trial and error procedure are used, by varying depth and width of channel and select appropriate section.

Example : A trapezoidal grassed line channel bottom width 2', with side slopes 2:1, and n = .05 slope = 0.01 discharge = 35 cfs find = Depth d, and velocity, V

$$A = (2+2d)d$$

$$WP = 2+2d \quad 5 = 2+4.47d$$

$$R = A/WP \quad Q = \frac{(2+2d)d}{2+4.47d} =$$

Now assuming 1.5 depth

$$R = \frac{(2+2x1.5)1.5}{2+4.47x1.5} = 0 = \frac{7.50}{8.71} = 0.86$$

Using Nomograph fig 5.1

$n = 0.05 \quad S = 0.01$

Velocity = 2.70

$Q = 2.70 \times 7.50 = 20.25$ cfs

Now assuming 2' depth.

$$R = \frac{(2+2x2)x2}{2+4.47x2} = \frac{12}{10.94} = 1.10$$

Velocity = 3.2 Using Nomograph (5.1)

3.5 FPS

$Q = 3.2 \times 12.0 = 38.4$ cfs

Now assuming depth 1.8 feet

$n = 0.05 \quad S = 0.01$

$R = 1.05$

From Nomograph

Velocity = 3.12

$Q = 11.02 \times 3.12 = 34.38 = 35$ cfs

and velocity 3.1 f.p.s which can be permitted in grass lined channels.

5.5 NON-UNIFORM OR VARIED FLOW

A. Varied steady flow occurs when the quantity of water remains constant, but the depth of flow, velocity, or cross-section changes from section to section. The continuity equation is

$$Q = A_1V_1 = A_2V_2 \ldots\ldots\ldots\ldots\ldots A_n \times V_n$$

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.

B. Conservation of Energy

1. The energy equation for open channel flow is:

$$\frac{V1^2}{2g} + D_1+Z_1-h_f = \frac{V2^2}{2g} + d_2 + Z_2$$

Where d = depth of flow at any section

d_c = critical depth of flow in channel

g = ft/sec^2 = Acceleration of gravity

z = ft = Elevation of bed of

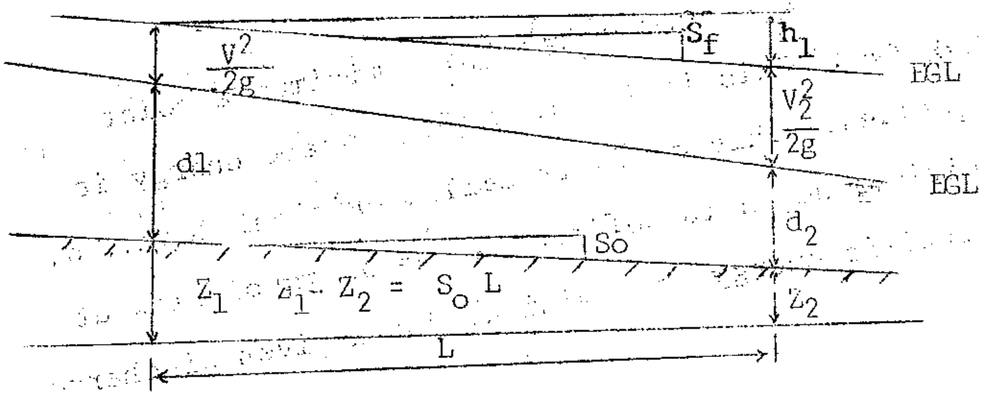
Channel above a datum

L = Length in feet

EGL = Energy gradient line

HGL = Hydraulic gradient line

S = Slope foot/feet



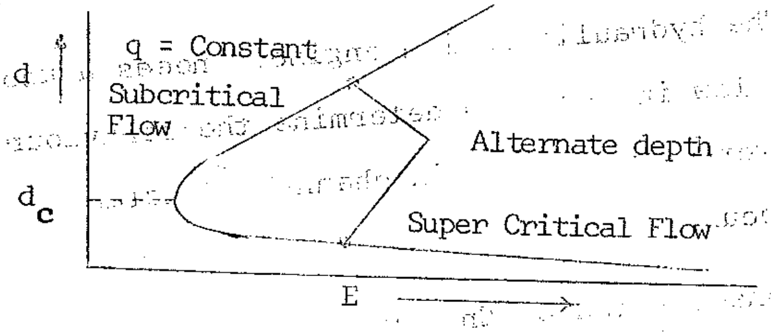
2. Specific Energy - Energy at a section with reference to the channel bottom is called specific energy, E

$$E = \frac{V_2}{2g} + d$$

for rectangular channel (b = width)

$$E = \frac{q^2}{2gd^2} + d \qquad \text{Where } q = Q/b$$

for a given discharge and channel shape, the specific diagram can be constructed.

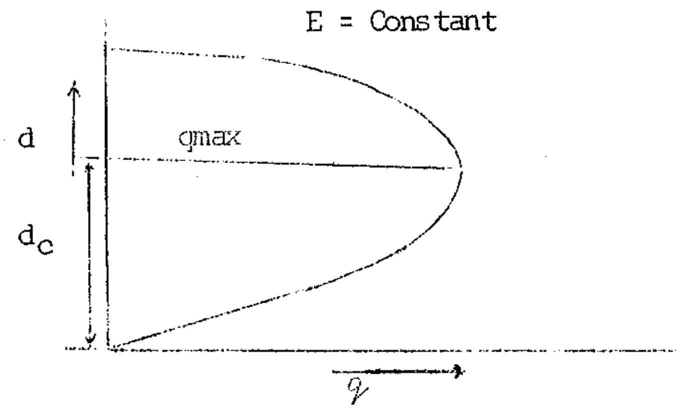


Depth - Vs - specific energy relation

It can be seen that for a given discharge there are two depths for which the flow has the same energy - one supercritical and one subcritical.

The specific energy reaches a minimum a point on the curve. This is the only point where energy is single valued. This depth is called critical depth, d. Thus critical depth is defined as the depth of flow at which specific energy is a minimum for a given discharge.

An alternative definition of critical depth is that depth for which the discharge is a maximum for a given specific energy.

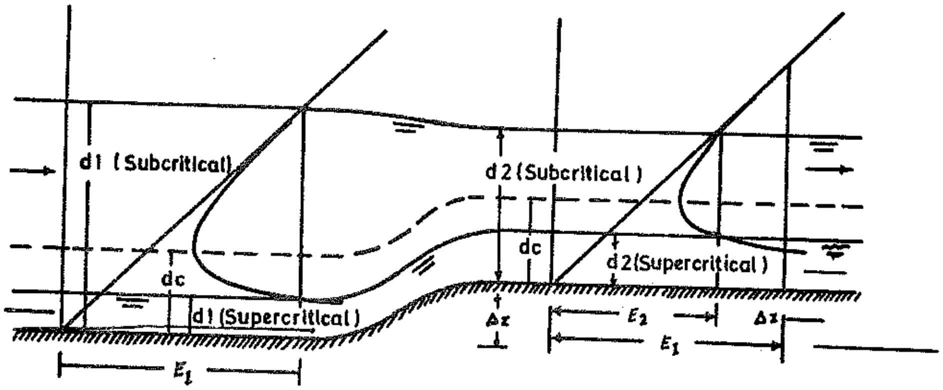


Variation of q and d constant specific energy.

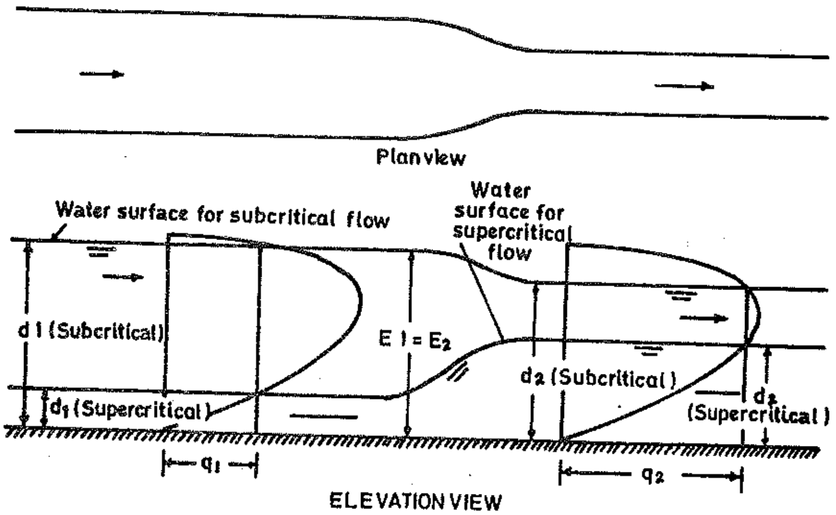
These two diagram can be used to determine depth changes due to channel transitions. By assuming no change in total energy.

a. a change in bottom elevation changes E by the amount of elevation change, ΔZ

b. a change in channel width changes the discharge per unit width, q



Changes in depth with change in bottom elevation



Changes in depth with changes in channel width

5.6 SIGNIFICANCE OF THE ROUGHNESS (MANNING 'N)

When new the carrying capacity of the channel is greater than that for the channel is designed. Therefore a value of n should be carefully selected from the table, usually midway, so that higher velocity does not damage the newly constructed channel, before it reaches the design condition.

5.7 CHANNEL PROTECTION

Maximum permissible velocities for channels in various soils types should be determined. If the mean velocity at the design flow exceeds the permissible velocity for the particular soil types, the channel should be protected from erosion. Channel protection can be provided by lining with grass, concrete, bituminous material, stone, fiber glass, or a preformed material such as metal or wood fiber impregnated with pitch. Generally, the lowest cost (including maintenance cost) lining should be used. The type of channel lining might vary along the length of the channel, using low cost lining such as grass on the flatter slopes and a high cost lining such as concrete on the steeper slopes.

5.8 BUOYANCY OF EMPTY CHANNEL

In saturated soils, empty channels with rigid linings may float, or breakup because of the uplift water pressure. The total upward force is equal to the weight of the water displaced by the channel or 62.4 times the volume in cubic feet of the portion of the channel cross-section which lies below the water table.

The uplift pressure is resisted by the total weight of the lining.

When the weight of the lining is less than the uplift pressure the channel is unstable in saturated soils. Then the lining should be increased in thickness to add additional weight or if the flow is subcritical, weep holes may be placed at intervals in the channel bottom to relieve the upward water pressure in the channel. The diameter of the holes should be 2 inches to 4 inches and spacing between depends on type of the soil. When the flow is supercritical, subdrainage should be used rather than weep holes to reduce uplift pressure.

5.9 BANK AND SHORE PROTECTION

Stream crossings are unavoidable, but most crossings expose highway embankments to attack by the stream. If the erosion of the highway embankment by a stream is to be prevented, the need for bank protection must be anticipated and the proper type and amount of protection provided in the right places. Bank protection is more economical, than the damage caused by flood to highway embankment, and repair cost after each flood.

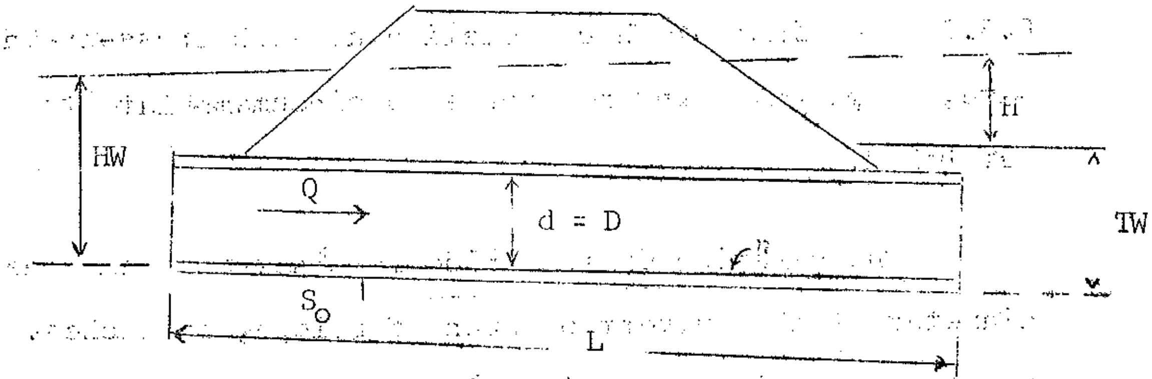
From "Roadside Drainage Channels" U.S. Dept. of Transportation.

SECTION - VI

6.1 A culvert is covered channel of comparatively short length installed to drain water through highway and railroad embankment.

A bridge carries traffic over a stream, a culvert carries a stream under the traffic !

The Installed Culvert



d = depth of flow in culvert ($\leq$ D)

D = inside height of culvert

H = culvert head or energy required to pass the flow

HW = headwater depth above invert at inlet

L = length of culvert

h = manning roughness co-efficient

Q = discharge through culvert

S_o = culvert barrel slope

TW = tailwater depth above invert at outlet

6.2 EARLIEST DESIGNS (HISTORY OF CULVERT DESIGN)

1. Judgement of non-professionals
2. Built with local materials by cheap Labor
3. Often washed out but economic loss small

6.3 EARLY EMPIRICION

6.3.1 One-man personal-judgement system-at each cross drainage site, estimate the average channel width in 2ft unit, then use that many parallel 24-in culverts.

6.3.2 The minute-inch correlation at each cross-drainage site, dispatch a man on a horse to circumambulate the watershed.

The duration of the ride in minutes bacame the diameter of the culvert or span of bridge, in inches. This was the beginning of rationalization !

6.4 FORMULA WORSHIP-ANYTHING CAN BE DESIGNED BY DIRECT APPLICATION OF A FORMULA

a. Talbot formula

b. Burkli-Ziegler formula

6.5 FREE ADVICE FROM PIPE VENDORS

a, Especially attractive if project engineer lacked a competent culvert designer.

b. Vendors could pad an estimate with over design, but this atleast prevented underdesign by incompetence.

6.6 USE OF "RATIONAL FORMULA"

a. Requires integration of hydraulics, hydrology and hydrography.

b. Design handled by specialists.

6.7 MORE COMPREHENSIVE DESIGN BASED ON A CONSIDERATION OF ASPECT IN

a. Hydrology

b. Hydraulics

c. Economics

d. Highway Engineering

6.8 TYPES AND USES OF CULVERTS

6.8.1 Longinitudinal Classification

a. Normal-straight through embankment

b. Sag (inverted siphon) culvert

c. Drop-inlet culvert

6.8.2 Cross-Section Classification

a. Box culvert

b. Circular Culverts

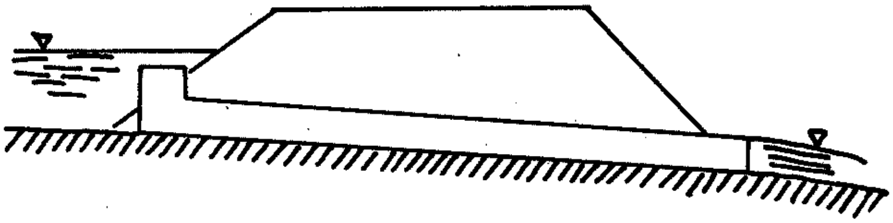
c. Oval Culvert

d. Pipe arch culvert

6.9 MATERIALS OF CONSTRUCTION

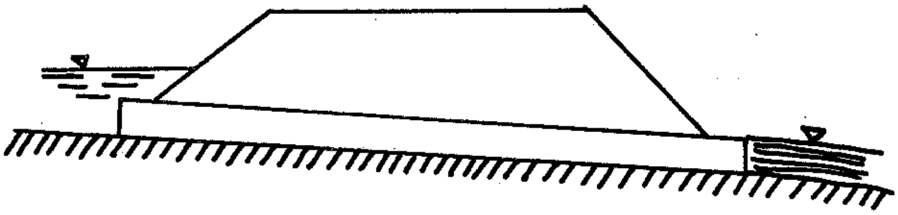
6.9.1 Culvert barrels are made from a variety of materials

a. Precast concrete	Small
b. Vitrified-clay	
c. Plastic	
d. Cast iron	
e. Corrugated metal pipe	
f. Corrugated metal arches	Large
g. Reinforced concrete ovals	
h. Concrete box culvert	



Drop Inlet

Sag or Inverted Siphon



Normal

Culvert types - Longitudinal.

BOX Circular Pipe arch Oval

Culvert types - cross sectional

6.9.2 Type of culvert material chosen on a competative economic basis with appropriate consideration given to the following variables :

a. durability

b. structural strength

c. bedding conditions

d. abrasion and corrosion resistance

e. water tightness

f. hydraulic roughness

  i) rougher pipe may aid in energy dissipation-reduce outlet velocity

  ii) smooth pipe may lower HW for a given discharge if outlet control

6.10 CULVERT USES

a. Highway, railway, and canal drainage
b. Outlet conduits for dams and tanks
c. Turnout structures for irrigation canals
d. Filling/Emptying conduit for lock chambers
e. Storm water sewers from street inlets to streams.

6.11 IMPORTANCE OF CULVERTS

6.11.1 Approximately 25%-30% of highway construction costs are associated with drainage, and more than half of this specifically for culverts.

6.11.2 Each culvert, no matter how small, is an integral part of the transportation system.

6.12 PROPER DESIGN IS ESSENTIAL

6.12.1 The fundamental objective

"to determine the most economical diameter (size) to pass the design discharge without exceeding the allowable headwater"

6.12.2 Design Philosophies:

a. Capacity - culvert should reliably carry design discharge
b. Performance - certainty of culvert performance; is the head-discharge relationship unique ?
c. Application-Use of all available knowledge of culvert hydraulics by competent designer.
d. Service life-design culvert to have service life comparable to the rest of the system.

6.13 DESIGN UNCERTAINTIES

6.13.1 Uncertainties complicate the determination of optimal design. The uncertainties can be classified as:

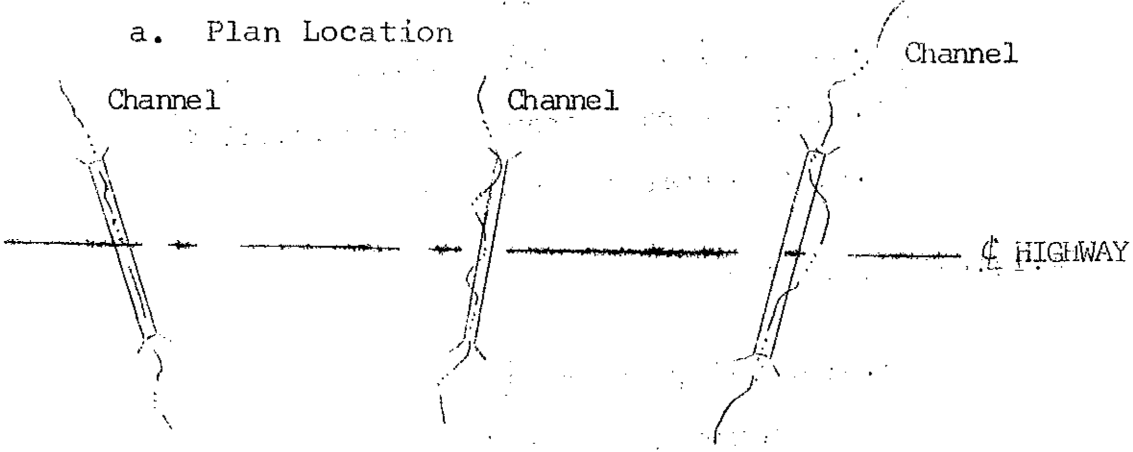
a. Hydrologic
b. Hydraulic
c. Structural
d. Economic

6.13.2 Design consideration of these uncertainties requires the use of statistical and probability models in conjenction with an optimization procedure.

