HYDROLOGY and DRAINAGE FOR RURAL ROADS

by

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INTRODUCTION

Objectives

"Highways occupy long, narrow strips of land and pose two types of drainage problems, namely;

a. Water collecting on the road (or on adjacent land slopes if the road is in cut) must be disposed of without flooding or damaging the highway and adjacent areas.

b. Highways cross natural drainage channels, and the water carried by these channels must be conveyed across the right-of-way without obstructing the flow in the channel upstream of the road and causing damage to the property outside of the right-of-way."[1]

The objective of the hydrology and drainage studies in highway design is to find solutions to these two problems. Generally, the solution will consist of the use of hydraulic structures in the form of lined and unlined ditches for longitudinal drainage and the bridges and culverts for cross drainage. Overflow spillways for relatively flat stream cross sections can also be adopted as a means of cross-drainage.

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[1] R.K. Linsley, J.B. Franzini. Water Resources Engineering, McGraw Hill Co. 1979, p. 528.

General Approach

The hydrologic and drainage procedure adopted in most road projects is summarized by the simplified flow chart in Figure 1. First, all the relevant data, i.e., rainfall, topographic maps and available streamflow records are gathered from different government offices. During this period, any previous study, i.e.; Feasibility Study, is reviewed and all the project roads are plotted and major waterways delineated. Preliminary values of the peak discharges are computed based on initial delineations and these discharges will be one of the factors used in the selection of the type of structure to be used; i.e.; bridge or box culvert. The initial plotting and delineations will be later on verified during the Reconnaissance Survey and the Topographic Survey. Where there are drastic changes in alignment, the physiographic characteristics on the affected major waterways will be adjusted and peak discharge will be recomputed. Among the drainage activities performed during Reconnaissance Survey are the following:

a. Hydraulic Inventory of Bridge Sites. (See Annex 1- Sample Form A)

b. Inventory of Existing Culverts. (See Annex 2-Sample Form B)

c. Inventory of Roadway Flooding and Side Road Drainage. (See Annex 3- Sample Form C)

The activities carried out during the topographic survey includes centerline horizontal and vertical alignment survey, profile survey of minor waterways and bridge site survey. After the detailed topographic survey of road alignment, all the project roads are replotted on the 1:50,000 topographic maps. The minor waterways will then be delineated and their peak discharges estimated. This will determine the appropriate size of the culverts to be used. From the bridge site surveys and the estimated peak discharge, a hydraulic computation is performed to estimate flood level and minimum bridge opening. The existing structures are checked for their hydraulic and structural adequacy and will either be removed, be replaced or be rehabilitated depending on their need and condition. Finally, these designs are checked and adjusted on a plans-in-hand verification survey.

This paper discusses the details of the following:

a. Hydrologic Studies (Determination of Peak Discharge for major and minor waterways.)

b. Hydraulic Design of Structures.

HYDROLOGIC STUDIES

Data Gathering

The following data are gathered from various government offices:

1. Physiographic Data from Philippine Coast and Geodetic Survey (PCGS) Maps.

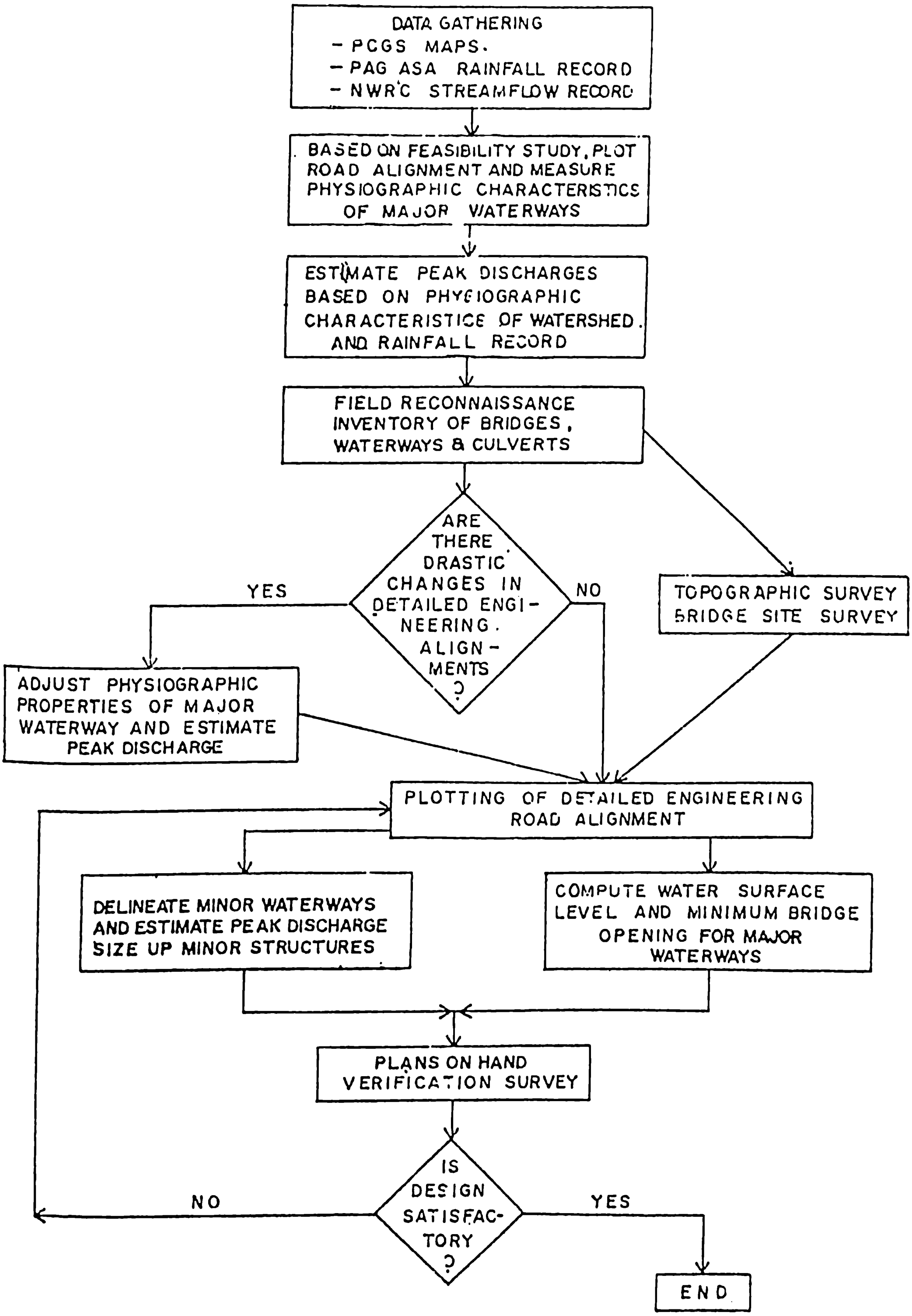

Figure 1 Hydrology and Drainage Work Flowchart

BASED ON 19 YEARS OF RECORD

Table 1.

COMPUTED EXTREME VALUES (IN MILLIMETERS) OF PRECIPITATION (Davao City)

RETURN PERIOD (YEARS)	5 MINS	10 MINS	15 MINS	30 MINS	60 MINS	2 HRS	3 HRS	4 HRS	12 HRS	24 HRS
2	12.2	21.7	29.6	46.9	67.1	75.5	78.7	03	04.3	95.3
5	14.9	26.9	36.7	59.2	02.9	92.9	96.9	111.3	114.1	129
10	16.8	30.4	41.4	67.3	93.3	104.2	107.7	130.1	133.9	151.2
15	17.8	32.4	44.1	71.9	99.2	110.6	114.3	140.6	145	163.8
20	18.5	33.7	45.90	75.20	103.4	115.1	118.8	148.1	152.0	172.6
25	19.1	34.8	47.4	77.6	106.6	110.6	122.4	153.8	158.8	177.4
50	20.8	38.1	51.8	65.3	116.4	129.3	133.2	171.3	177.3	200.3
100	22.4	41.3	56.20	92.9	126.1	139.7	144	188.8	195.4	221

Table 2.