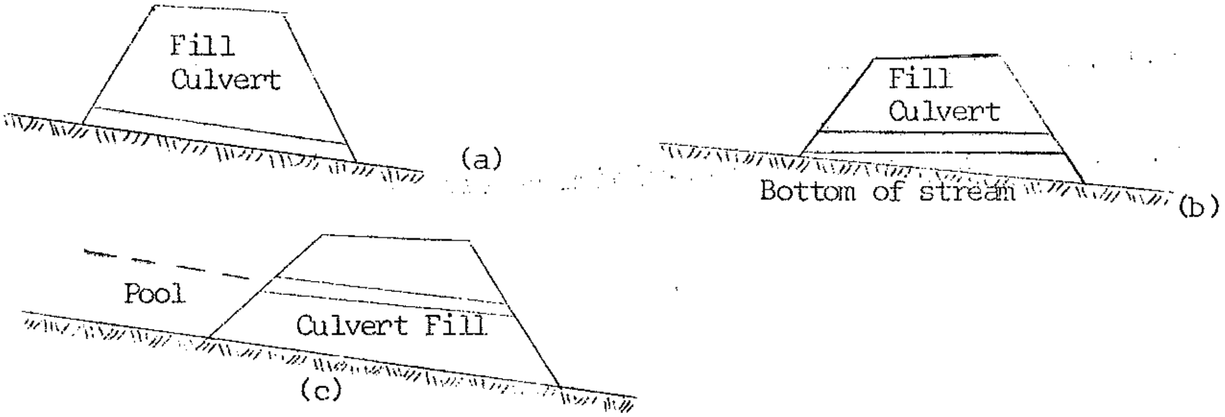
6.14 DESIGN ASPECTS

6.14.1 Planning

a. Plan Location



b. Profile Location



6.14.2 Alignment

As a general rule, it is best that culvert alignment conform to the natural stream alignment.

6.14.3 Hydrology-determines design discharge

6.14.4 Channel Training

a. Longnitudinal drainage-ditches or channels along the road

b. Cross-drainage-approach to culverts through road

6.14.5 Culvert Type

a. Cross-Section shape

b. Materials of Construction

c. Structural Strength and Durability

d. Installation Procedure

6.14.6 Hydraulic

a. Barrel roughness

b. Inlet/outlet conditions, beadwalls,wingwalls

c. Appurtenances; debris control structures, energy dissipators.

6.14.7 Maintenance

6.14.8 Economical design-minimize costs

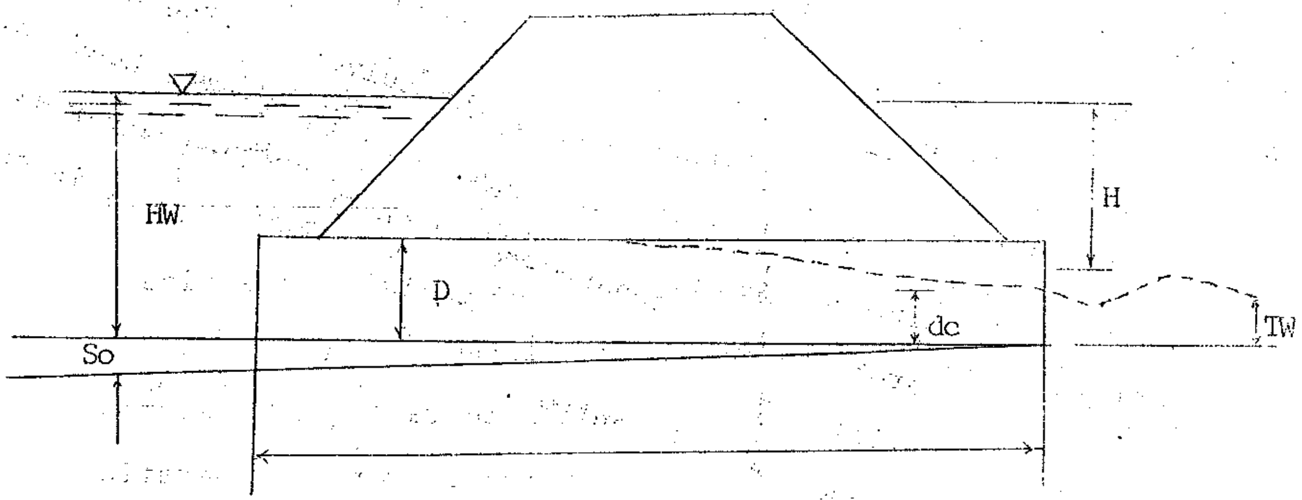
SECTION - VII

DEFINITION AND BASIC CONCEPTS

The Culvert

7.1 A culvert is simply a short, enclosed conduit which, due to the highly variable discharge it carries, may behave as a pipe flowing full, an open channel with a free surface, or a combination of both.

7.2 The culvert may or may not have special appurtenances such as headwalls, wingwalls, debris barries, and tapered inlets at the culverts entrance, and endwalls, wingwalls, and energy disipators at the outlet.



$$\frac{d_c+D}{2} \quad \text{Or } TW = h_o$$

WATER LEVELS

7.2.1 The headwater elevation, HW, is controlled by the culvert and its appurtenances.

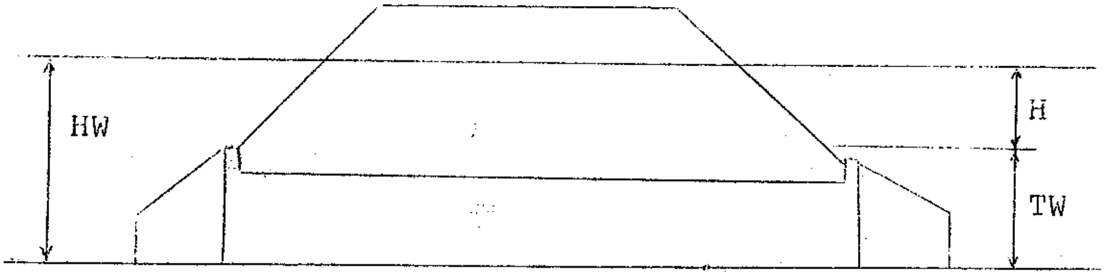
7.2.2 The tailwater elevation, TW is controlled by the channel downstream from the culvert.

7.3 FLOW CONDITION AND HYDRAULIC EQUATIONS AND TYPE OF CULVERTS FLOW

For practical purpose, culvert flow may be classified into six types, three have submerged entrance conditions and three have a free entrance condition.

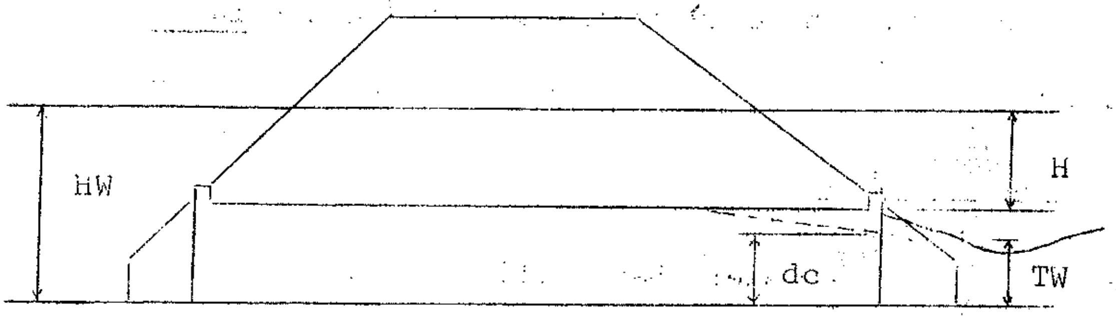
7.4 SUBMERGED ENTRANCE

7.4.1 Outlet submerged, culvert flow full



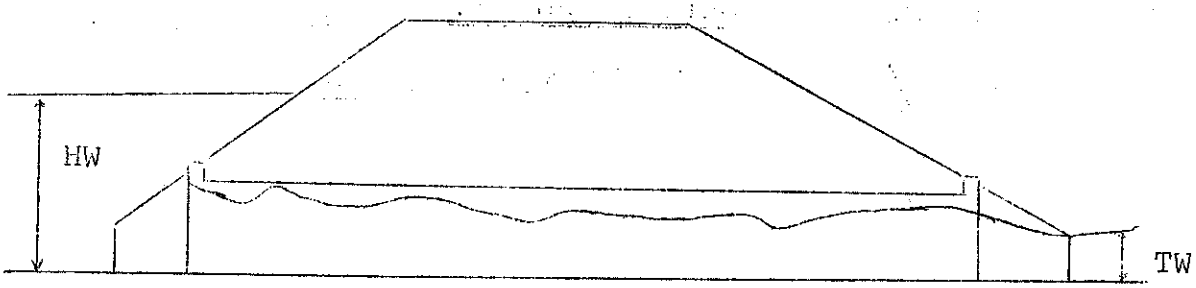
In this condition the culvert behave as a pipe connecting two reserviores. The larger the head, H, the greater the discharge. Any change in tailwater elevation will effect the headwater, hence the culvert is operating under outlet control.

7.4.2. Outlet Free Culvert Flow Full



In this condition the culvert flow full because the barrel slope is too flat to overcome friction losses. The normal depth of flow for an open channel of this slope is greater than the pipe diameter. The tailwater elevation does not affect the flow, but the culvert length, culvert slope, and culvert roughness have an important influence on the flow. When critical depth falls below the crown of the culvert, the culvert is operating under outlet control.

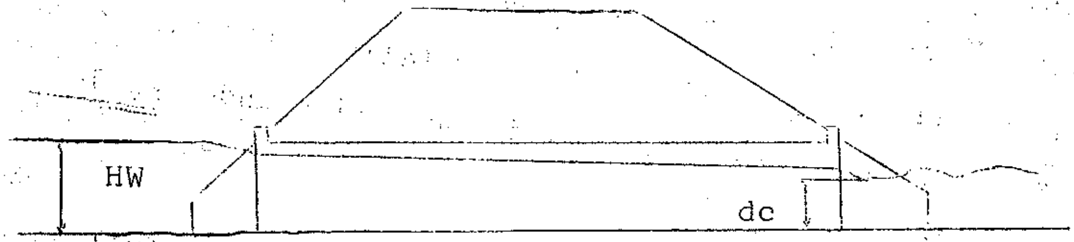
7.4.3 Outlet free, culvert flowing partially full



In this condition a free water surface exists through the culvert barrel. The inlet behaves as an orifice and neither the tailwater nor the barrel conditions affect the flow. The culvert is operating under inlet control.

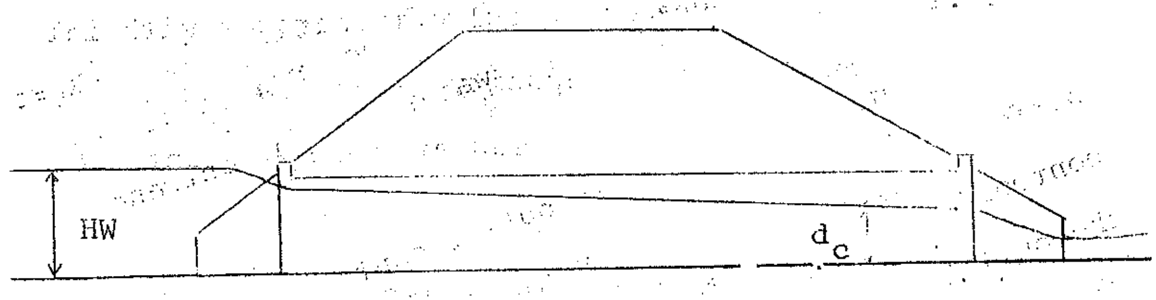
7.5 FREE ENTRANCE

7.5.1 Tailwater greater than critical depth



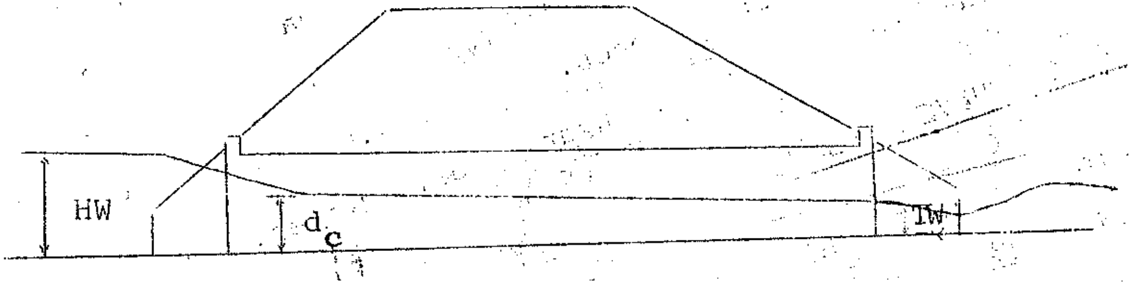
In this condition the flow is subcritical throughout the culvert. Any change in tailwater elevation will influence the headwater elevation. The culvert is operating under outlet control.

7.5.2 Tailwater less than critical depth, critical depth at outlet.



In this condition the tailwater elevation has no effect on the flow. However the barrel flow is subcritical and the headwater elevation is influenced by culvert length, culvert slope, and culvert roughness. The culvert is operating under outlet control

6.5.3 Tailwater less then critical depth, critical depth at inlet.



In this condition the entire flow through the culvert is supercritical. Hence conditions in the barrel or at the outlet have no influence on the headwater elevation. The inlet behaves as a weir, and the culvert is operating under inlet control.

7.6 FLOW CONDITIONS

There are two major conditions of culvert flow, inlet control and outlet control.

7.6.1 Inlet control: A culvert operates with inlet control when the flow capacity is controlled at the entrance by the depth of headwater and the entrance geomatry.

7.6.2 Outlet control: In outlet control, the culvert performance is determined by the factors governing inlet control plus the controlling water surface elevation at the outlet and the slope, length, and roughness of the culvert barrel.

7.7 INFLUENCE OF CULVERT SLOPE

Some of the types of culvert flow depends upon whether the slope of the culvert barrel is hydraulically steep or hydraulically mild. Types 6.4.1 and 6.4.3 can occur for either mild or steep slopes. Types 6.4.2, 6.5.1 and 6.5.3 can only occur on mild slopes. Type 6.5.4. a can only occur if the barrel slope is hydraulically steep.

7.8 EQUATIONS OF CULVERT FLOW

To develop a relationship between discharge and headwater elevation, the energy equation is used. The basic analysis is different for the two control conditions.

7.9 INLET CONTROL

7.9.1 For headwater levels small enough such that the entrance is free, the inlet behaves as a weir with critical depth occuring at the inlet. The barrel slope is steep.

7.9.2 For headwater levels large enough such that the entrance is submerged, the inlet behaves as an orifice.

7.9.3 In either case the following equations may be used.

$$Q = Cd \quad A \quad 2g \quad (h + \frac{Va^2}{2g})$$

Cd = discharge co-efficient $= \frac{Cc}{1+Ke}$

(Cd = 0.62 for a sharp edged inlet, while for a well rounded inlet Cd approaches unity)

Cc = Co-efficient of contraction

Ke = Entrance loss co-efficient

A = Available flow areas at entrance

h = $HW-d_c$, for free inlet

$HW-D/2$ for submerged inlet

Va = Velocity of approach (usually negligible)

7.10 OUTLET CONTROL

7.10.1 For culverts flowing full under outlet control, the effect of inlet, outlet and barrel conditions must be included in the analysis.

Total energy loss consists of

i) entrance loss, $H_e = Ke \frac{V^2}{2g}$

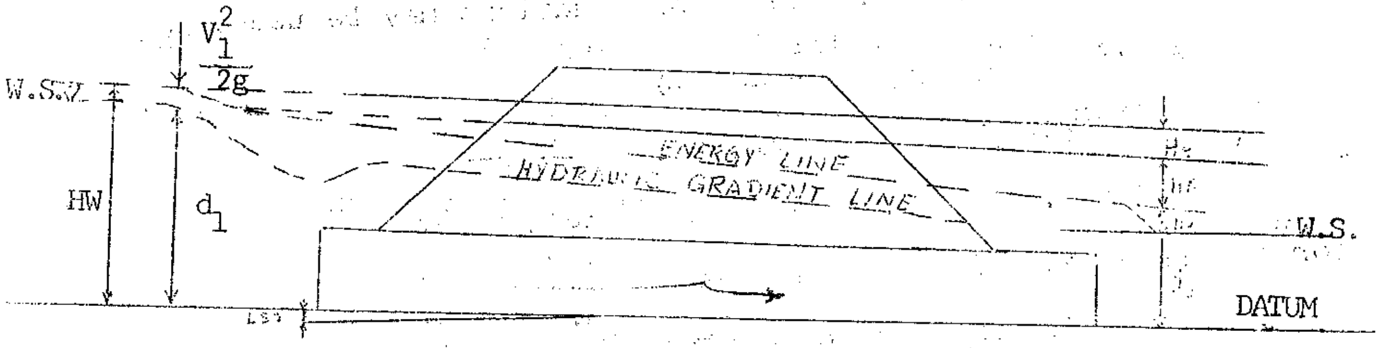
ii) friction loss (from manning equation)

$$H_f = \frac{29 \ n^2 L}{r^{4/3}} \quad \frac{V^2}{2g}$$

iii) exit loss or energy at exist

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

These losses are shown schematically in the following sketch

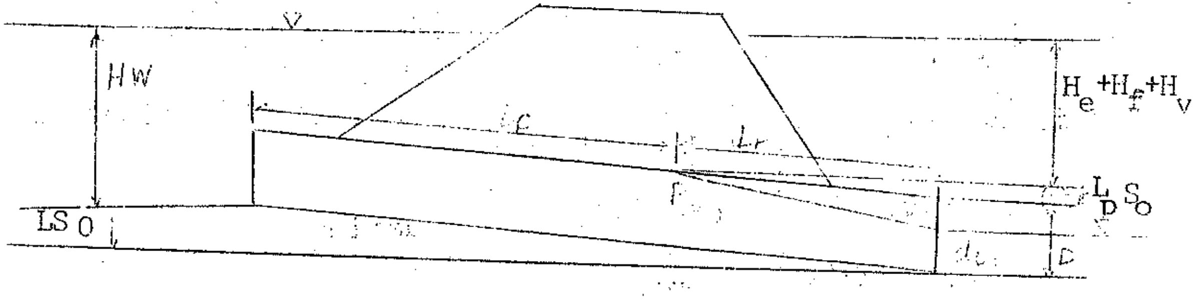


The headwater elevation is determined by the flowing equate.

$$HW = H_e + H_c + H_v + TW - Lso$$

NOTE : The HW includes the approach velocity head.

7.10.2 For the flow condition in which only a portion of the culvert barrel flows full with critical depth at the outlet, the analysis is more complicated.



Computations based on the gradually varied flow equation must be carried out to determine the location at the water surface attaches to the culvert crown (Point-P).

Then the headwater elevation is determined from the following equation :

$$HW = D + S_o L_p + H_e + H_f + H_v - Lso$$

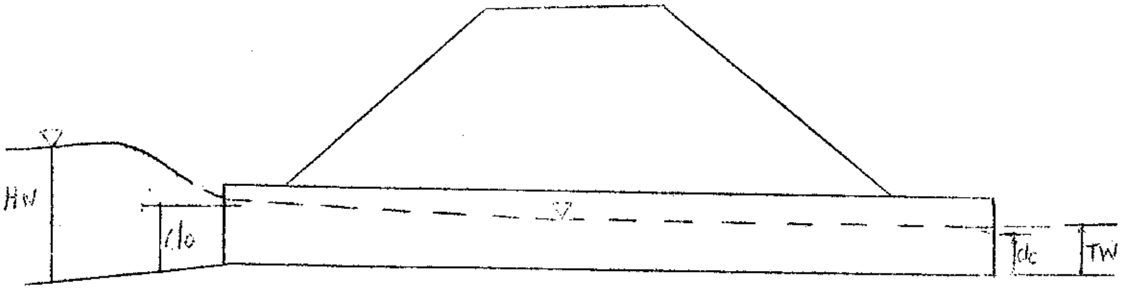
Where $H_e = K_c \frac{V^2}{2g}$

$$H_f = \frac{2g\ n^2\ L_f}{R^{4/3}} \frac{V^2}{2g}$$

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

NOTE : This HW includes the approach velocity head

7.10.3 If the headwater elevation is such that the entrance is free and the depth at outlet is either critical or controlled by TW, the headwater elevation is determined by using flow calculations are carried out for the entire length of the culvert, starting at d_c or TW, to determine the depth of flow at the inlet do. Then the headwater elevation simply.



$$HW = d_o + (1+K_e) \frac{Vo^2}{2g}$$

Vo and d_o are the velocity and depth at inlet, respectively.

7.11 INLET CONDITIONS

The inlet of a culvert provides a localized energy loss. If this inlet loss can be reduced, the headwater elevation for a given discharge can be lowered. Furthermore, with a culvert flowing part full, appropriate inlet modification may cause the barrel to flow full and hence allow the use of a smaller diameter barrel to carry the same discharge.

A list of the different type of inlets along with loss co-efficient is given in the following table.

TABLE 1 - ENTRANCE LOSS COEFFICIENT

Outlet Control, Full or Partly Full

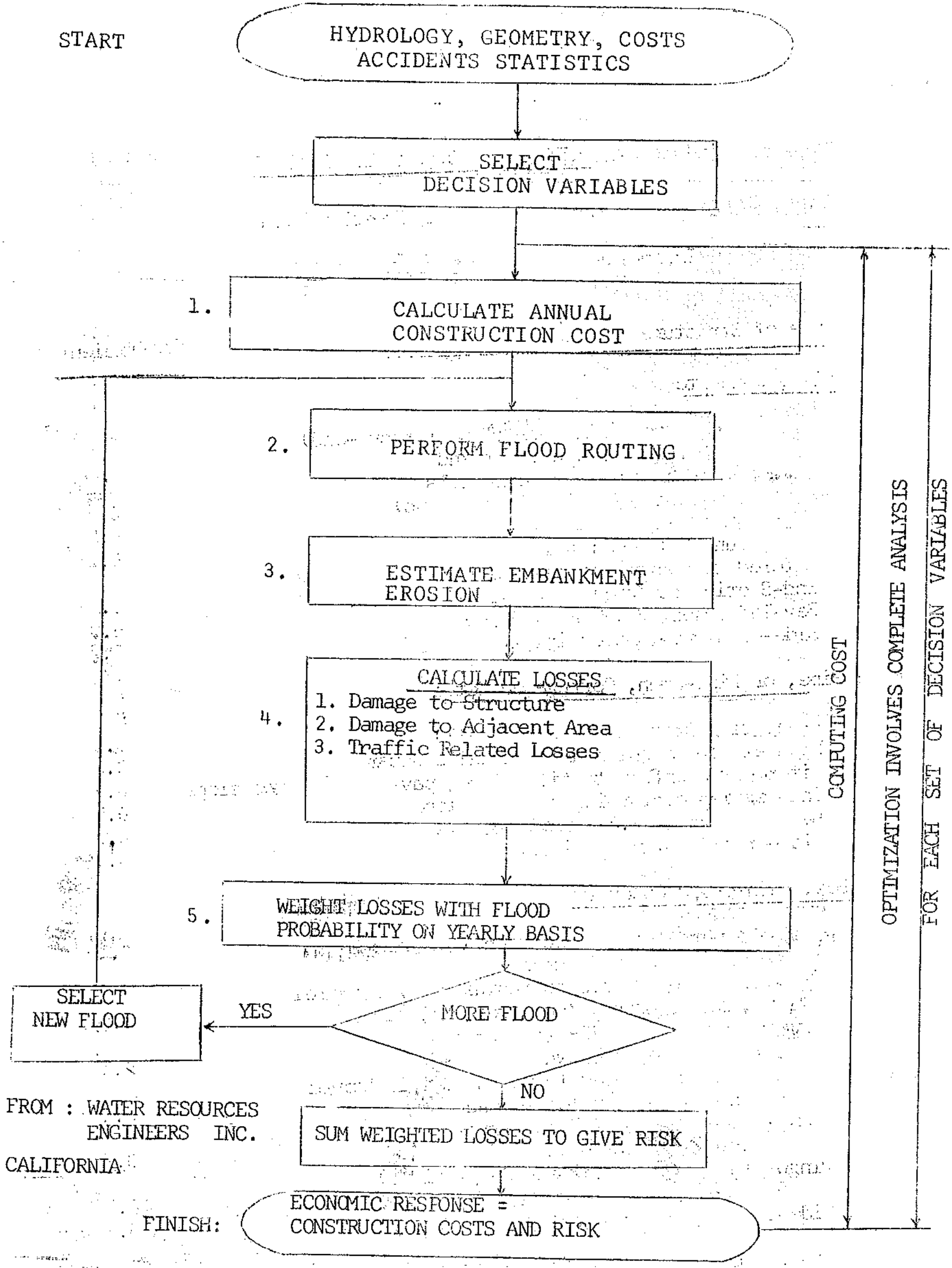
Entrance head loss $H_e = K_e \frac{V^2}{2g}$

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket and (groove-end)	0.2
Projecting 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
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment(no wingwalls)	0.5
Square-edged on 3 edges	
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of1/12 barrel dimension, or beveled top edge	0.2
Wingwalls 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

*Note : "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet, P. 5-13.

ANALYSIS PROCEDURE

EVALUATION OF THE FLOOD RISK FACTOR IN THE DESIGN OF BOX CULVERTS



FROM : WATER RESOURCES
ENGINEERS INC.
CALIFORNIA

SECTION - VIII

PROCEDURE FOR SELECTION OF CULVERT

EXAMPLE - 1

STEP-1 The roadbed and slope lines have been drawn and drainage divides are shown by heavy dashed lines, but other roadway details have been ommitted. (see fig 8.1)

The drainage channels are indicated by short lines with arrows to indicate direction of flow :

STEP-2 Measure the area draining towards culvert, crossing at station 50 + 000.

The area can be measured either by dividing the area into small sectors, and finding area of each sector and adding together or by planimeter or other means available. From Fig 8,1

		L (ft)		W (ft)			
Rolling Farm Land	=	42 Acres					
Side Slopes	=	1360	x	80	=	2.5	AC
Pavement	=	1360	x	12	=	0.37	AC
Shoulder	=	1360	x	8	=	0.25	AC

STEP-3 ESTIMATING RUNOFF

a) Compute Tc (Time of Concentration)

i) locate most remote point (longestic) along the principal drainage line, above the outlet at which flow is to be estimated. from fig: (8.1)

Highest point elevation = 151.3 H

Outlet elevation = 98.2 ft

Difference = 53.1 ft

FIG: 8.1 Scale : 1" = 200'

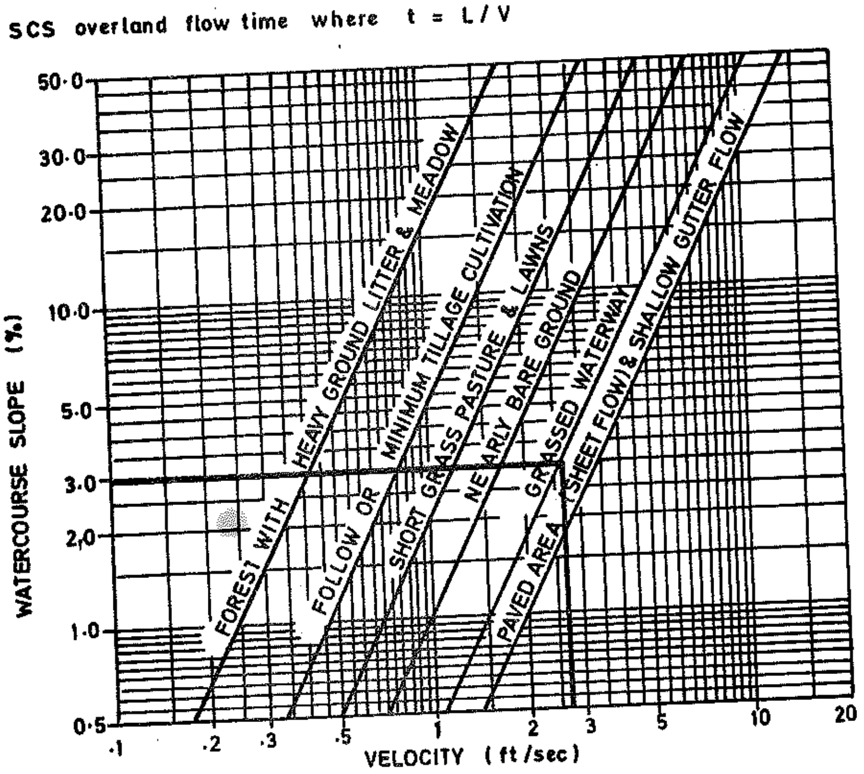


FIG: 8.2 - AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR OVERLAND FLOW (SCS METHOD)

RAINFALL INTENSITY DURATION FREQUENCY CURVE FOR CHANDPUR AND VICINITY (Assumed Town)

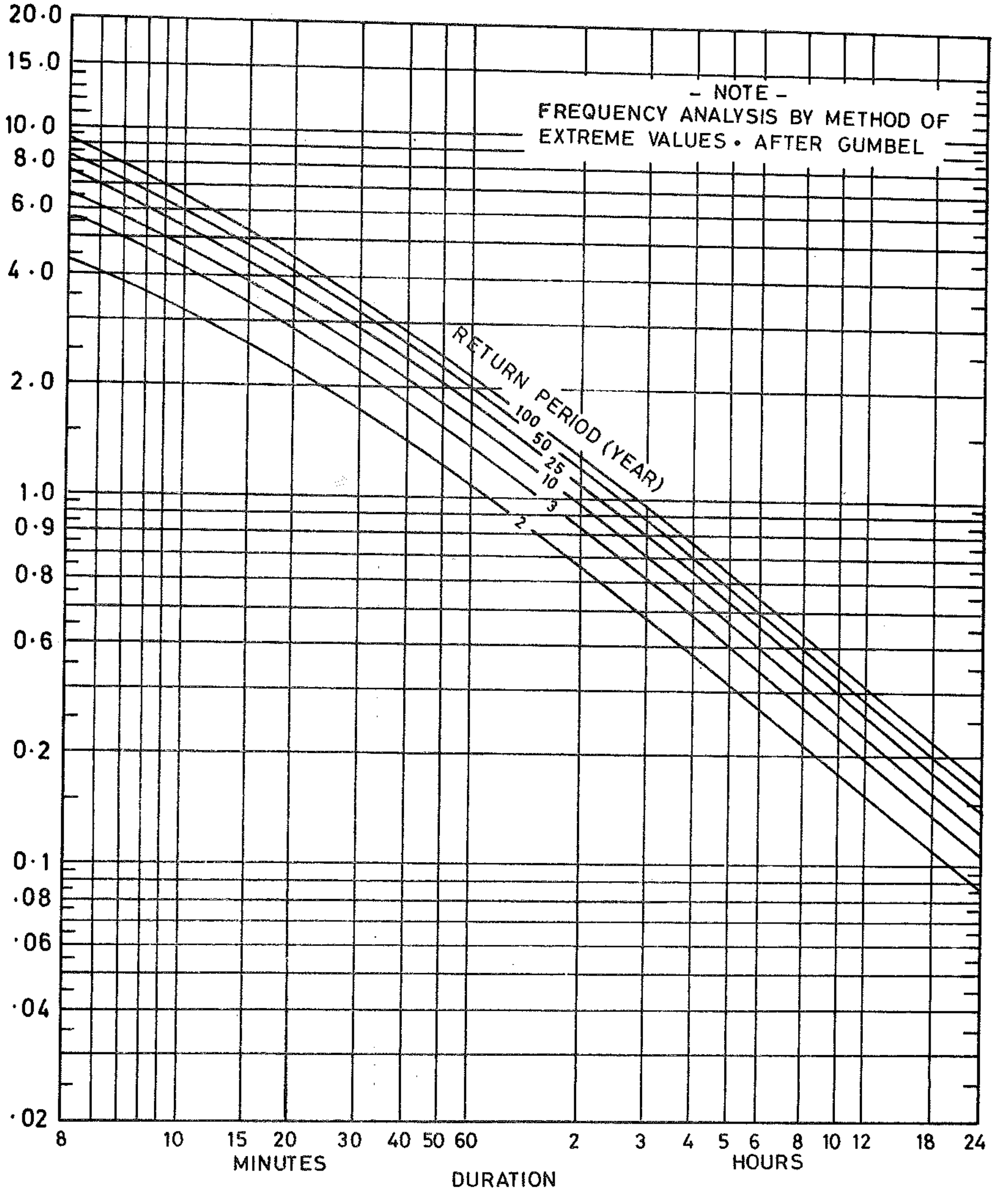


Fig : 8·3

STEP-4 Find the length from this point to outlet (From Fig 8.1)

$$L = 1950 \text{ ft}$$

Slope of Channel = $\frac{\text{difference in elevation}}{\text{Length}} = \frac{53.1}{1950} = .03$ feet/foot

STEP-5 Using Kirpich formula

$$T_c = 0.0078\ (1950)^{0.77} \times 0.03^{-0.385}$$

$$T_c = 10.67 \text{ Min} = 11 \text{ minutes}$$

This result can also be achieved by using nomograph shown in fig 4.3

(T_c) also can be obtained from fig (8.2) soil conservation service nomograph from fig (8.2) 3% slope

$V = 2.7$ ft/sec $L = 1950$ ft

$T_c = \frac{1950}{2.7} = 722$ sec = 12 min

Choose T_c = 12 minutes

NOTE: The values of T_c from fig (4.3) are based on meagre data, and should be used when better information are not available.

A minimum time of concentration of 5 minutes is recommended for finding the intensity used for estimating the design discharge. (More research is needed in this area to study variety of water-sheds (Location and Sizes) in different regions of Pakistan).

STEP-6 Assuming Rural Highway (2) shown in fig (8.1) passes nearby town of chandpur, (assumed location) then (i_{25}) = 5 in/hr from fig (8.3), IDF for assumed town of chandpur, using T_c = 12 minutes.

NOTE : Use frequency of 25 years for Rural Highway.

STEP-7 Find Weighted 'C' Runoff Co-efficient

Area	Runoff-Co-efficient Value from Table		CA
45	0.3	=	13.50
2.5	0.5	=	1.25
0.37	0.9	=	0.33
0.25	0.7	=	0.18
		CA	15.26
			≈ 15

STEP-8 Find Discharge

Using Rational Formula . $Q = C\ i\ A$

$CA = 15$

$i_{25} = 5$ in/hr

$Q_{25} = 15 \times 5 = 75$ cfs

Evaluate different sizes and kind of culverts and make selection according to the requirement.

Following are the recommendation for selection of culverts and Headwater.

Minimum size of culvert = 24" Pipe

Maximum allowable headwater

24" Pipe = 1.5 x diameters

30" " = Same

36 or large = 1.0 x diameter

OR Maximum allowable headwater 6" below sub-grade elevation; or whichever, is greater.

Minimum cover over pipe 1.5 ft from top of the pavement.

STEP-9 Federal Highway Administration. U.S. Department of Transportation uses approximation to the theoritical procedure described in previous section inorder to develop design charts (included in Appendix 'A').

Using this charts find the appropriate size of the culverts.

To determine headwater (HW), given Q, and size and type of culvert, FOR INLET CONTROL .

Use charts 1 through 7 in appendex 'A' and follow instructions.

For Outlet Control Nomograph

Use charts 8 through 14 in appendex 'A' and follow instructions.

SEE EXAMPLE = 1

PROJECT: HIGHWAY 2
CHAND PUR

DESIGNER: M . K .
DATE: MAY, 22 . 84.

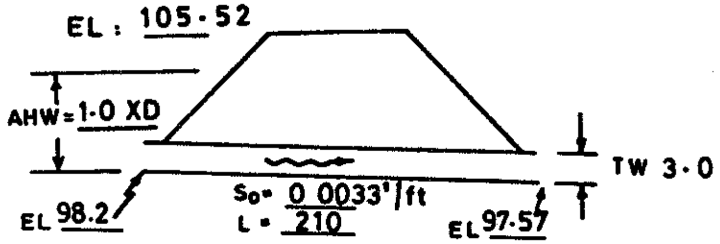
HYDROLOGIC AND CHANNEL INFORMATION

$Q_{1_{25}}$ = 75 cfs
Q_2 = ____
TW_1 = 3·0 ft
TW_2 = ____

(Q_1 = DESIGN DISCHARGE SAY Q_{25}
Q_2 = CHECK DISCHARGE SAY Q_{50} OR Q_{100})

SKETCH

STATION: 50.00



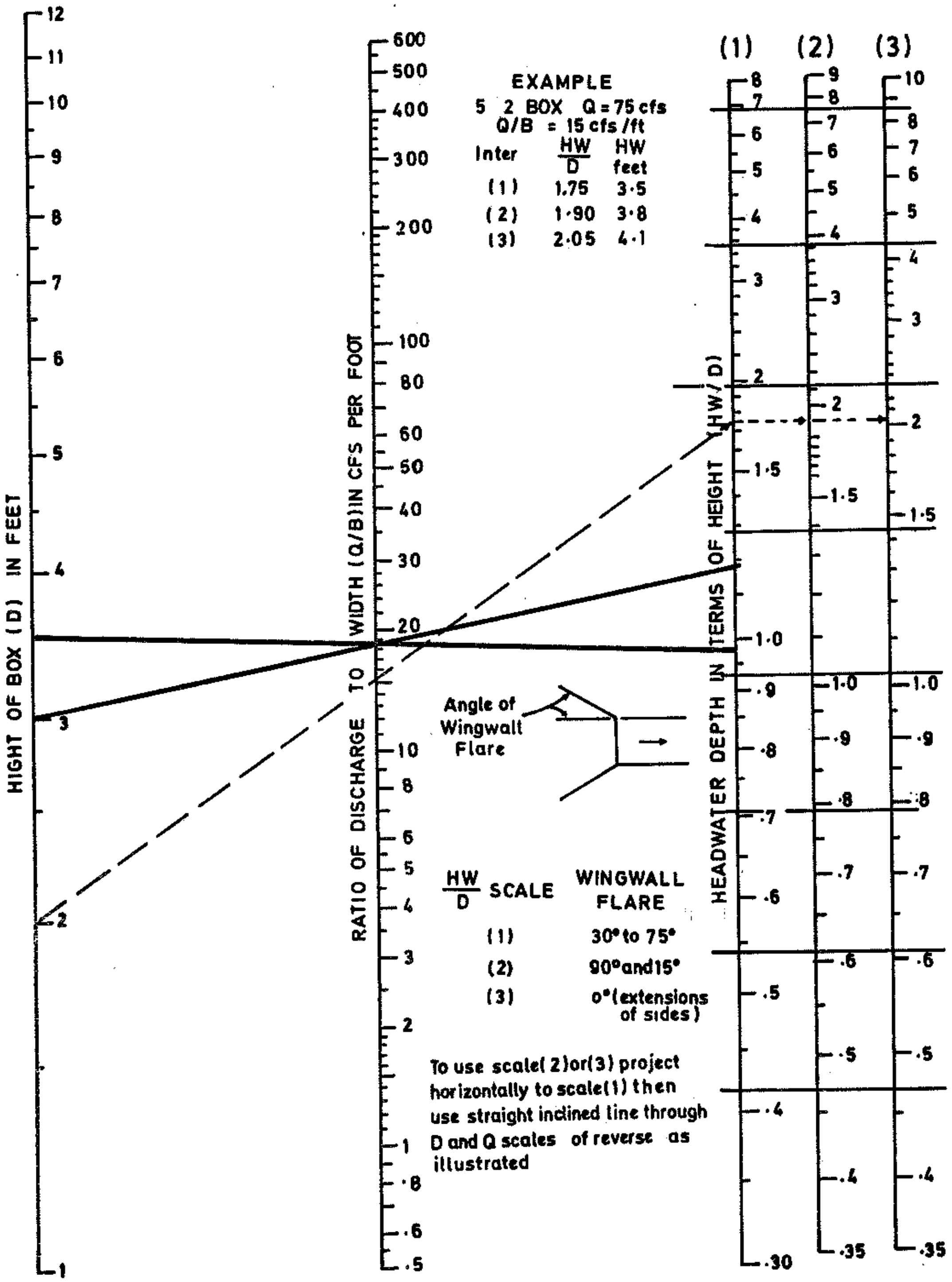
MEAN STREAM VELOCITY = ____
MAX. STREAM VELOCITY = ____

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q cfs	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL HW = H + h_o − LS_o											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
R.C. BOX 30° W.F.	75	4x3	1·18	3·54												HW > AHW
R.C. BOX 30° W.F.	75	4x3·5	0·97	3·40	0·5	1·18	2·2	2·9	3·0	3·0	0·7	3·48	3·48	5·4		HW ≤ AHW OUTLET CONTROL
GROOVE END WITH HW CONC. PIPE	75	48"	0·97	3·88	0·2	1·45	2·7	3·35	3·0	3·35	0·7	4·1	4·1	5·9		HW > AHW
G. END WITH H.W. CONC. PIPE	75	54"	0·82	3·69	0·2	0·8	2·5	3·5	3·0	3·5	0·7	3·6	3·69	4·8		HW < AHW INLET CONTROL