INTENSITY IN (MILLIMETERS/HOUR) OF COMPUTED EXTREME VALUES (Davao City)

RETURN PERIOD (YEARS)	5 MINS	10 MINS	15 MINS	30 MINS	60 MINS	2 HRS	3 HRS	6 HRS	12 HRS	24 HRS
2	146.4	130.2	118.4	93.80	67.1	37.0	26.2	13.0	7	4.00
5	178.8	161.4	146.8	118.4	82.9	46.4	32.1	18.50	9.5	5.4
10	201.6	182.4	165.6	134.6	93.3	52.1	35.9	21.7	11.2	6.3
15	213.6	194.4	176.4	143.8	99.2	55.3	38.1	23.4	12.1	6.0
20	222	202.2	183.6	150.4	103.4	57.5	39.6	24.7	12.7	7.2
25	229.2	208.8	189.6	155.2	106.6	59.3	40.8	25.6	13.2	7.5
50	249.6	228.6	207.2	170.8	116.4	64.6	44.4	20.5	14.0	0.3
100	268.8	247.8	224.8	185.6	126.1	70	48	31.5	16.3	9.2

Source: PAGASA, Rainfall Intensity Duration Frequency Data of the Philippines, Vo. 1, P.17

All the 1:50,000 topographic maps covering the project roads can be secured from PCGS office. The waterways crossing the project roads are delineated with physiographic characteristics such as catchment areas, stream lengths and stream elevations measured.

2. Rainfall record from Philippine Atmospheric and Geophysical and Astronomical Services Administration (PAGASA). Rainfall records for the project roads are obtained from PAGASA. Normally the extreme rainfall record of the closset rainfall station is used for the project. Figure 2 shows a sample rainfall intensity duration curve and Tables 1 and 2 tabulates sample values for the extreme rainfall duration and rainfall intensity, respectively.

3. Streamflow Records from National Water Resources Council (NWRC).

   NWRC can also be checked for possible streamflow records.

4. Other Stream Channel Details and General Topographic Features from Field Surveys.

   The physiographic characteristics of catchment areas taken from PCGS Maps can be verified for their general accuracy. Additional stream channel details can also be gathered during the detailed bridge site surveys. This includes channel roughness, channel slope and conditions of river banks. The local people can be also asked about flooding history along particular road sections and waterways.

Frequency Level Adopted

The common frequency level adopted for the road design is shown on Table 3 below.

TABLE 3 - RECOMMENDED DESIGN FREQUENCY LEVEL[2]

Structure	Return Period in years
Pipe Culvert	10
Box Culvert/Overflow Structures	25
Bridge	50

[2] Ministry of Local Government, Design Guidelines - Rural Road Improvement Project, July 1983, p. 4.21.

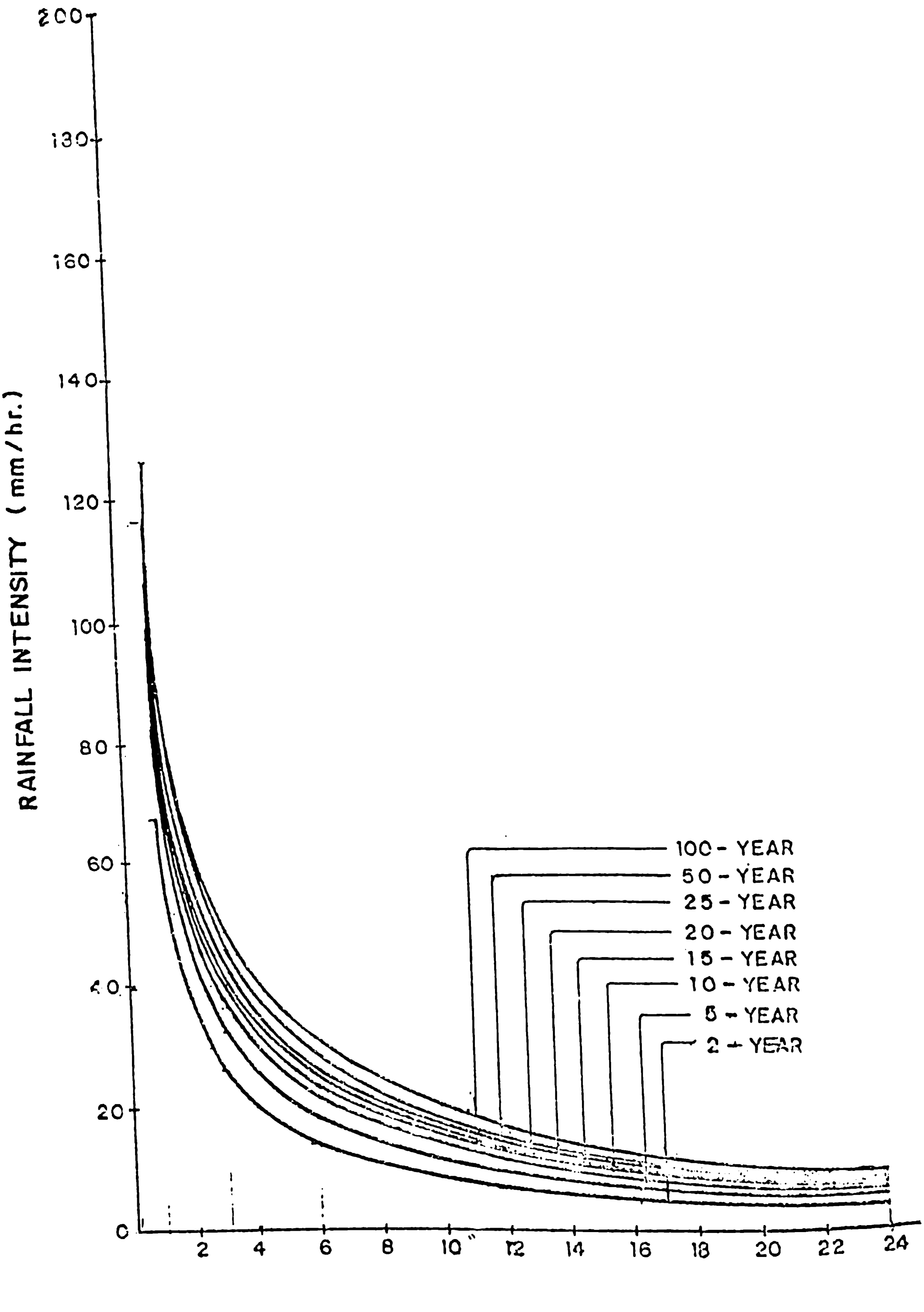

Figure 2 Rainfall Intensity Duration Frequency for Davao City

Rational Formula for Peak Discharge of Small Catchment Areas.

The Rational Formula is used in determining design flood discharges for watersheds less than 100 hectares. The formula is more popularly known in its fundamental form as

$$Q = CIA \text{ (Imperial Units)}$$

In Metric Units,

$$Q = CIA/360$$

Where:

C = Runoff Coefficient

I = Rainfall Intensity at time of concentration in mm/hr

A = Drainage Area in hectares

Q = Discharge, m^3/sec

The coefficients of runoff adopted for the design are shown in Table 4 below:

TABLE 4- RUNOFF COEFFICIENTS (C) FOR RATIONAL FORMULA[3]

Surface	Range
Impervious Pavements	0.90 – 0.95
Gravel Surface	0.70 – 0.85
Built-up Area (Light)	0.30 – 0.55
Built-up Area (Dense)	0.40 – 0.60
Bare Surface (Rocky)	0.70 – 0.90
Bare Surface (Clayey)	0.50 – 0.60
Bare Surface (Sandy)	0.30 – 0.40
Cultivated Land (Flat)	0.40 – 0.50
Grassed Area (Hilly to Steep)	0.50 – 0.70
Forest Area	0.30 – 0.45
Flooded Paddy	0.60 – 0.70

[3] C. B. Burke, D. D. Gray, "A Comparative Application of General Methods for the Design of Storm Sewers," Purdue University, Water Resources Research Center, Technical Report No. 118, August 1979, p. 7.

The time of Concentration can be obtained from the formula developed by Kirpich as indicated below:

$$Tc = \frac{L^{1.15}}{51\ (H)^{0.385}}$$

Where:

L = Length of main water course from the farthest source to the point of interest (meter)

H = Difference in elevation of the highest point and the point of interest along the main water course (meter).

T = Time of concentration (minutes).

Design Storm Unit Hydrograph Approach for Large Catchment Areas

This second approach is adopted for catchment areas larger than 100 hectares.

1. Basin Lag Time and Instantaneous Unit Hydrograph

First, the basin lag time and the instantaneous ûnit hydrograph is computed using the physiographic characteristics of the basin. In the computation of lag time, which is the time from centroid of excess rainfall to the peak discharge, the modified Snyder's Equation of lag is used.

$$t1 = C_t \left[\frac{(LL_{ca})}{\sqrt{s}} \right]^{0.38}$$

Where:

t1 = Time to peak in hours

C_t = Coefficient representing variation catchment slopes and storage. It ranges from 1.8 to 2.2 with steeper slopes generally associated with lower values of CL.