SUMMARY & RECOMMENDATIONS: SELECT CULVERT
(a) HYDRAULIC EFFICIENCY
(b) COST
(C) AVAILABILITY
(d) DURABILITY

FIGURE.7

EXAMPLE - 1



EXAMPLE

5 2 BOX Q = 75 cfs

Q/B = 15 cfs /ft

Inter	$\frac{HW}{D}$	HW feet
(1)	1.75	3·5
(2)	1·90	3·8
(3)	2·05	4·1

$\frac{HW}{D}$ SCALE	WINGWALL FLARE
(1)	30° to 75°
(2)	90° and 15°
(3)	o° (extensions of sides)

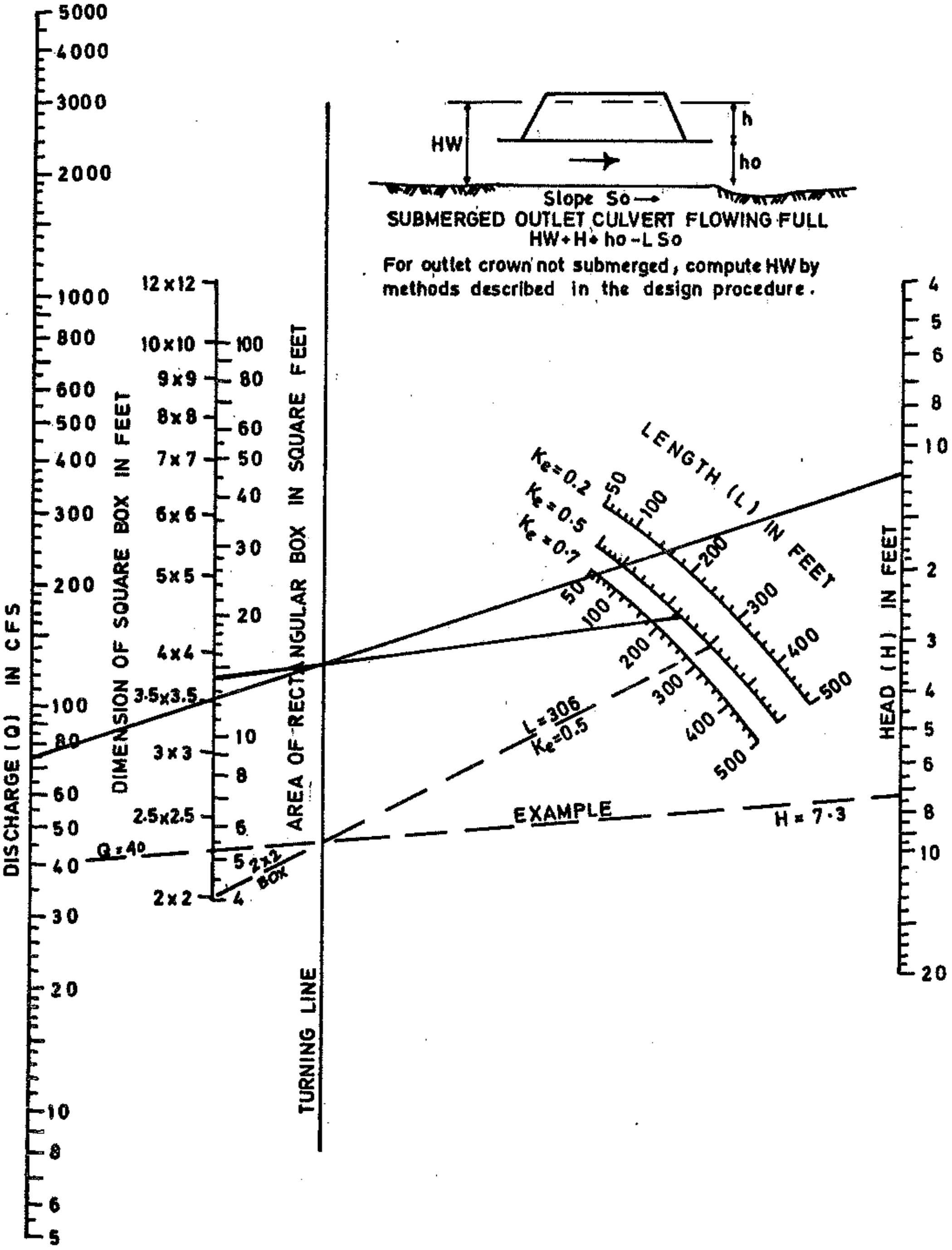
To use scale (2) or (3) project horizontally to scale (1) then use straight inclined line through D and Q scales of reverse as illustrated

HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN 1963

EXAMPLE - 1

CHART 8

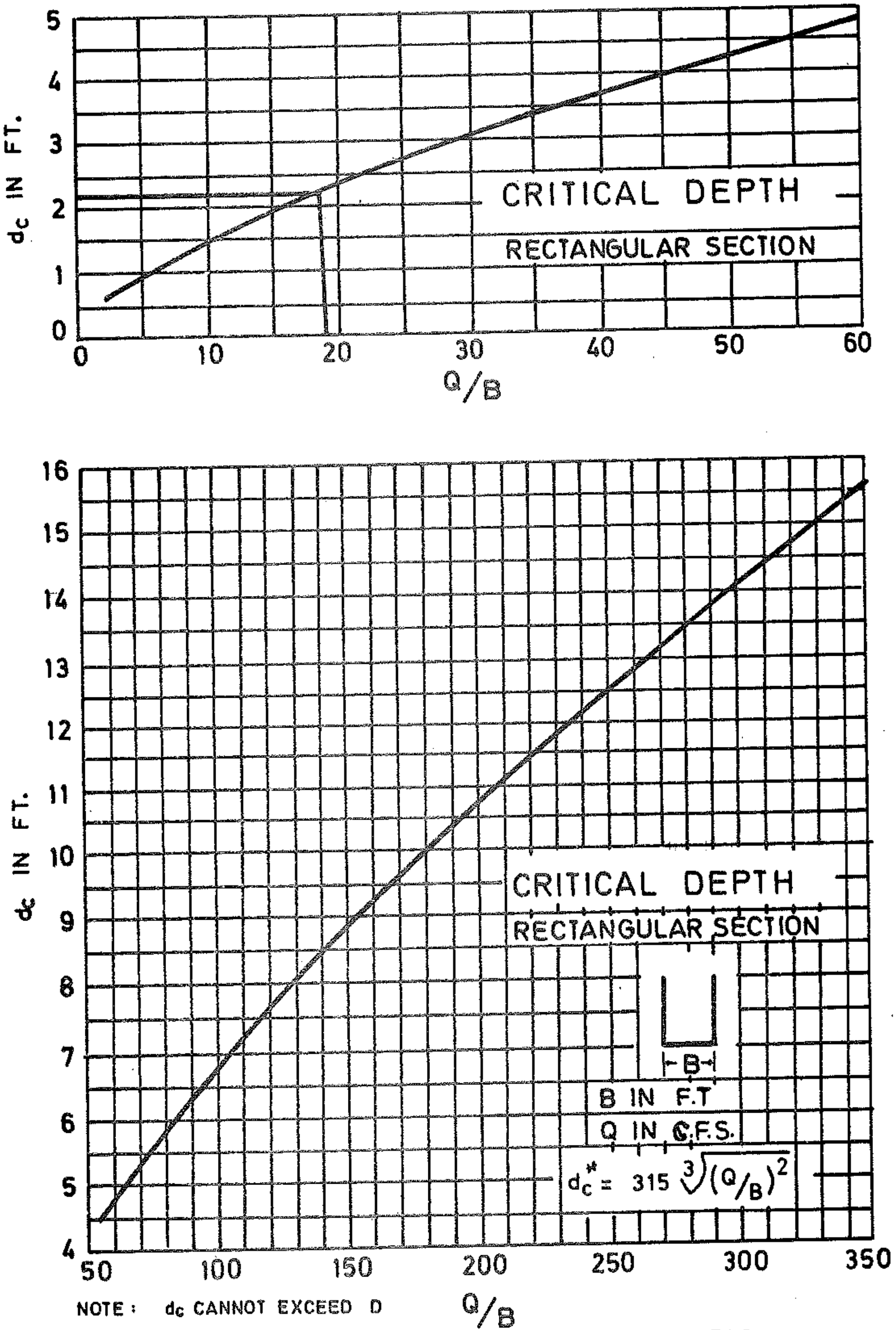


HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0·012

$$L_1 = 210 \begin{pmatrix} 0\cdot015 \\ 0\cdot012 \end{pmatrix}^2 = 328$$

EXAMPLE - 1

Chart 15

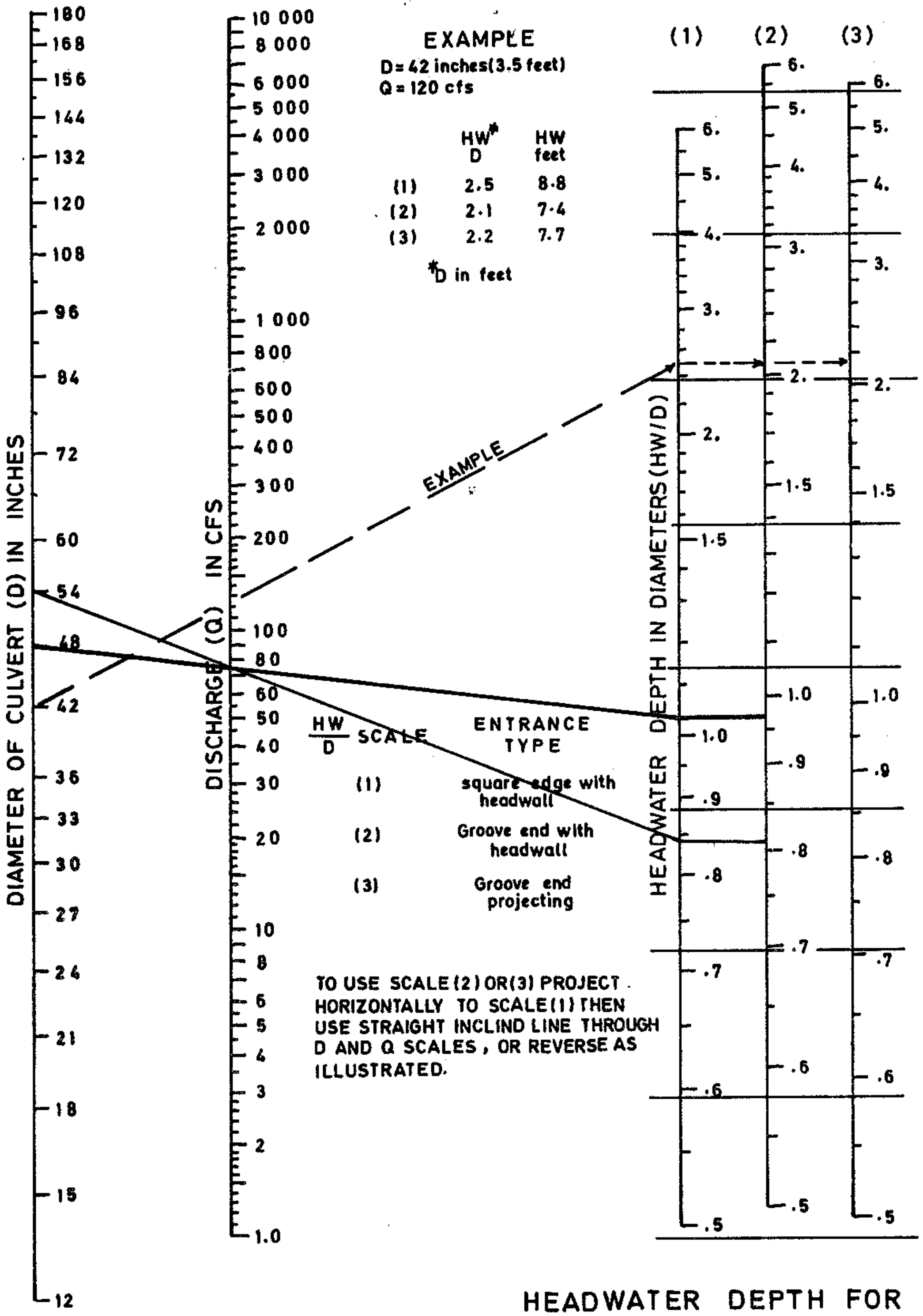


NOTE : d_c CANNOT EXCEED D

CRITICAL DEPTH
RECTANGULAR SECTION

EXAMPLE - 1

CHART 2



EXAMPLE

D = 42 inches (3.5 feet)
Q = 120 cfs

	$\frac{HW}{D}$*	HW feet
(1)	2.5	8.8
(2)	2.1	7.4
(3)	2.2	7.7

*D in feet

$\frac{HW}{D}$ SCALE	ENTRANCE TYPE
(1)	square edge with headwall
(2)	Groove end with headwall
(3)	Groove end projecting

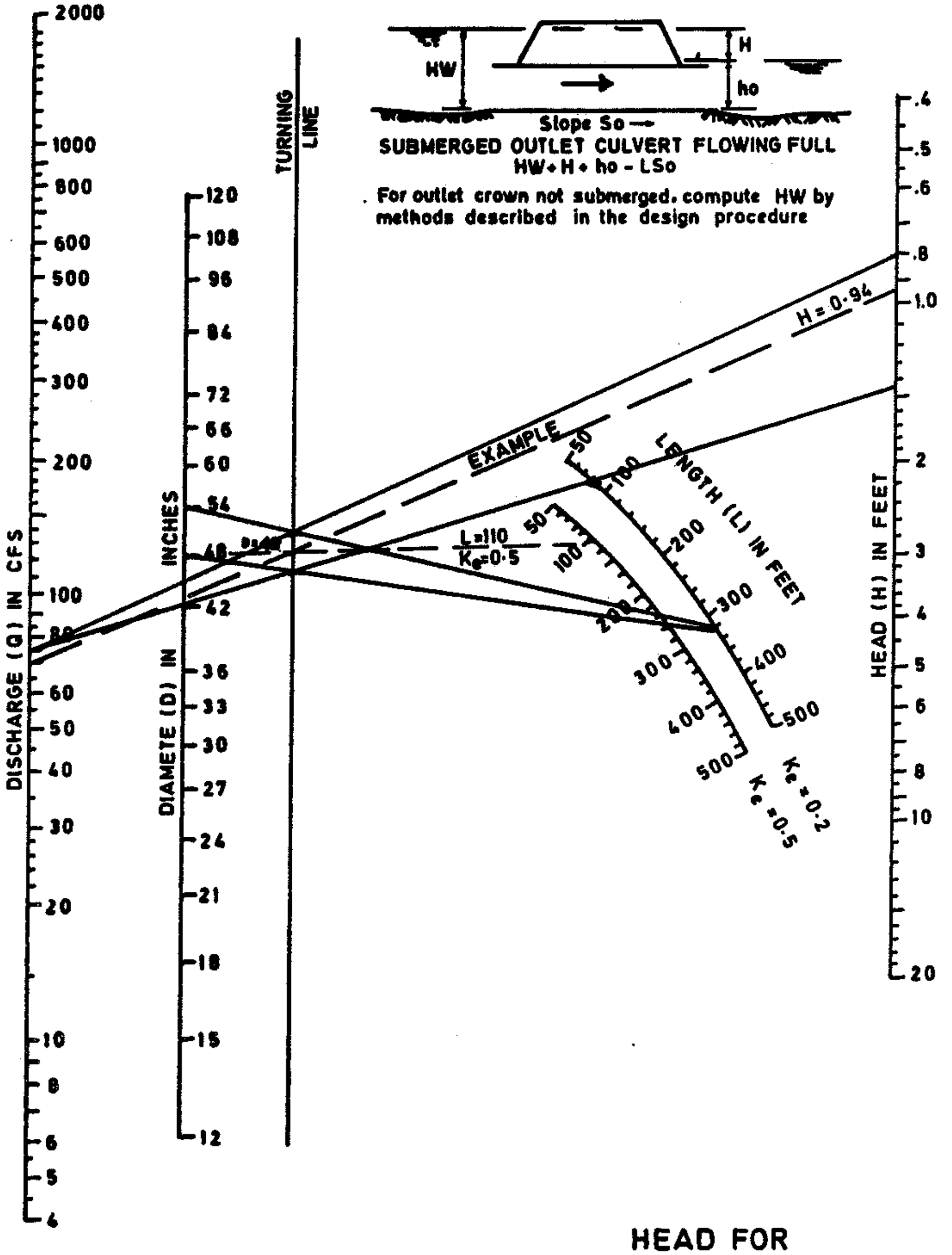
TO USE SCALE (2) OR (3) PROJECT HORIZONTALLY TO SCALE (1) THEN USE STRAIGHT INCLIND LINE THROUGH D AND Q SCALES, OR REVERSE AS ILLUSTRATED.

HEADWATER SCALES 2 & 3
REVISED MAY 1964

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

EXAMPLE - 1

CHART 9



SUBMERGED OUTLET CULVERT FLOWING FULL
HW+H+ho-LSo

. For outlet crown not submerged, compute HW by methods described in the design procedure

HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0·012

$$L_1 = L \times \left(\frac{h_1}{h}\right)^2$$

EXAMPLE = 1

$$210 \times \left(\frac{0\ 015}{0\ 012}\right)^2 = 328$$

[illegible]

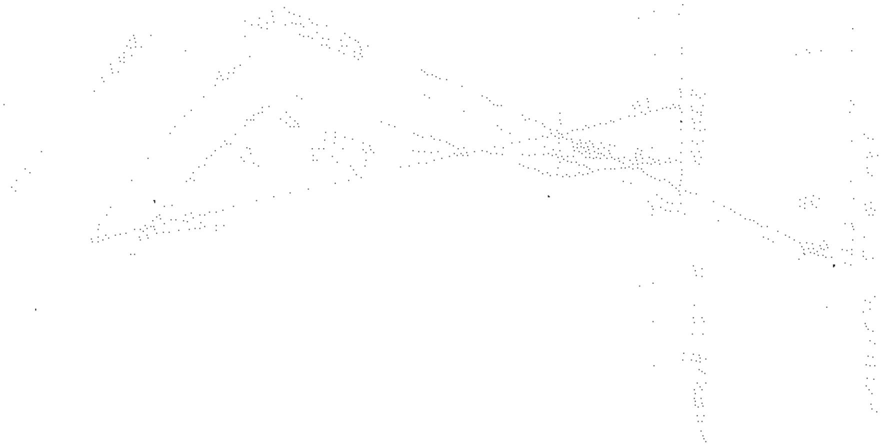
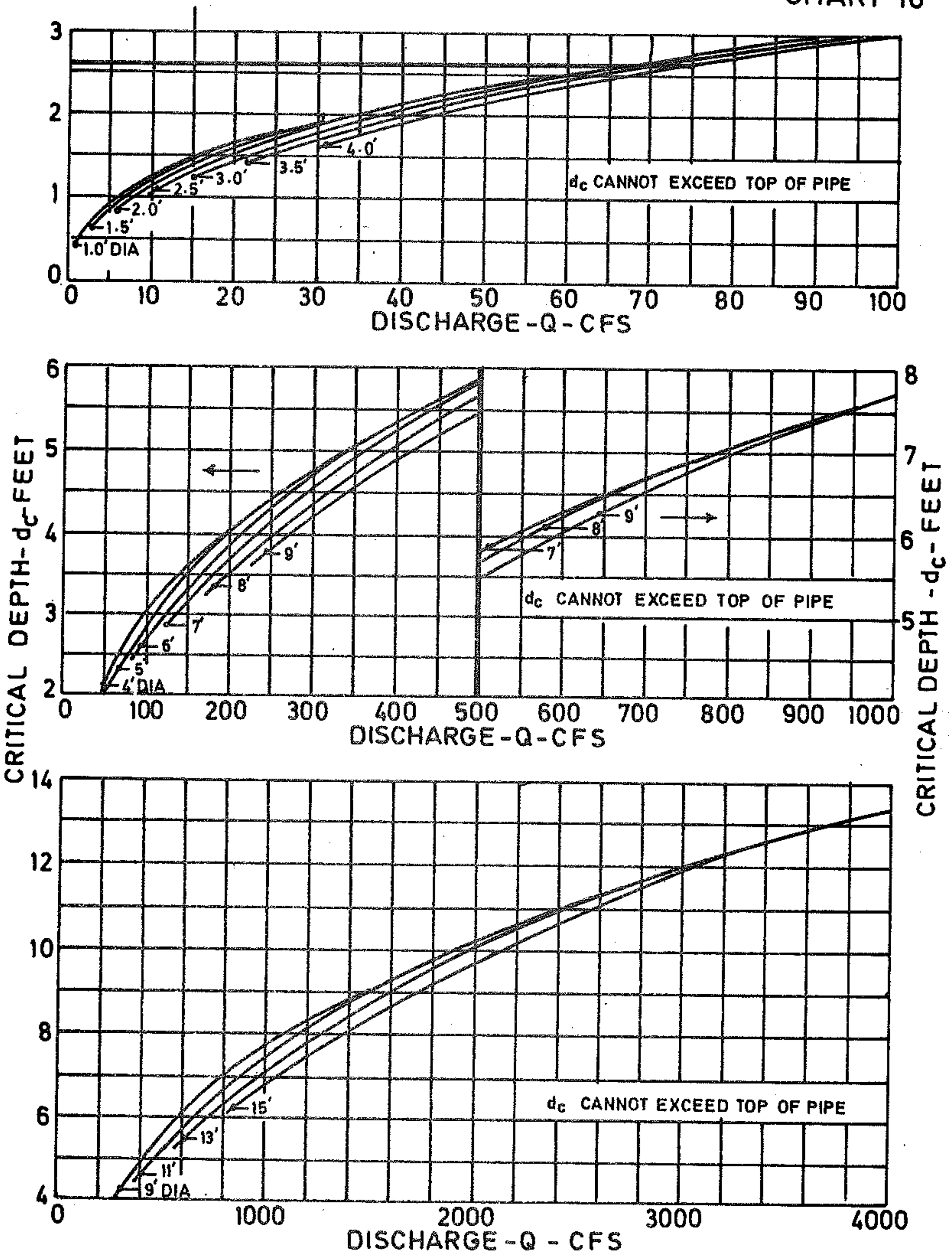


CHART 16



CRITICAL DEPTH
CIRCULAR PIPE

REFERENCES

(1) American Association of State Highway Officials, A Policy on Geometric Design of Rural Highways, Washington, D.C.

(2) Design of Roadside Drainage Channels Hydraulic Design Series No.4 U.S. Department of Transportation.

(3) ASCE, Hydrology Handbook, Manual of Engineering Practice, No. 28, 1949.

(4) Urban Hydrology for Small Watershed TR No. 55 Soil Conservation Service U.S. Department of Agriculture

(5) Beard, L.R(1962) Statistical Methods in Hydrology, U.S. Army Corps of Engineers, Sacramento, CA

(6) Cnow, V.T. (Editor) (1964) Handbook of Applied Hydrology Haan, C.T. (1977) Statistical Method in Hydrology, Iowa State University Press, Ames, Iowa.

(7) Linsley, R.K. Kohlar, M.A., Paulhus, J.L.H. (1975) Hydrology for Engineers.

(8) U.S. Weather Bureau (1955), Rainfall Intensity-Duration-Frequency Curve.

(9) Wanielista, M.P. (1978) Storm Water Management, Ann Arbor Science Publishers, Inc., Ann Arbor, MI.

(10) Concrete Pipe Handbook (1959), American Concrete Pipe Association, Chicago.

(11) Fair, G.M., Geyer, J.C. and Okun, D.A. (1966) Water Supply and Waste Removal volume 1, John Wiley and Sons, New York.

(12) Linsley, R.K. Kohler, M.A. Paulhus, J.L.H., (1975) Hydrology for Engineers., McGraw Hill, New York.

(13) Ragan, R.M. and Duro, J.O. (1972). Kinematic Wave Nomograph for Time of Concentration Journal of the Hydraulic Division, ASCE 98.

(14) Schaake, J.C., Geyer, J.C., Kanpp, J.W. (1967), Experimental Examination of Rational Method, Journal of Hydraulic Division, ASCE, 93 (HY6)

(15) Chow, V.T. (1962), Hydrologic Determination of Waterway Areas for the Design of Drainage Structures in Small Drainage Basin, University of Illinois.

(16) Tholin, A.L. and Kiefer, C.J. (1960), The Hydrology of Urban Runoff, Trans. ASCE.

(17) Federal Aviation Agency (1966), Airport Drainage, Advisory Circular ACNO/AC/50/5320-5A.

(18) Capacity Charts for the Hydraulics Design of Highway Culverts. U.S. Department of Transportation.

(19) Herr, L.A., Hydraulic Charts for the Selection of Highway Culverts. U.S. Department of Transportation

(20) Bureau of Reclamation, U.S. Department of Interior, Design of Small Dams. Washington D.C.

(21) Highway Research Board. Traffic Safe and Hydraulically Efficient Drainage Structure, NHRCP Synthesis Report No. 3. HRB Washington. D.C.

(22) ASSHTO, An Informational Guide for Physical Maintenance, Washington, D.C.

APPENDIX ' A '

U.S. DEPARTMENT TRANSPORTATION
NOMOGRAPHS FOR SELECTION OF
HIGHWAY CULVERTS

INLET-CONTROL NOMOGRAPHS

Charts 1 through 7

Instructions for Use

1. To determine headwater (HW), given Q, and size and type of culvert.

   a. Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or $\frac{Q}{B}$ for box culverts; mark intersection of straightedge on $\frac{HW}{D}$ scale marked (1).

   b. If $\frac{HW}{D}$ scale marked ( ) represents entrance type used, read $\frac{HW}{D}$ on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read $\frac{HW}{D}$.

   c. Compute HW by multiplying $\frac{HW}{D}$ by D.

2. To determine discharge(Q) per barrel, given HW, and size and type of culvert.

   a. Compute $\frac{HW}{D}$ for given conditions.

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ scale (1) as found in (b) above and the size of culvert on the left scale. Read Q or $\frac{Q}{B}$ on the discharge scale.

   d. If $\frac{Q}{B}$ is read in (c) multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, allowable HW and type of culvert.

   a. Using a trail size, compute $\frac{HW}{D}$

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ scale (1) as found in (b) above to given discharge and read diameter,height or size of culvert required for $\frac{HW}{D}$ value.

   d. If D is not that originally assumed, repeat procedure with a new D.

[illegible]

[illegible]

1. To [illegible]

a. Describe [illegible]

[illegible]

2. To [illegible]

3. To formulate the [illegible]

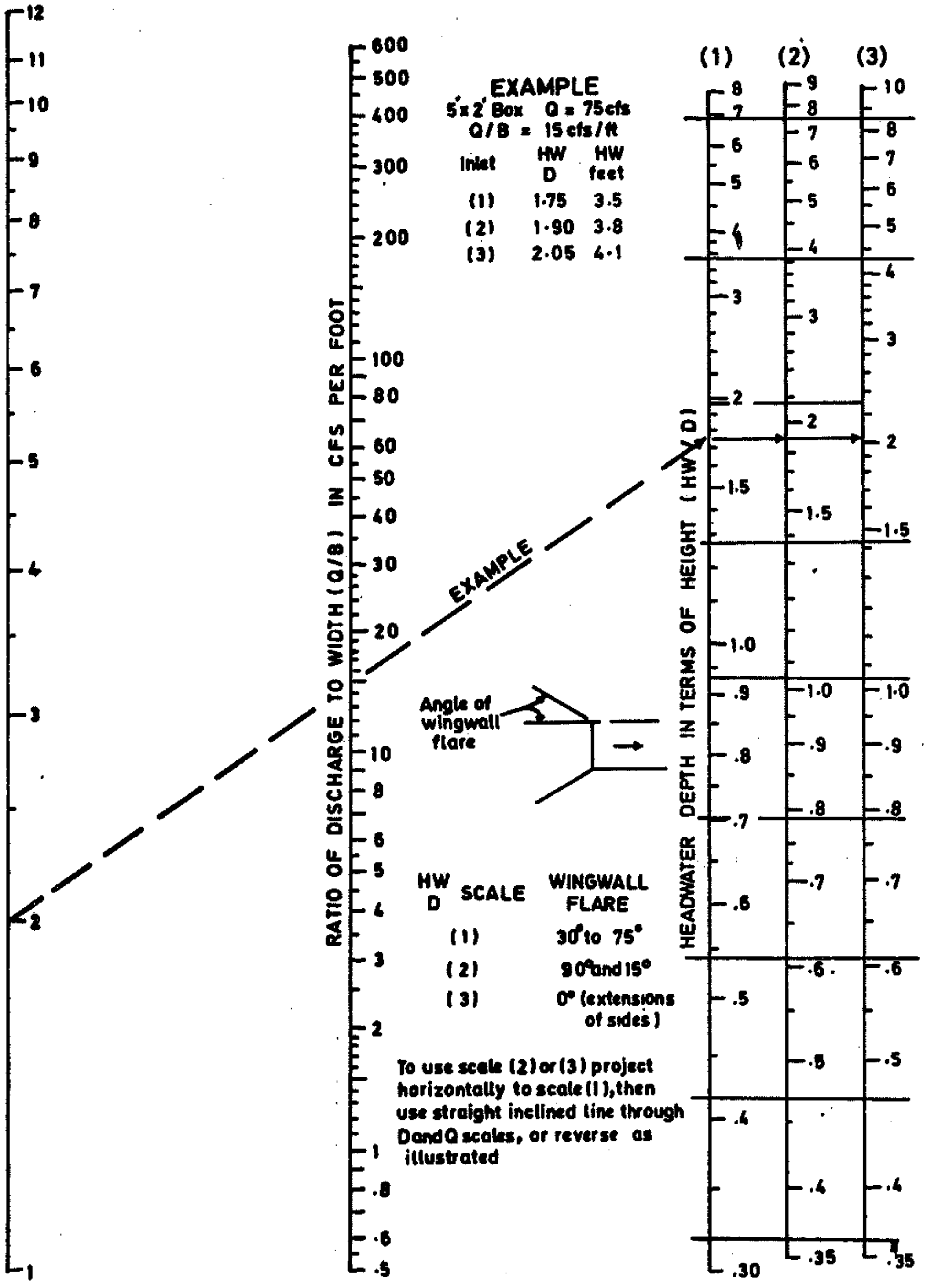
a. Using [illegible]

b. [illegible]

c. [illegible]

d. [illegible]

CHART 1



EXAMPLE

5'x 2' Box Q = 75cfs
Q/B = 15 cfs/ft

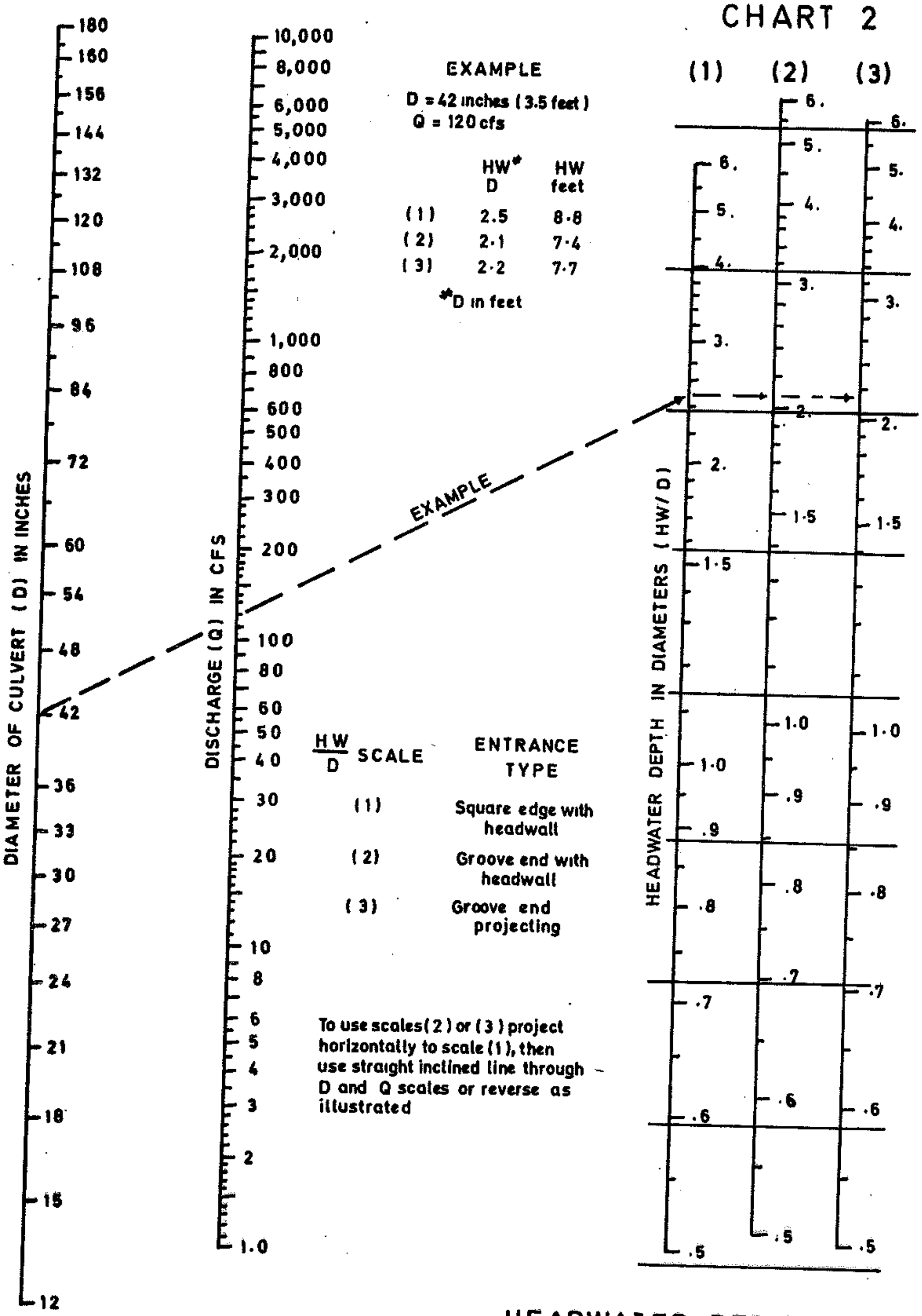
Inlet	HW/D	HW feet
(1)	1.75	3.5
(2)	1.90	3.8
(3)	2.05	4.1

HW/D SCALE	WINGWALL FLARE
(1)	30° to 75°
(2)	90° and 15°
(3)	0° (extensions of sides)

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated

HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

CHART 2



EXAMPLE

D = 42 inches (3.5 feet)
Q = 120 cfs

	HW*/D	HW feet
(1)	2.5	8·8
(2)	2·1	7·4
(3)	2·2	7·7

*D in feet

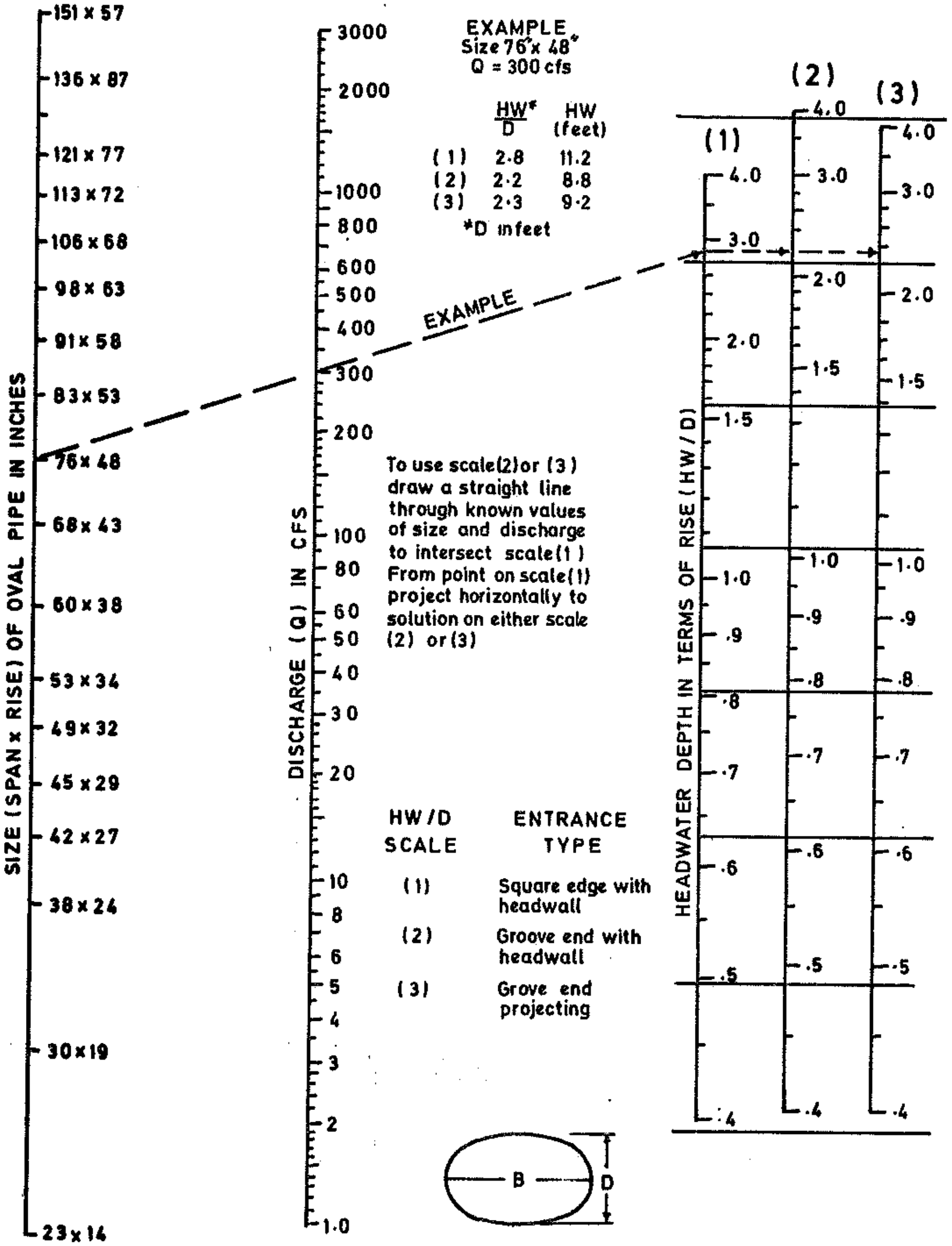
HW/D SCALE	ENTRANCE TYPE
(1)	Square edge with headwall
(2)	Groove end with headwall
(3)	Groove end projecting

To use scales (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales or reverse as illustrated