L = Maximum travel distance along the main stream (km)

Lca = Distance along the main stream from the outlet to a point opposite the center of gravity of the basin (km).

s = Weighted physical slope of the main stream.

The peak discharge for a given duration of rainfall that produces 1 mm of direct runoff is given below:

$$Q = 0.275\ CA/tl$$

Where:

Q = Peak discharge in m/sec resulting from 1 mm direct runoff.

A = Catchment size in km

t1 = Lag time in hours

C = Discharge coefficient accounting for floodwave and storage conditions. It is a function of lag time and duration of runoff producing rain effective area contribution to peak flow and drainage area.

This Q is multiplied by the ordinates of the Soil Conservation Service (SCS) unit graph in order to get the unit hydrograph for the catchment area. The SCS dimensionless unit hydrograph is shown in Figure 3.

2. Design Storm

The second major computation involved is the modified SCS computation for the design storm. Initially, the PAGASA point rainfall is corrected for area and duration. Figure 4 shows the percentages used for adjusting the point rainfall. As can be seen in this figure, the bigger the catchment area and the longer the rainfall duration, the smaller is the effective point rainfall. Rainfall increments are then computed and rearranged in order to yield the highest runoff. These increments are corrected for interception, depression storage and infiltration using the SCS procedure. In this procedure, a runoff curve number(CN) is extracted from Table 5.

TABLE 5

RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL COVER[4] COMPLEXES
(ANTECEDENT MOISTURE CONDITION II AND Ia = 0.2S)

LAND USE OR COVER	TREATMENT OR PRACTICE	HYDROLOGIC CONDITION	A	B	C	D

[4] W. Viessman, Jr. J.W. Knapp, G.L. Lewis, T.E. Harbaugh, Introduction to Hydrology, Harper and Row Publishers, New York 1977, p. 620.

t/t_p	q/q_p
0	0
0.25	0.14
0.50	0.46
0.75	0.83
1.00	1.00
1.25	0.87
1.50	0.62
1.75	0.44
2.00	0.30
2.25	0.20
2.50	0.15
2.75	0.10
3.00	0.06
3.25	0.045
3.50	0.03
3.75	0.02
4.00	0.01