HEADWATER SCALES 2&3
REVISED MAY 1964

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

CHART 3



EXAMPLE
Size 76" x 48"
Q = 300 cfs

	HW*/D	HW (feet)
(1)	2.8	11.2
(2)	2.2	8.8
(3)	2.3	9.2

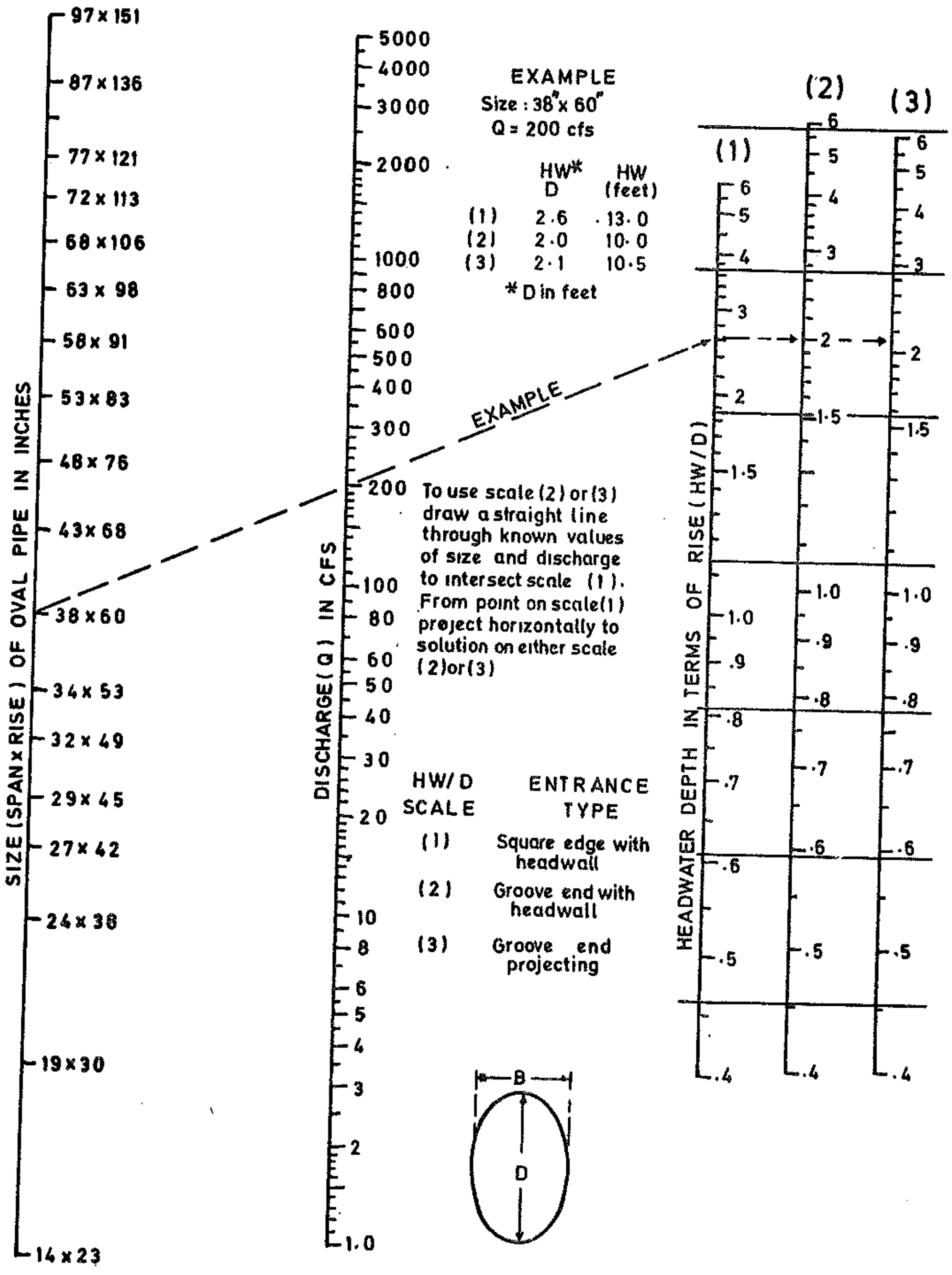
*D in feet

To use scale(2) or (3) draw a straight line through known values of size and discharge to intersect scale(1) From point on scale(1) project horizontally to solution on either scale (2) or (3)

HW/D SCALE	ENTRANCE TYPE
(1)	Square edge with headwall
(2)	Groove end with headwall
(3)	Grove end projecting

HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL WITH INLET CONTROL

CHART 4



EXAMPLE
Size : 38"x 60"
Q = 200 cfs

	HW*/D	HW (feet)
(1)	2.6	13.0
(2)	2.0	10.0
(3)	2.1	10.5

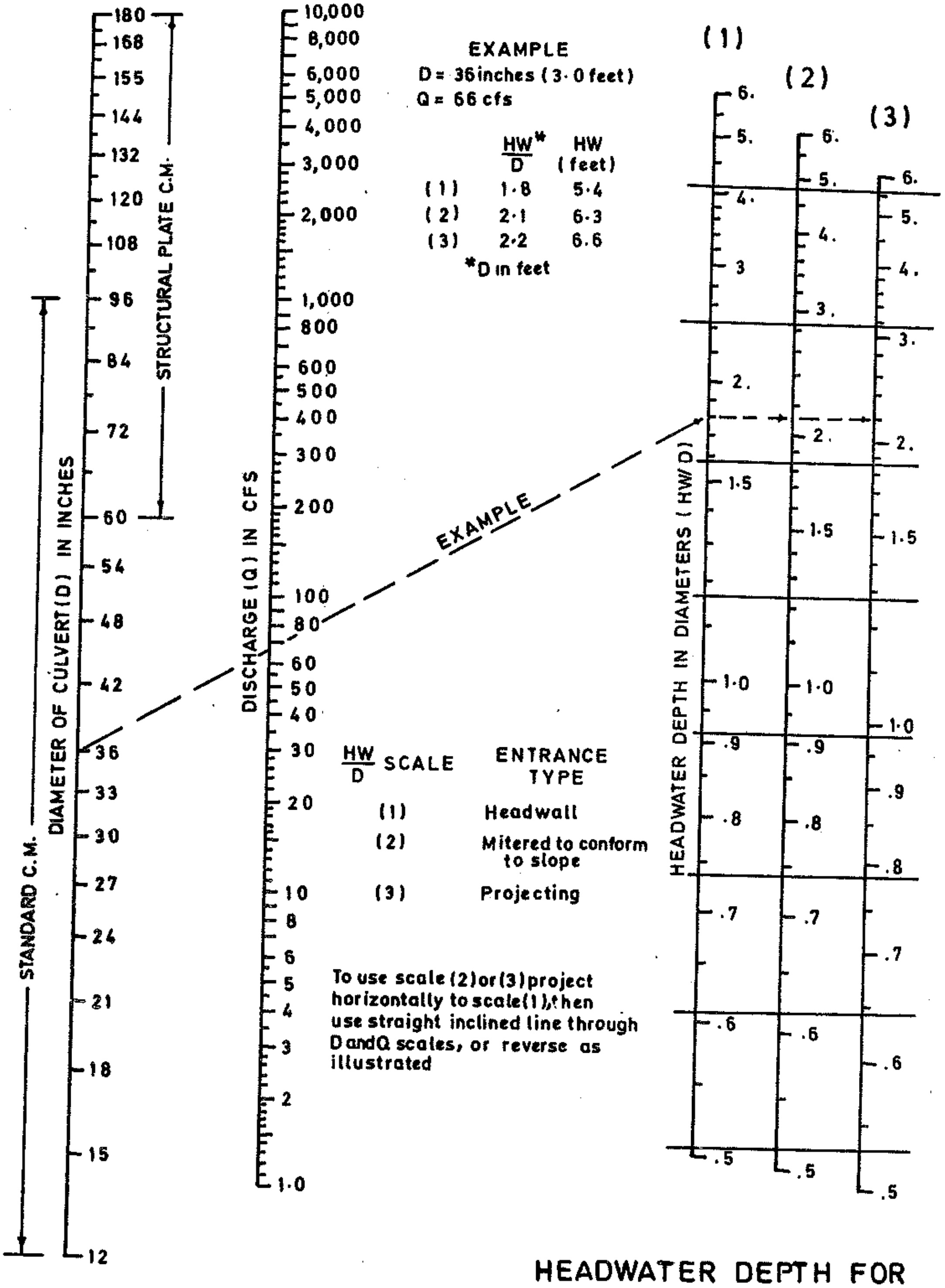
* D in feet

To use scale (2) or (3) draw a straight line through known values of size and discharge to intersect scale (1). From point on scale (1) project horizontally to solution on either scale (2) or (3)

HW/D SCALE	ENTRANCE TYPE
(1)	Square edge with headwall
(2)	Groove end with headwall
(3)	Groove end projecting

HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

CHART 5



EXAMPLE
D = 36inches (3·0 feet)
Q = 66 cfs

	$\frac{HW}{D}$*	HW (feet)
(1)	1·8	5·4
(2)	2·1	6·3
(3)	2·2	6.6

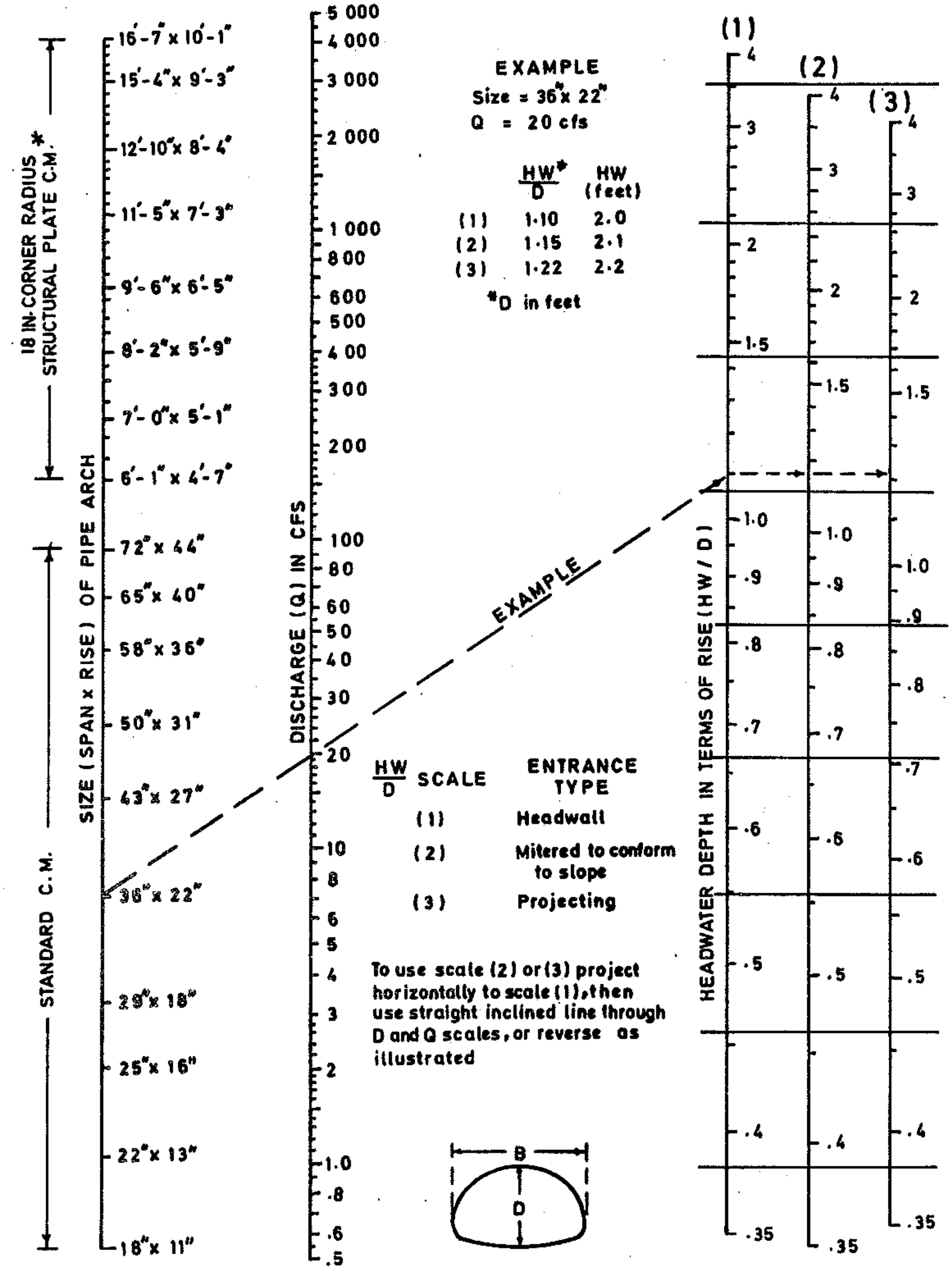
*D in feet

$\frac{HW}{D}$ SCALE	ENTRANCE TYPE
(1)	Headwall
(2)	Mitered to conform to slope
(3)	Projecting

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated

HEADWATER DEPTH FOR
C M PIPE CULVERTS
WITH INLET CONTROL

CHART 6

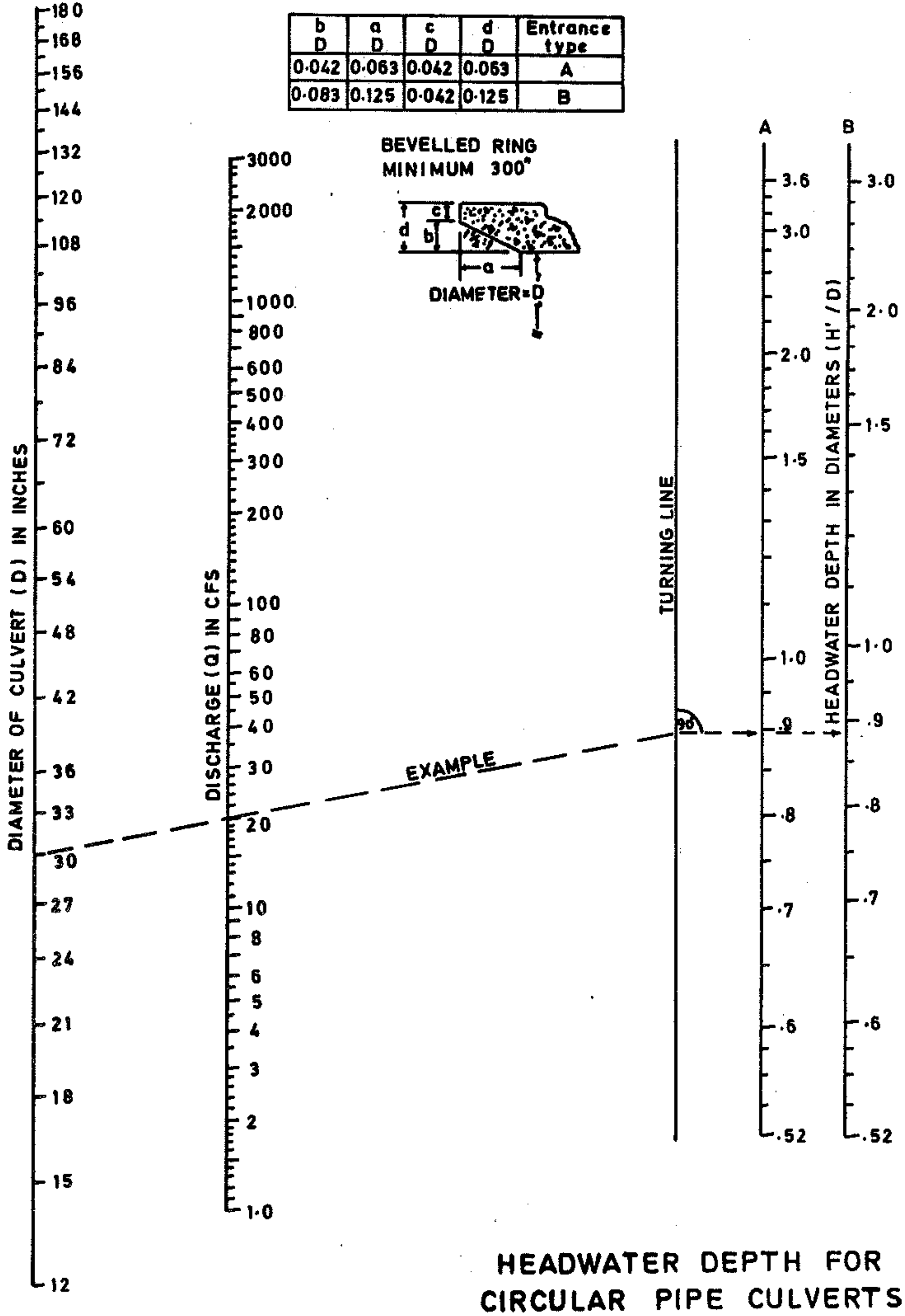


ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATORS CATALOG

HEADWATER DEPTH FOR
C. M. PIPE-ARCH CULVERTS
WITH INLET CONTROL

Chart 7

b/D	a/D	c/D	d/D	Entrance type
0·042	0·063	0·042	0·063	A
0·083	0.125	0·042	0·125	B



HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

OUTLET-CONTROL NOMOGRAPHS

Charts 8 through 14

Instructions for Use

Outlet control nomographs solve equation 2, p.5-6, for head H the culvert barrel flows full for its entire length. They are used to determine head H for some part-full flow conditions with a control. These nomographs do not give a complete solution for kg headwater HW, since they only give H in equation 3, $HW = H + h_o - LS_o$ discussion for "Culverts Flowing with Outlet Control", P.5-5.)

determine head H for a given culvert and discharge Q.

Locate appropriate nomograph for type of culvert selected. Find K_e for entrance type in Appendix B, Table 1, p.5-49.

Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales follow instruction below:

(1) If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper k_e and locate the starting point at the given culvert length. If a k_e curve is not shown for the selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.

(2) 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.

(3) For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula

$$L_1 = L \left[\frac{n_1}{n}\right]^2$$

See instruction 2 for n values.

Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line". See instruction 3 below for size considerations for rectangular box culvert.

Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving equation 2, p.5-6.

2. Values of n for commonly used culvert materials.

Concrete

Pipe	Boxes
0.012	0.012

Corrugated Metal

	Small Corrugations (2 2/3" x 1/2")	Medium Corrugations (3" x 1")	Large Corrugations (6" x 2")
Unpaved	0.024	0.027	Varies*
25% paved	0.021	0.023	0.026
Fully paved	0.012	0.012	0.012

*Variation in n with diameter shown on charts. The various n values have been incorporated into the nomographs and no adjustment for culvert length is required as instructed in 1b(3).

3. To use the box culvert nomograph, chart 8, for full-flow for other than square boxes.

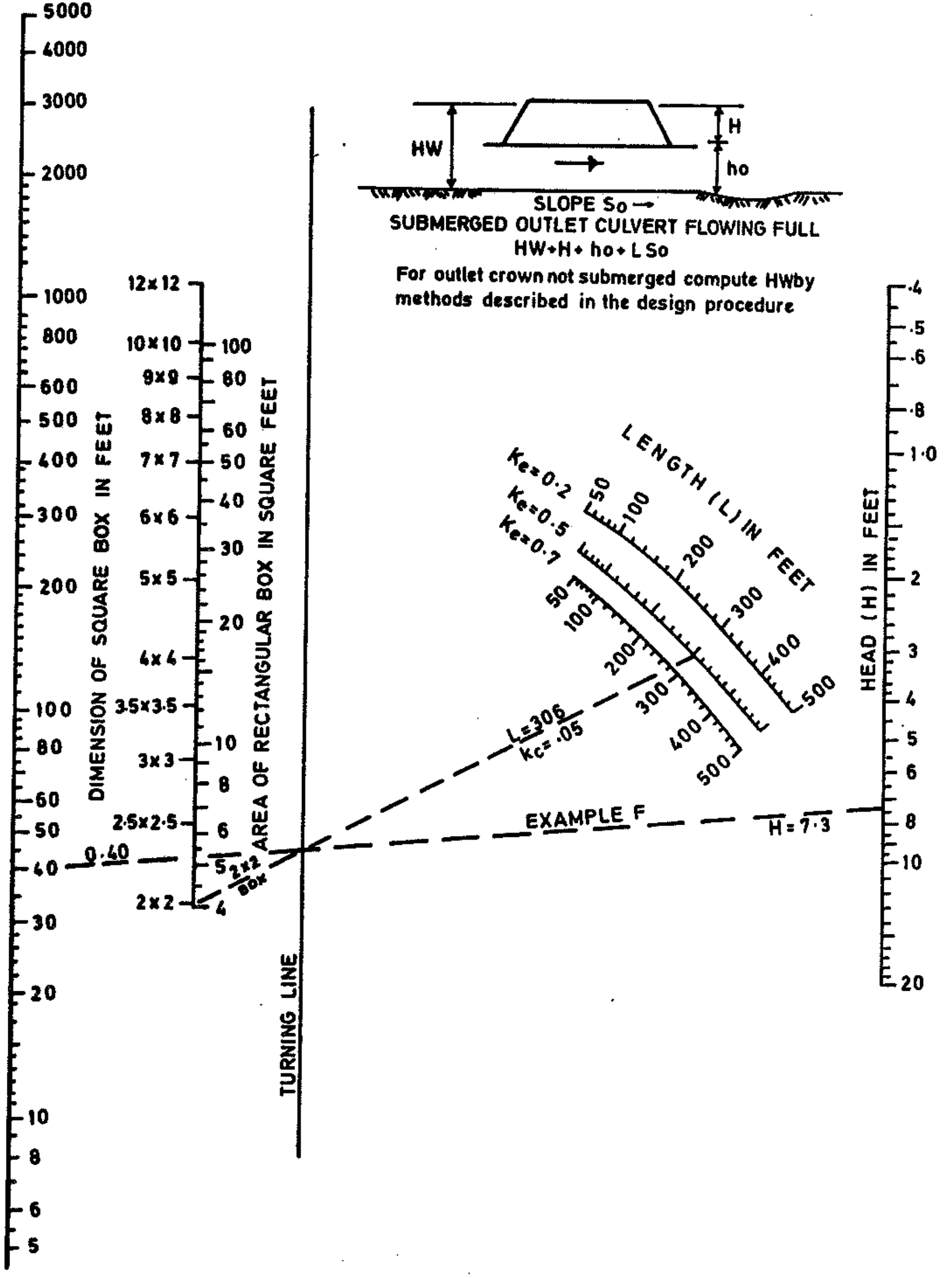
a. Compute cross-sectional area of the rectangular box.

b. Connect proper point (see instruction 1) on length scale to barrel area 3/ and mark point on turning line.

c. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (II) scale.

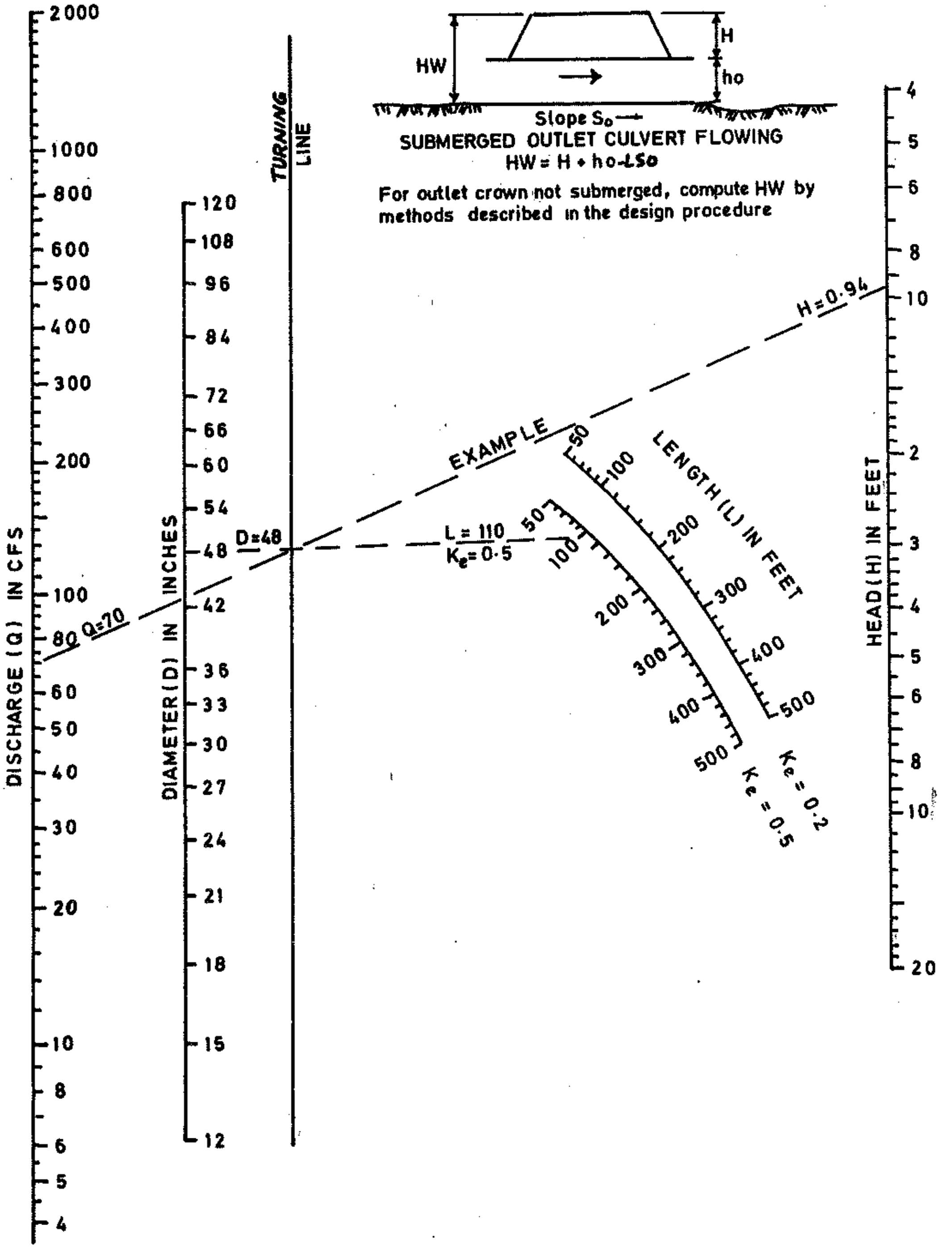
3/ The area scale on the nomograph is calculated for barrel cross sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions use equation 2 for more accurate results.

CHART 6



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0·012

CHART 9

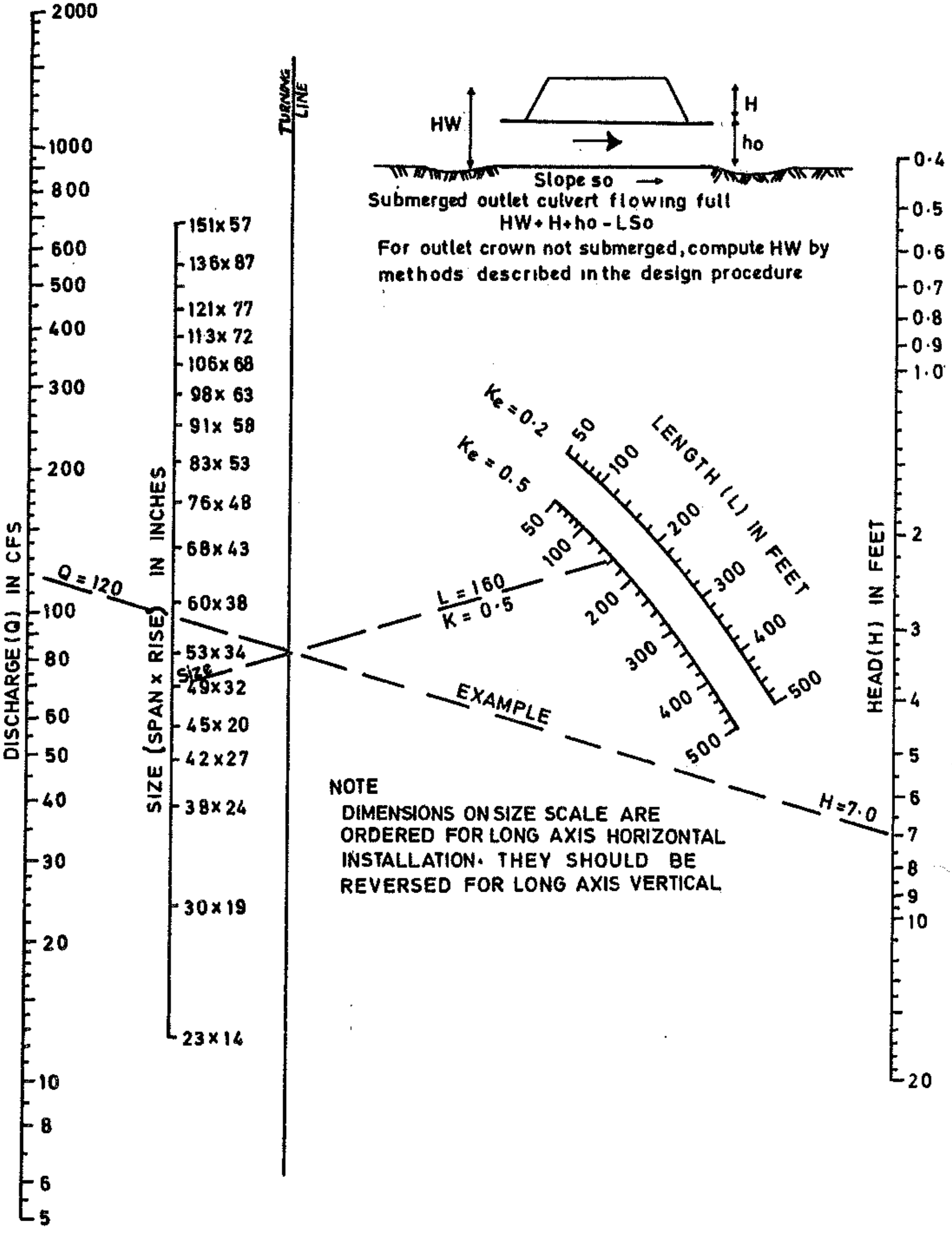


SUBMERGED OUTLET CULVERT FLOWING
HW = H + ho-LSo

For outlet crown not submerged, compute HW by methods described in the design procedure

HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0·012

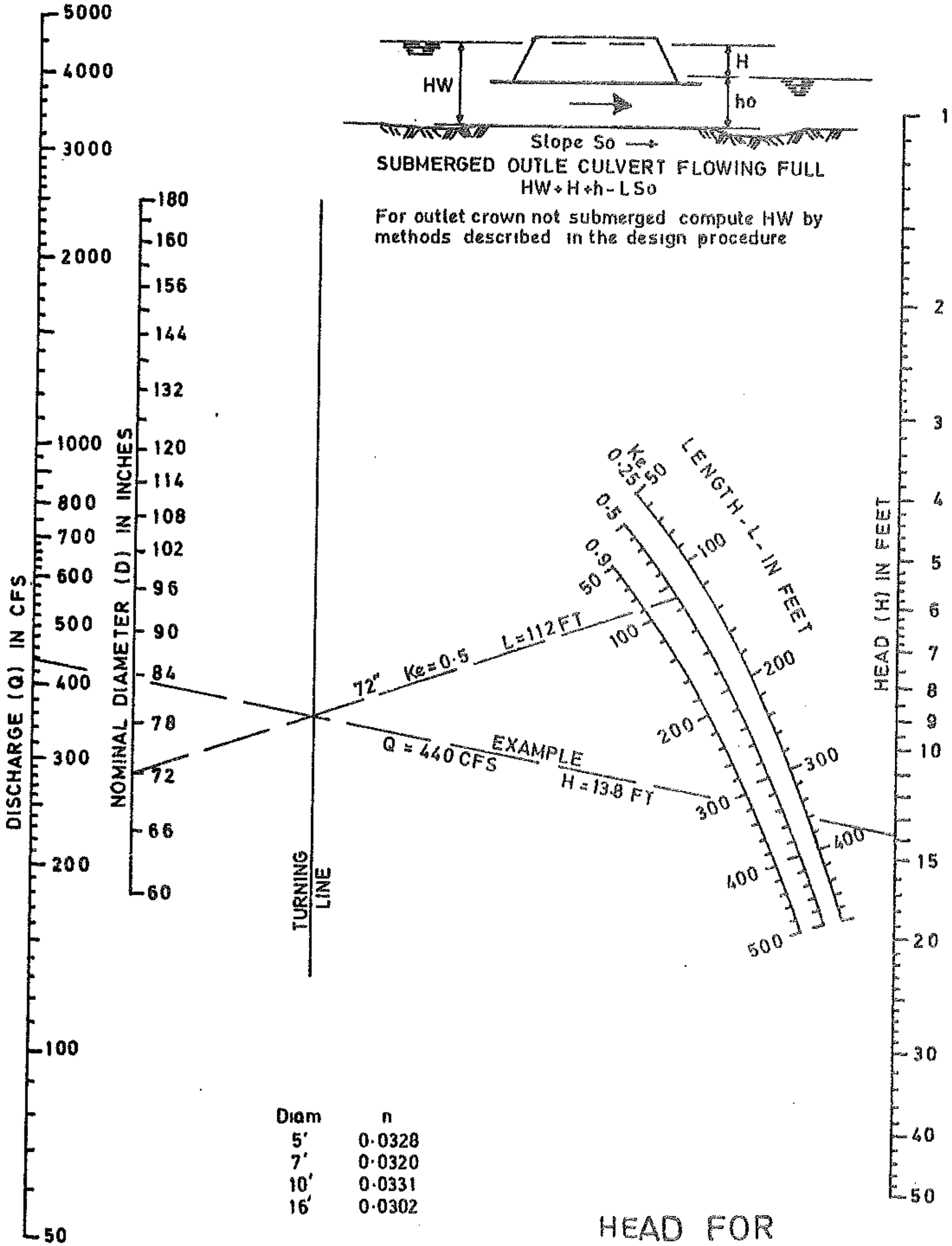
CHART 10



NOTE
DIMENSIONS ON SIZE SCALE ARE ORDERED FOR LONG AXIS HORIZONTAL INSTALLATION. THEY SHOULD BE REVERSED FOR LONG AXIS VERTICAL

HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
n= 0.012

CHART 13



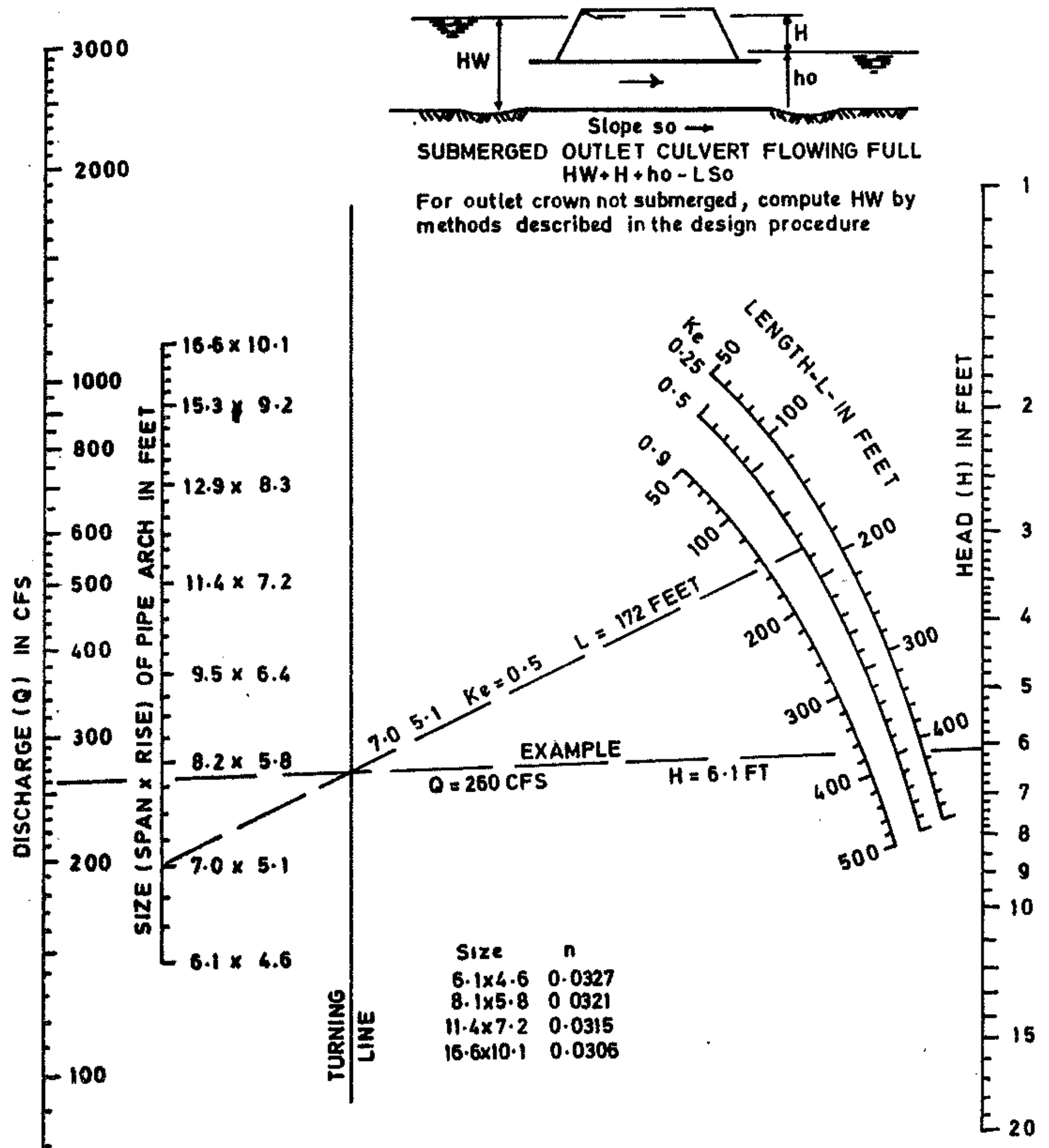
SUBMERGED OUTLE CULVERT FLOWING FULL
HW+H+h-LSo

For outlet crown not submerged compute HW by methods described in the design procedure