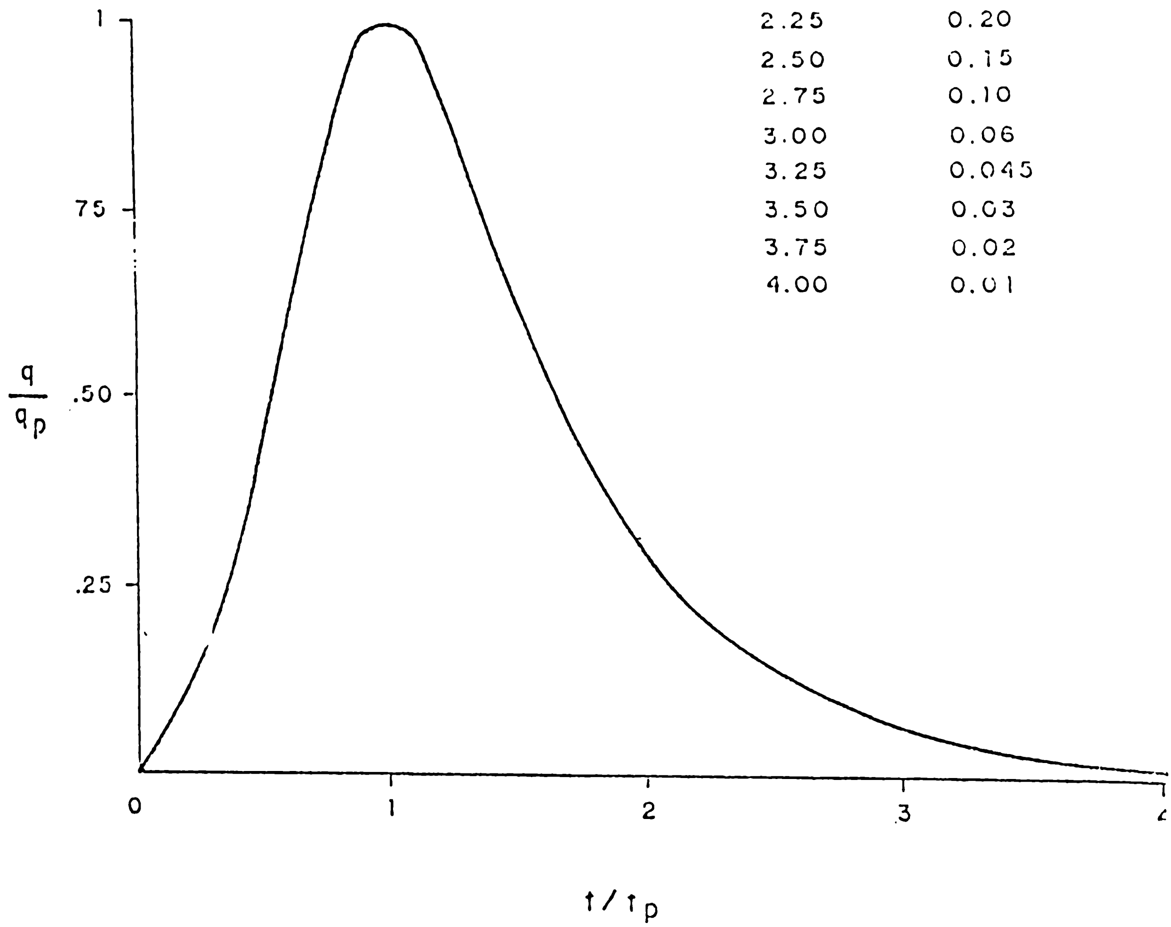

Figure 3 The Soil Conservation Service Dimensionless Unit Hydrograph

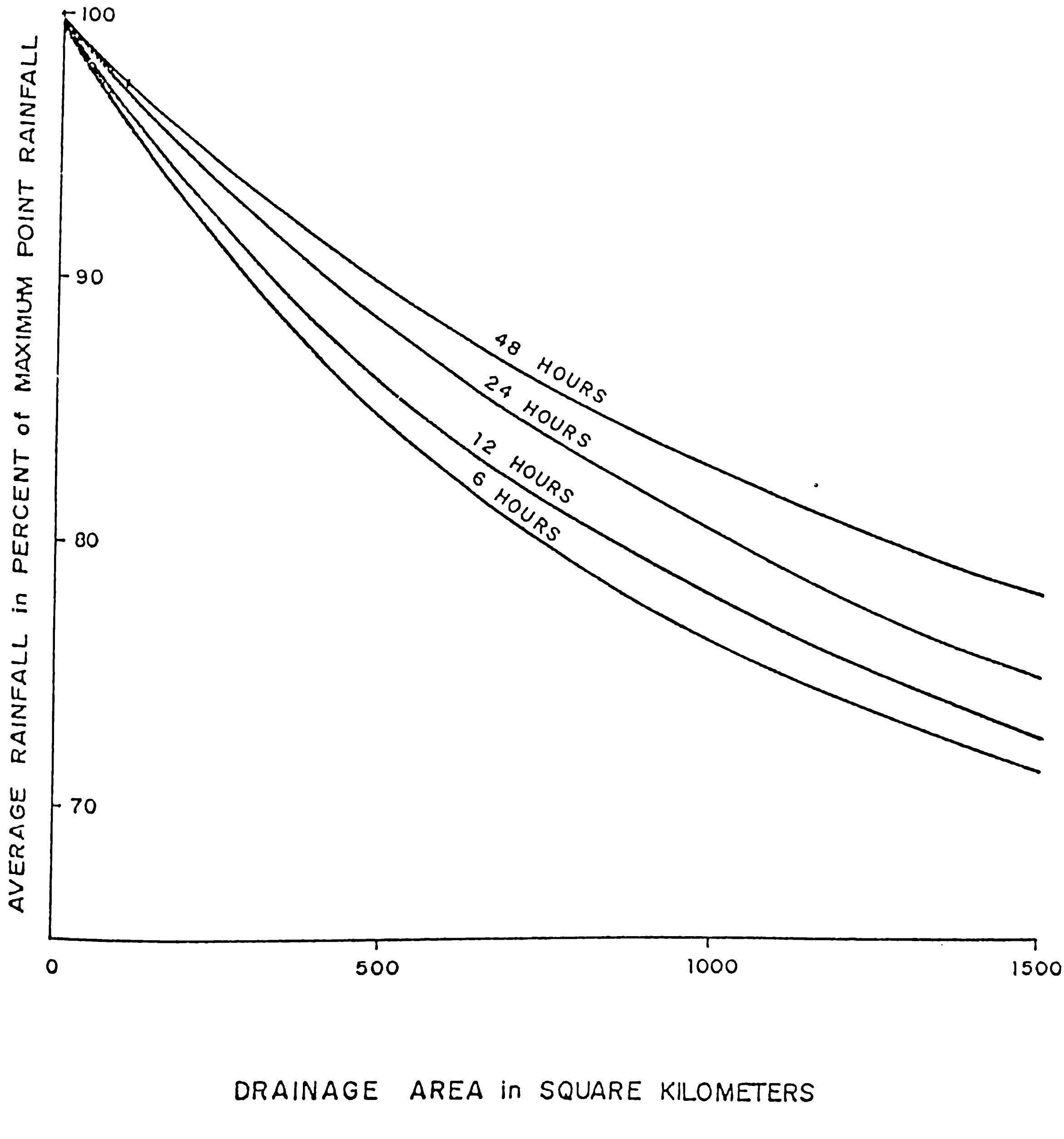

Figure 4 Area Duration Effective Point Rainfall

SOURCE : Maximum Depth Area Duration Curves
Central Luzon Basin Study,
Magat River Feasibility Report

Fallow	Straight Row		77	86	91	94
Row Crops	Straight Row	Poor	72	81	88	91
	Straight Row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured and terraced	Poor	66	74	80	82
	Contoured and terraced	Good	62	71	79	81
Small Grain	Straight Row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	63	75	83	87
	Contoured and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Closed Seeded Legumes or rotation meadow	Straight Row	Poor	66	77	85	89
	Straight Row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured and terraced	Poor	63	73	80	83
	Contoured and terraced	Good	51	67	76	80
Pasture or Range		Poor	68	79	86	89
		Fair	49	69	79	84
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		Good	25	55	70	77

The selection of the runoff curve number is dependent on antecedent moisture conditions, types of cover and soil runoff potential. Soils are classified A, B, C, and D, according to the following criteria:

Soil Type A

(Low runoff potential). Soils having high infiltration rates even if thoroughly wetted and consisting mainly of well-drained sands or gravels. They have a high rate of water transmission.

Soil Type B

Soils having moderate infiltration rates if thoroughly wetted and consisting mainly of well-drained soils with fine to coarse textures. They have a moderate rate of water transmission.

Soil Type C

Soils having slow infiltration rates if thoroughly wetted and consisting chiefly of soils with a layer that impedes the downward movement of water, or soils with fine. They have a slow rate of water transmission.

Soil Type D

(High runoff potential). Soils having very slow infiltration rates if thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. They have a very slow rate of water transmission.

The curve numbers shown in Table 5 are applicable to average antecedent moisture conditions (AMC II).

AMC I. A condition of catchment soils where the soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place. (This condition is not considered applicable to the design flood computation methods presented in this text.)

AMC II. The average case for annual floods, that is, an average of the conditions that have preceeded the occurrence of the maximum annual flood on numerous catchment areas. (This condition is considered applicable to the design flood computation methods presented in the text.)

AMC III. If heavy rainfall and low temperatures have occurred during the 5 days previous to the given storm and the soil is nearly saturated. (This condition would result to over conservative estimates of flood discharge and is not considered applicable to the design flood computation methods presented in this text.)

The following formula are associated with the SCS procedures.

$$S = \left[\frac{100}{CN} - 1\right] \qquad (254)$$

$$Ia = 0.2\,S$$

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

with $P > Ia$

$$S >= Ia + F$$

$$F = P - Ia - Q$$

where:

S = maximum storage potential of soil in mm.

F = Cumulative Infiltration loss in mm.

Q = Cumulative Runoff in mm.

P = Cumulative Rainfall in mm.

3. Convolution Equation and Baseflow Computation

The final steps involved the design storm unit hydrograph approach are the application of the convolution equation and computation of storm baseflow.

The convolution equation is:

$$Qj = \sum_{i=1}^{j} S(i)\ U(j - i + 1)$$

Where:

Qj = Runoff at time j in m^3/sec

$S(i)$ = Ordinates of unit hydrograph in m^3/sec.

$U(j - i + 1)$ = Excess rainfall during time $j - i + 1$ in mm.

In the absence of simultaneous rainfall runoff records in where bareflow can be separated, the storm base flow for majority of waterways in the Philippines, computation used is majority of waterways in the Philippines, computation used is that recommended by VEN TE CHOW in Applied Hydrology.

Based on Ven Te Chow's Study, the ratio of baseflow to peak runoff at the start of the storm hydrograph is 0.01 while the peak base flow ratio is 0.10. The baseflow recession coefficient from the peak is 0.9750.

These computations can be facilitated with the use of computers using a program (refer to program flowchart, Figure 5) developed by the Author. Sample computer program output are indicated in Annex 4.

HYDRAULIC DESIGN

A. BRIDGES

1. Mannings Equation for Bridges Without Constructions.

For bridges without any construction due to the abutment and piers, the design flood level can be determined by the use of Mannings Equation for open channel flow. Essentially, the cross-section of the river at bridge site is obtained by actual field survey. Mean bed slope of the stream is extrapolated from bridge site topography by map-scaled measurement of distances between contours crossing the stream channel or between representative spot elevations along the stream bed. Manning's roughness coefficient, n, is determined by ocular investigation of the channel bed and bank characteristics. From these parameters, a rating curve relating discharge with elevation was developed. The design flood discharge is then entered on the curve to determine the corresponding flood level.

In metric units, the Manning's Formula is:

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

where:

V = velocity in m/s (meters per second)

Q = discharge in m^3 /s (cubic meters per second)

R = hydraulic radius in meters

$$= \frac{\text{area of flow A, in } m^2}{\text{wetted perimeter P, in m}}$$

S = slope in meters per meter (of bed of water surface)

n = coeficient of roughness, tabulated as follows:

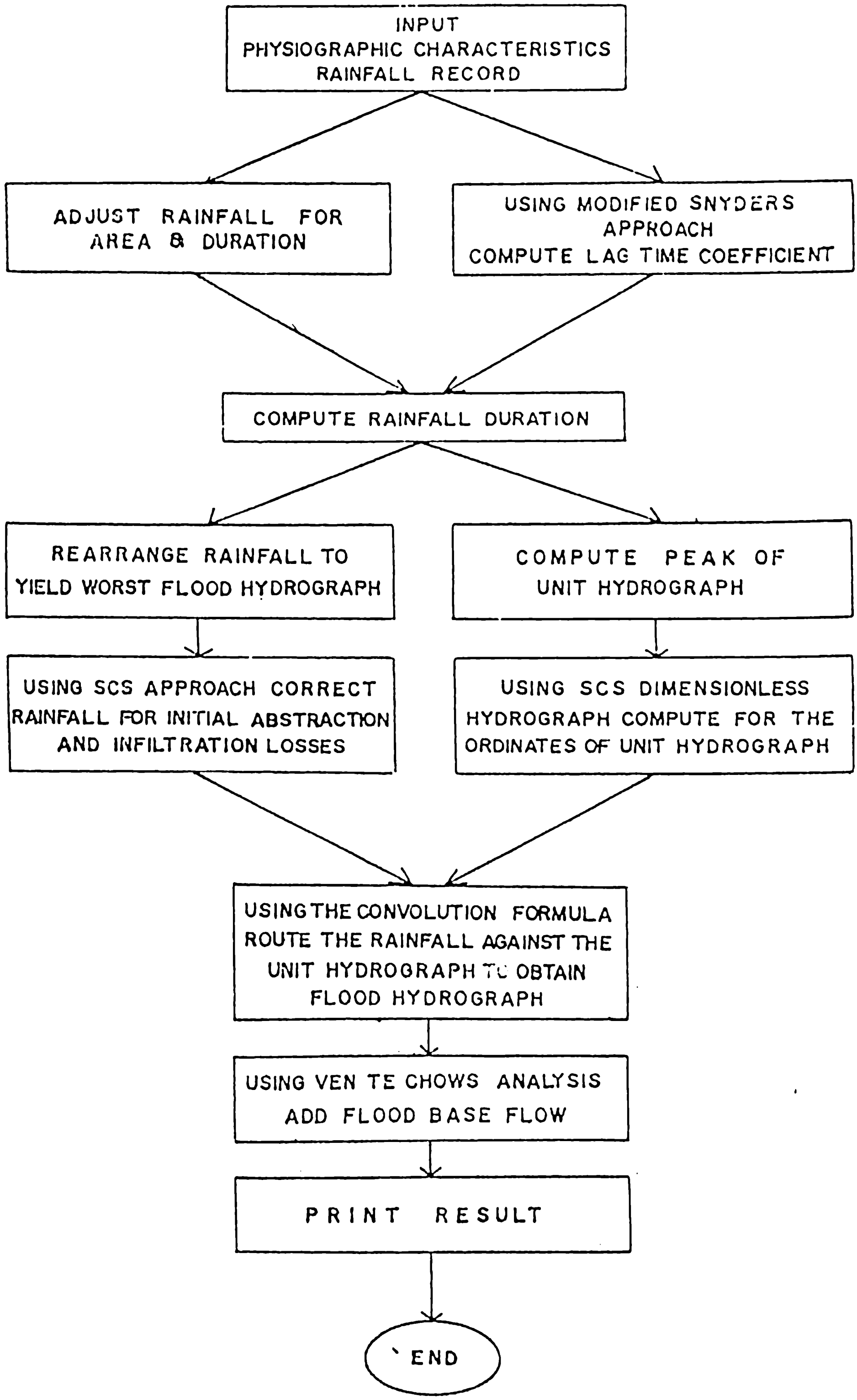

Figure 5 **Flow Chart For The Design Storm Unit Hydrograph Approach**

TABLE 6 - MANNING'S ROUGHNESS COEFFICIENT (n)[5]

Surface/Description	Range
Natural Streams:	
Regular, Straight Banks, fairly uniform bottom	0.027 – 0.033
Regular, Straight Banks with some vegetation	0.033 – 0.040
Meandering, with minor pools and eddies	0.035 – 0.050
Meandearing, with some pools and vegetation	0.50 – 0.070
Sluggish Steams, meandering with deep pools	0.060 – 0.080
Rough, rocky streams in mountainous terrain	0.050 – 0.080
Flood Plains (adjacent to stream beds):	
Grassland, short grass and no brush	0.030 – 0.035
Grassland, tall grass, with some brush	0.035 – 0.050
Cultivated land, row crops	0.035 – 0.045
Cultivated land, field crops	0.040 – 0.050
Scattered to heavy brush	0.050 – 0.070
Excavated Ditches and Channels:	
Earth, Irregular	0.025 – 0.035
Earth, w/ light vegetation	0.035 – 0.045
Earth, w/ heavy vegetation	0.040 – 0.050
Earth, dragline excavated	0.028 – 0.033
Rock cut, regular	0.030 – 0.035
Rock cut, irregular	0.035 – 0.045
Lined Ditches:	
Concrete, smooth	0.013 – 0.017
Earth, Straight and Uniform	0.020 – 0.025

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[5] F.M. Henderson, Open Channel Flow McMillan Publishing House, 1966, p. 99.

2. Backwater Effect of Bridges Piers

Backwater effect is the increase in upstream depth as a result of the obstruction coming from the presence of piers. The Nagler's equation which is based on experimental work of Yarnell is used to calculate the backwater effect of pior. Nagler's Equation is shown below:

$$\frac{dy}{y} = K\,Fr^2\,(K + 5Fr^2 - 0.60)\,(a + 15a)^4$$

$$a = 1 - e$$

$$e = \frac{b}{b_1}$$

$$Fr = \frac{Q^2 B}{gA^3}$$

Where: dy = backwater effect of the pier in m
y = depth of flow without the pier
e = contraction ratio
a = pier width to span ratio
b2 = effective channel with the introduction of pier in m.
b1 = channel width without the pier in m.
Fr = Froude number without the introduction of pier
Q = design discharge in m^3/sec
B = Surface width of channel in square meters.
A = Steam x-sectional area without the pier in m^2.
g = acceleration due to gravity, 9.8 m/sec^2
k = characteristics the pier shape according to the following table;

TABLE 7 - K VALUES FOR NAGLER'S EQUATION[6]

Pier Shape	K
Semi circular nose and tail	0.90
Lens shaped nose and tail	0.90
Twin cylinder piers w/ connecting Diaphragm	0.95
Twin Cylinder piers w/o Diaphragm	1.05
90 Triangular nose and tail	1.05
Square nose and tail	1.25

[6] F.M. Henderson, Open Channel Flow, Mc Millan Publishing House, New York, 1966, p. 265

3. Scour Around Bridge Piers

When bridge piers are set in an erodible bed, the high local values of water velocity around the upstream end of the pier create local scour, which in times of high flood can become very deep (See Figure 6). Prediction of the amount of scour in any particular situation must depend largely on experimental results. The total scour depth will depend on the upstream velocity. The experiment of Laursen[7] on model bridge piers indicated that the Froude number has no material effect, and that in the live-bed case Ys/b is related to Yo/b alone. The design relationship recommended on the basis of these experiments is shown graphically in Figure 7. If the piers are placed at an angle to the flow, the scour depth will be increased substantially.

The effect of angle of attack as measured by Laursen is shown in Figure 8; the scour depth for a pier with zero angle of attack is multiplied by a factor K.

B. BOX CULVER AND PIPES

Box culverts and pipes are designed in accordance with hydraulic standards, methods and procedures practiced by the US Bureau of Public Roads. Various monographs from Figure 9 to Figure 12 are used to determine the opening and capacity of culverts.

For design of concrete culverts with inlet control, the head water depth ratio (HW/D) is kept as close as possible to unity to minimize inundations of properties upstream, to relieve pressure on embankments and lessen outflow velocities. A HW/D of 0.85 is used for timber box culverts as a structural precaution.

For the design of culverts with outlet control, a head sdifference of 0.15m. for pipe and 0.30m for box culverts is adopted for the same reasons.

C. OVERFLOW STRUCTURE

The use of spillways, for economic reason has been recommended for streams with relatively flat cross-sections and high storm discharges. The general procedure used in the hydraulic design of spillways is shown in Figure 13. The overflow spillway is composed of two hydraulic components, culverts within the spillway to drain normal flows and the spillway to pass the flood flows.

[7] E. M. Laursen, "Scour at Bridge Crossings", Transactions American Society of Civil Engineers, Vol. 127, Part 1, 1962, p. 166.