Diam	n
5'	0·0328
7'	0·0320
10'	0·0331
16'	0·0302

HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0·0328 TO 0·0302

CHART 14

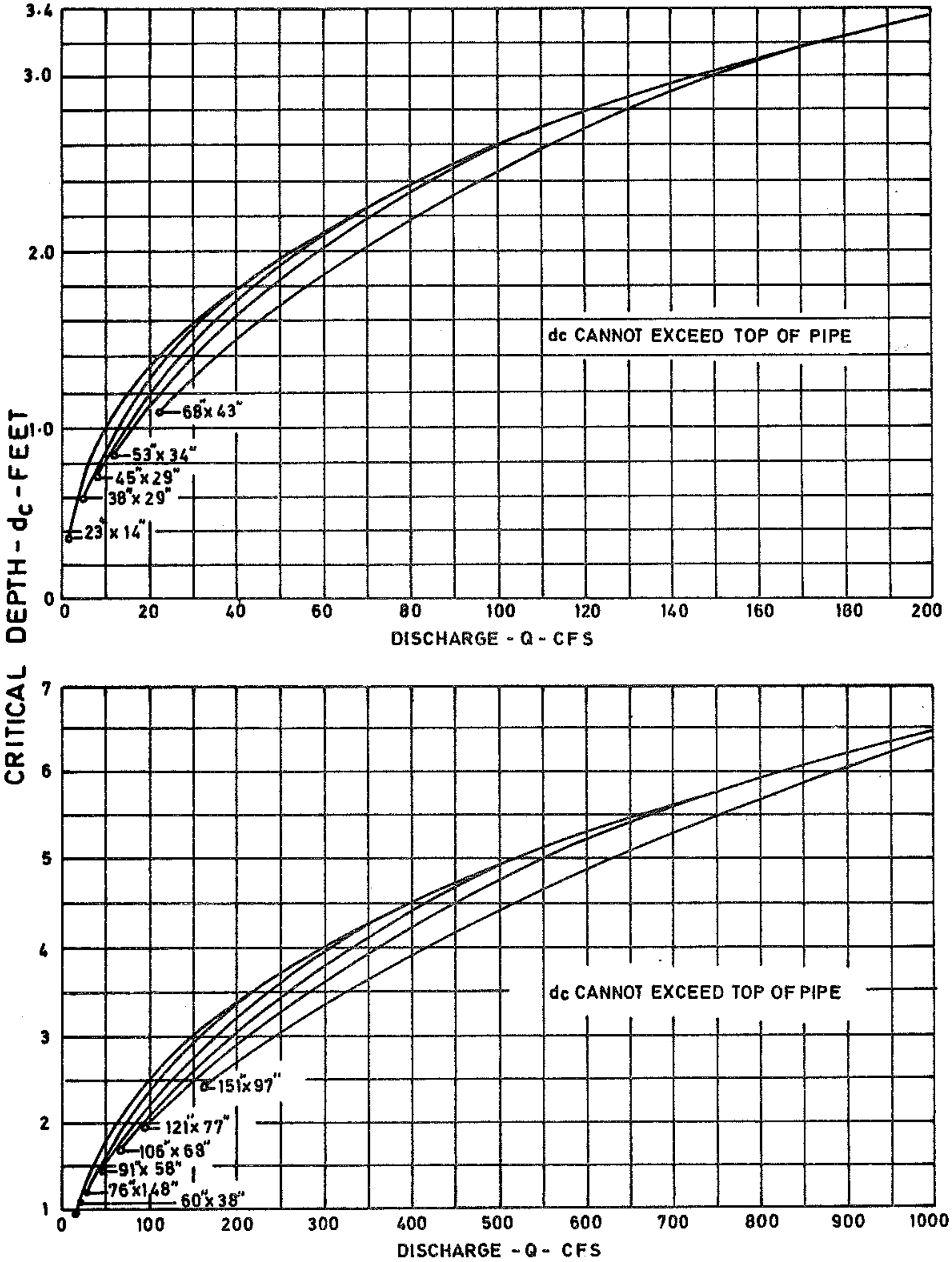


Size	n
6·1x4·6	0·0327
8·1x5·8	0 0321
11·4x7·2	0·0315
16·6x10·1	0·0306

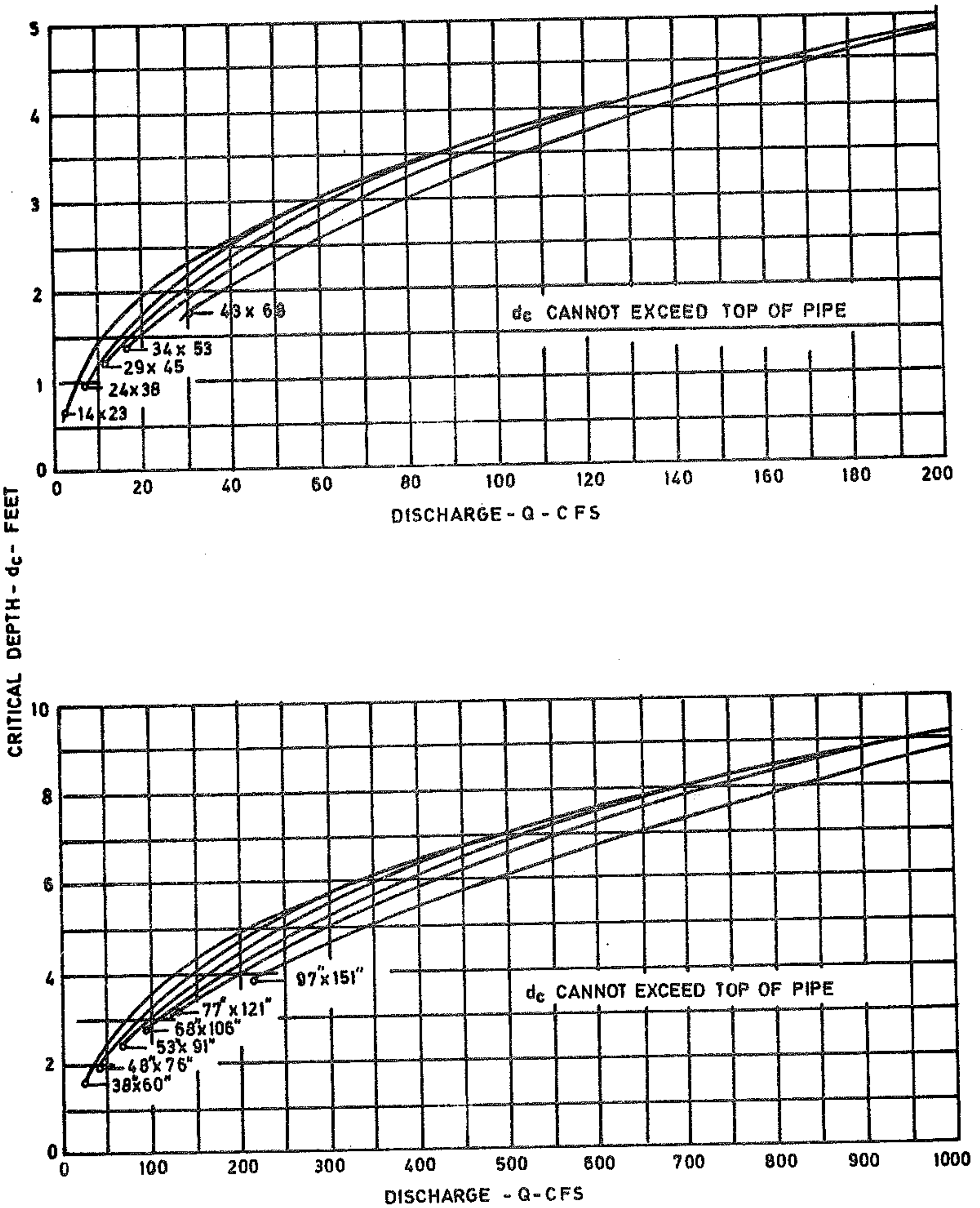
HEAD FOR STRUCTURAL PLATE CORRUGATED METAL PIPE ARCH CULVERTS 18 IN. CORNER RADIUS FLOWING FULL

n = 0.0327 TO 0.0306

CHART 17

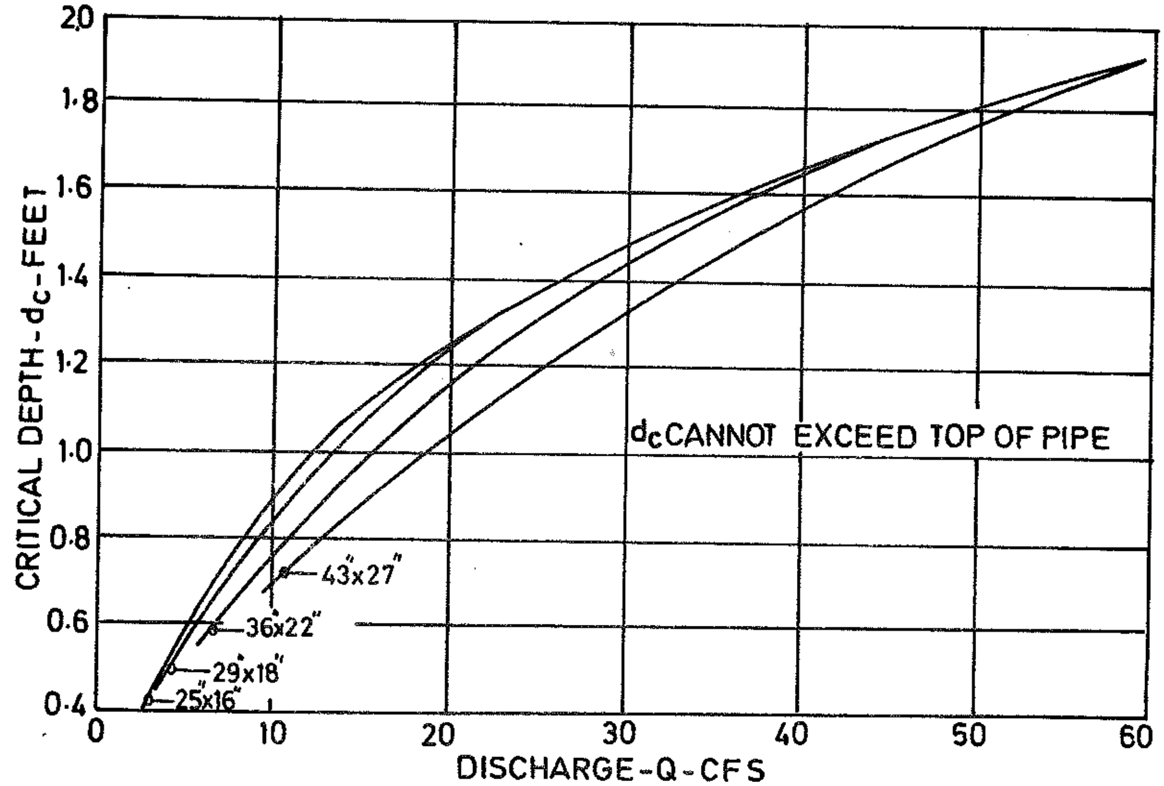


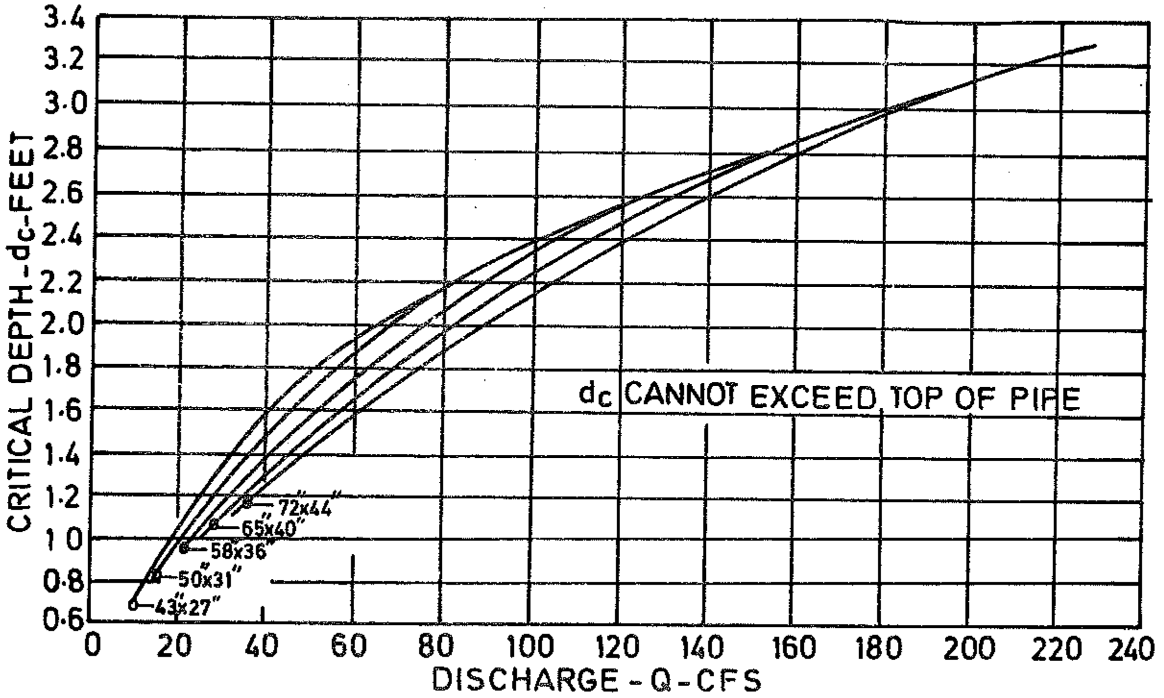
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL



CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL

CHART 19





BUREAU OF PUBLIC ROADS
JAN 1964

CRITICAL DEPTH
STANDARD C.M. PIPE ARCH

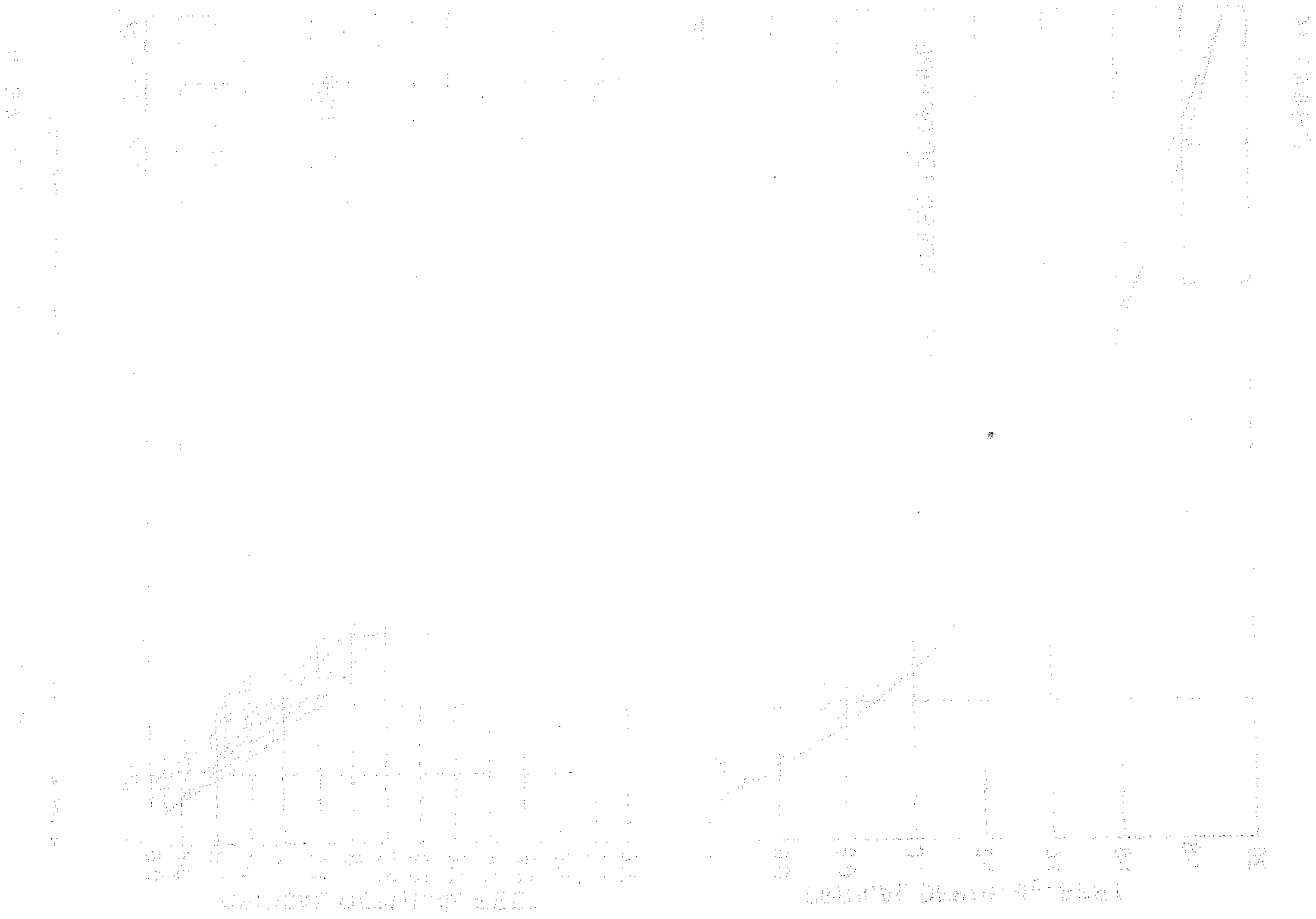
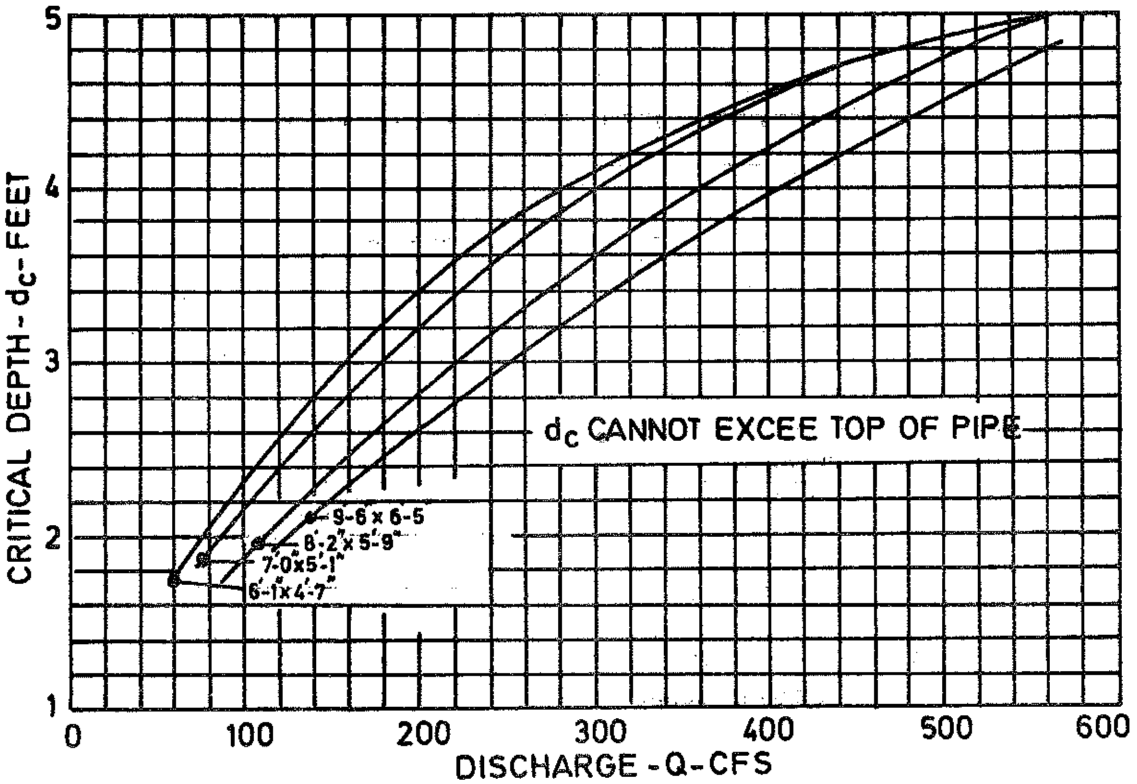
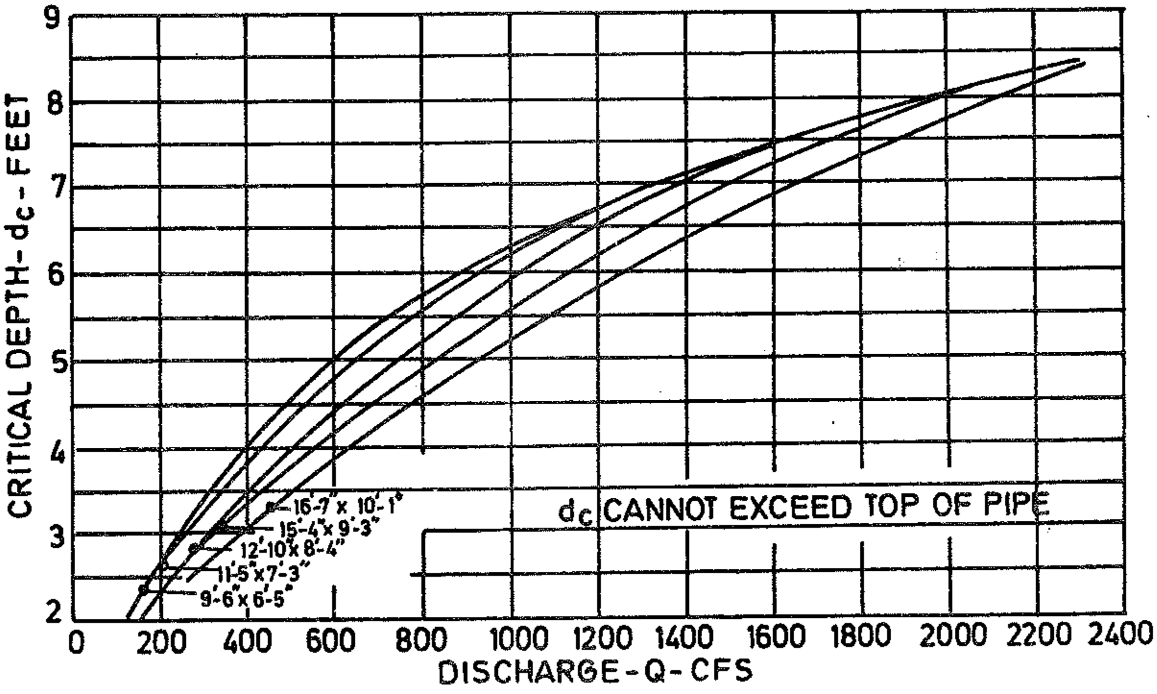


CHART 20





BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH
18 INCH CORNER RADIUS

PROJECT. ______

DESIGNER ______

DATE ______

HYDROLOGIC AND CHANNEL INFORMATION

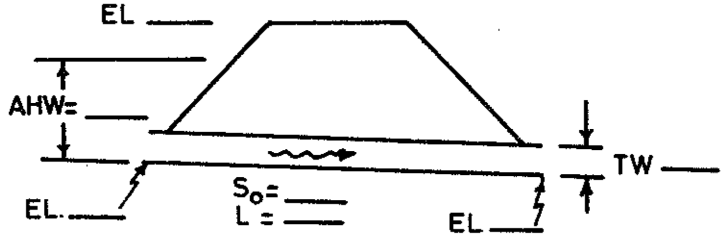
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE SAY Q_{25}
Q_2 = CHECK DISCHARGE SAY Q_{50} OR Q_{100})

SKETCH

STATION : ______



MEAN STREAM VELOCITY= ______

MAX. STREAM VELOCITY= ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				

SUMMARY & RECOMMENDATIONS:

Figure · 7