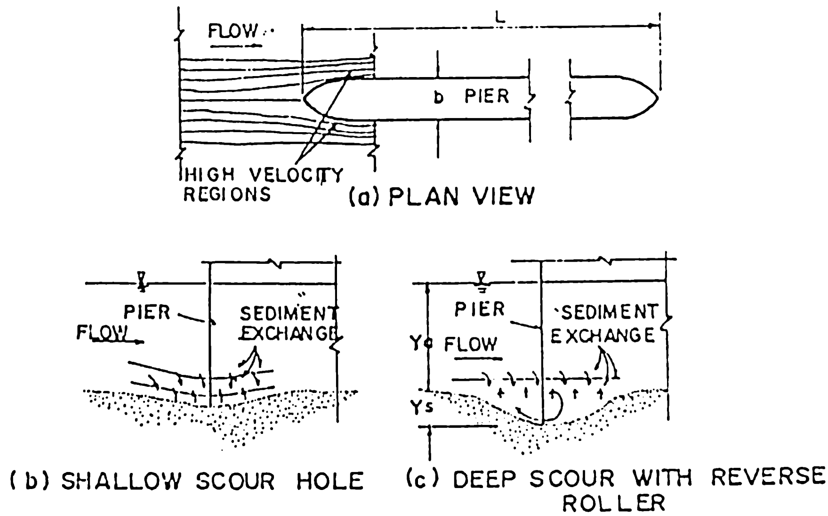


Figure 6 Scour Around Bridge Piers

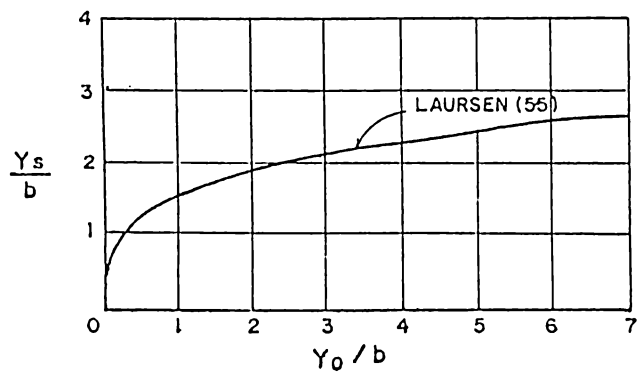


Figure 7 Scour At Bridge Piers With Live Bed Upstream

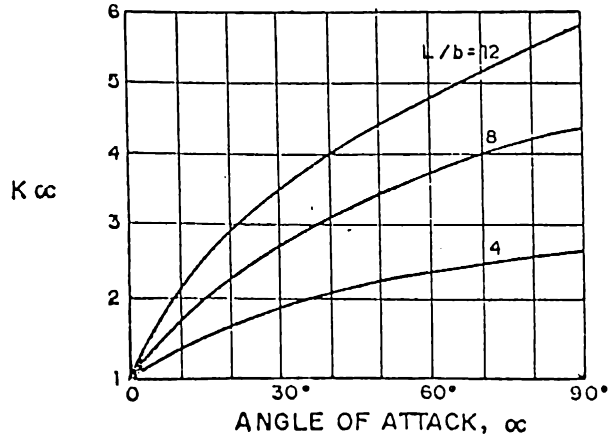
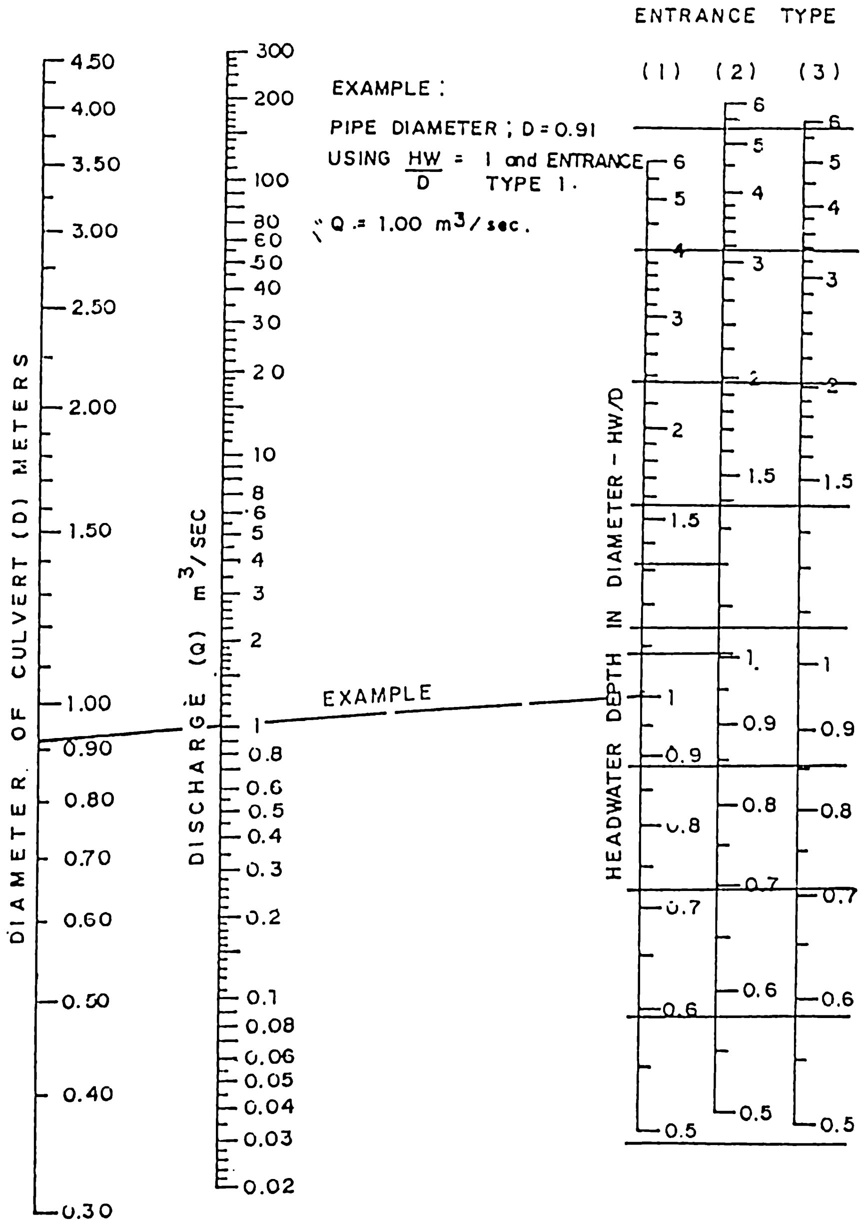


Figure 8 Effect Of Angle Of Attack On Scour Around An Isolated Bridge Pier



ENTRANCE TYPE

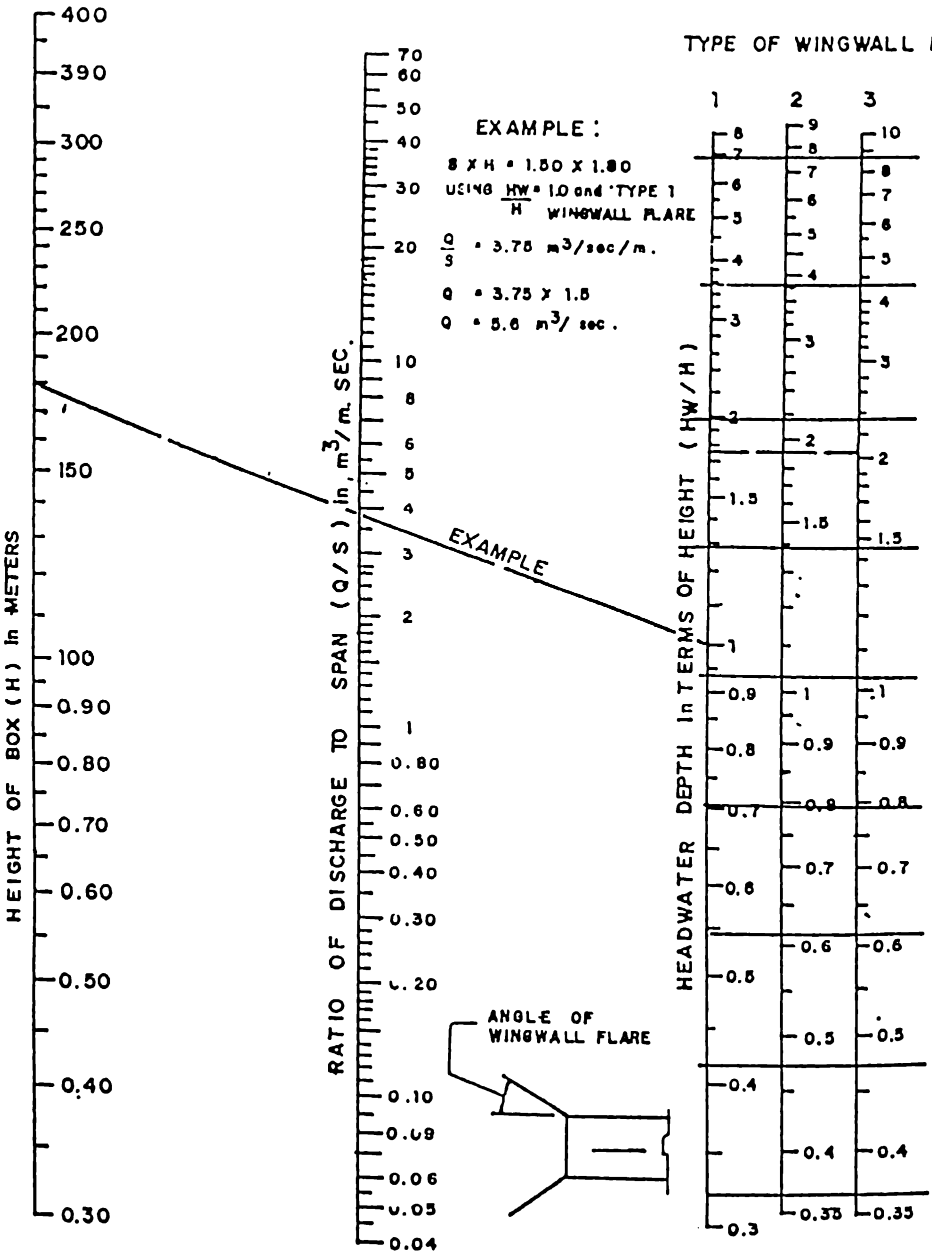
(1) SQUARE EDGE WITH HEADWALL

(2) GROOVE END WITH HEADWALL

(3) GROOVE END PROJECTING

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

Figure 9 Head water Depth For Concrete Pipe Culverts With Entrance Control



TYPE OF 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 SCALE, OR REVERSE AS ILLUSTRATED.

Figure 10 Headwater Depth For Concrete Box Culerts With Entrance Control

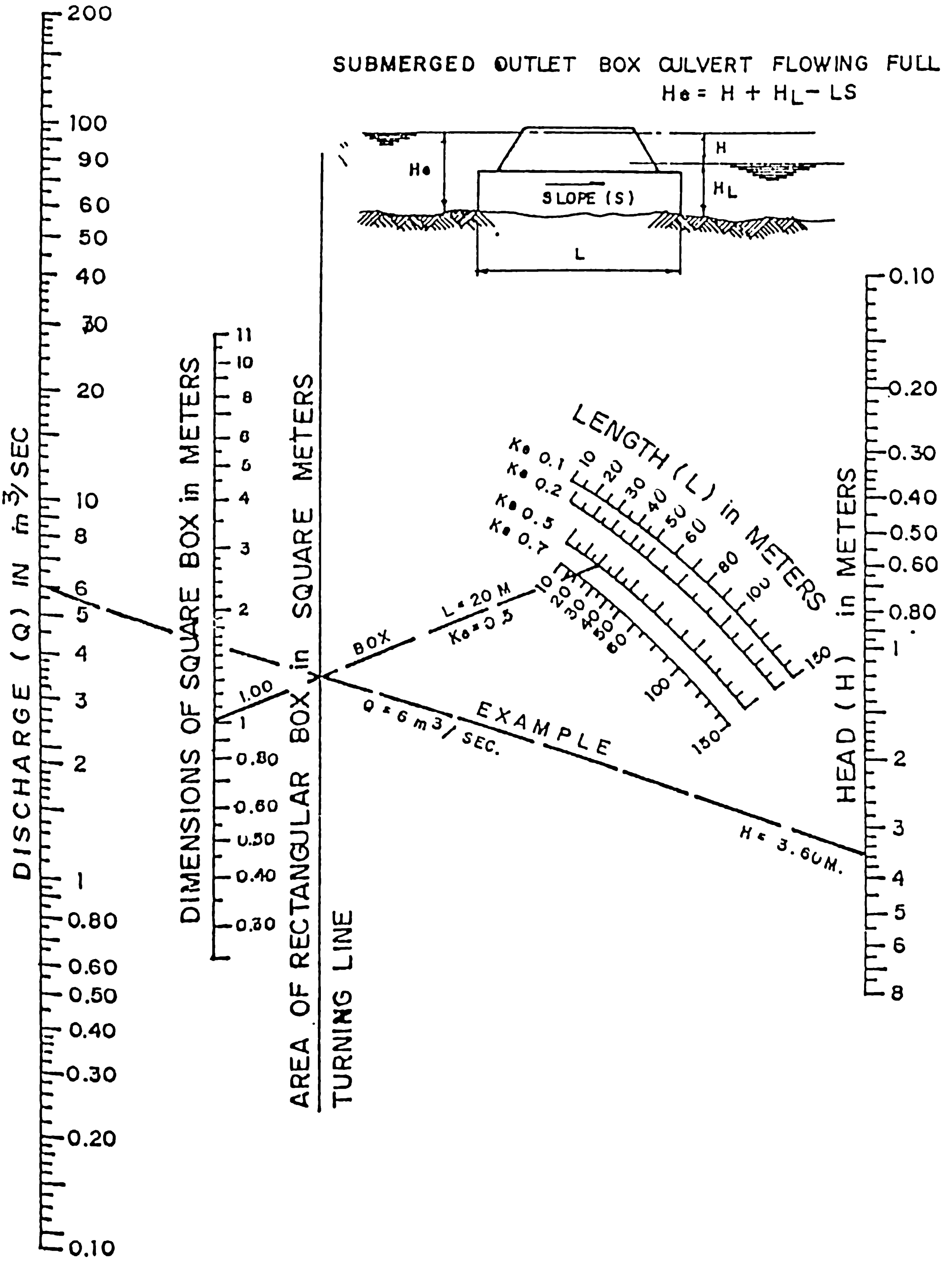


Figure 11 Head For Concrete Box Culverts Flowing Full, n= 0.012

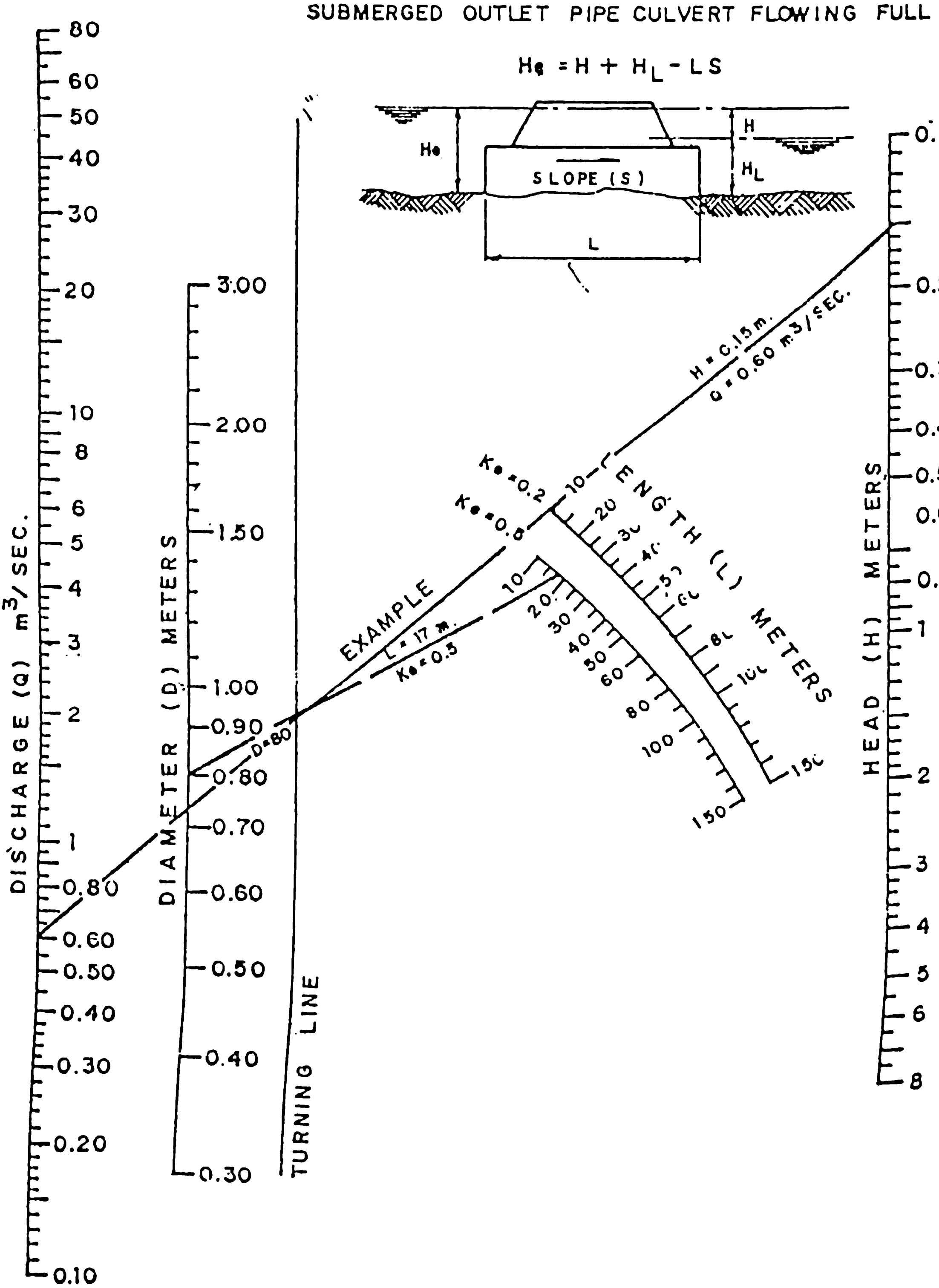


Figure 12 Head For Concrete Pipe Culverts Flowing Full, n= 0.012

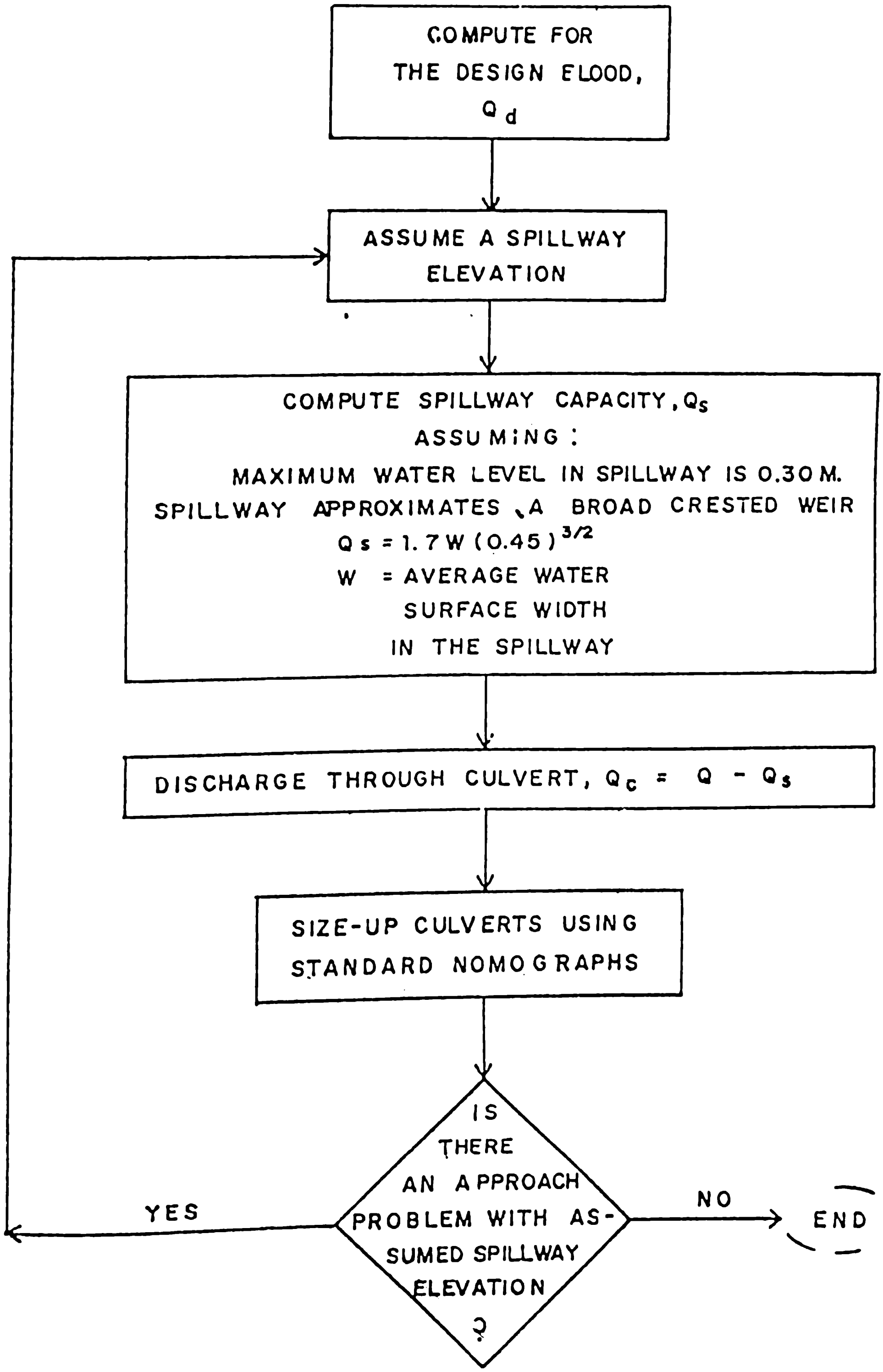


Figure 13 Flowchart For Design Of Spillways

In the design of spillway are the following are assumed:

1. The spillway approximates a broad-crested weir. Its rating curve is therefore that of a broad-crested weir, namely;

$$Qs \quad = 1.7WH^{3/2}$$

Where:

Qs = flow over spillway in m^3/sec.

W = average water surface width on top of spillway in meters

H = head on top of spillway

2. The maximum depth of flow in the spillway is 0.30 m. With critical depth in the spillway, H becomes 0.45 m.

3. The discharge over the spillway is independent of the discharge of the culverts below the spillway. Thus:

$$Qc = Qd - Qs$$

Where:

Qc = culvert discharge in m^3/sec

Qd = design discharge in m^3/sec

4. The culverts are sized using the standard monographs shown in Figure 9-12.

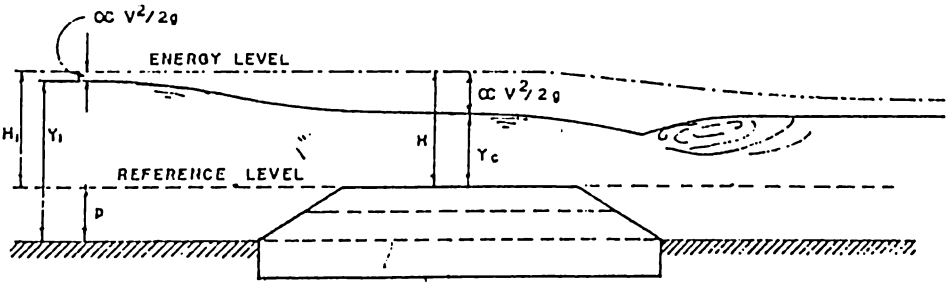
The typical overflow spillway is shown in Figure 14.

D. ROADSIDE DITCHES

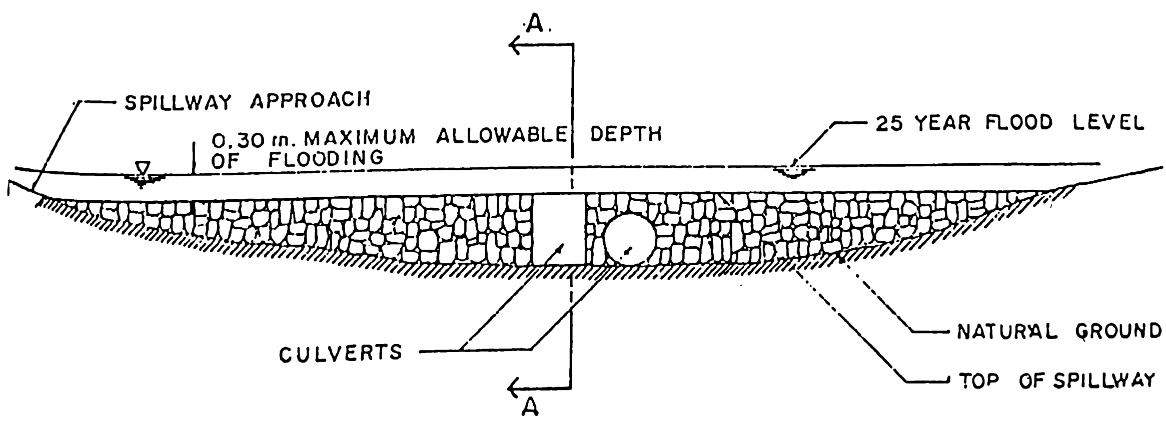
Two common problems are commonly observed in most of the project areas, namely: lack of maintenance of existing side ditches and absence of side ditches along some sections.

The new road typical section is designed with side ditches that drains surface runoff from intervening areas and deep enough to have the water surface lower than the subgrade.

To minimize the extent of damage caused by erosion, concrete lining of side ditches on steep slopes is recommended.



SECTION A-A



SPILLWAY ELEVATION

$Q_s = 1.7\, W\, H^{3/2}$ (ASSUMED TO BE A BROAD CRESTED WEIR)

$Q_C = Q_D - Q_S$

Figure 14 Flow Over a Spillway

E. ROADWAY FLOODING

Much roadway flooding is present along the many project roads. Each individual case is to be field inspected and investigated. The most common reasons for roadway flooding have been observed to be:

1. The existing finished grade of the road located on a flood plain is low and flooding occurs as water level on the flood plain rises.

2. The drainage structure is either non-existing, inadequate or silted causing the water to rise and overtop the roadway.

To eliminate this problem, the installation of a new, or the rehabilitation of existing culvert and the raising of finish grade are commonly recommended. Also, ditching on both sides of the road must be undertaken.

F. MINOR EROSION

The majority of the minor erosion problems are commonly encountered in mountainous terrain. Where the road surface have inadequate cross-slopes, and when combined with steep gradient, runoff flows down the road causing erosion to the road surface. Long intervals of cross culverts contributes greatly to the erosion of the sides parallel to the road.

To check the minor erosion to and on the road itself, extensive ditching (lining of the ditch with grouted riprap where the grades are steep) and the installation of additional cross culverts between long intervals of culverts where needed, is recommended. Lateral ditches are also needed on sites where runoff can be discharge away from the roadway to eliminate standing water on flat terrain, and to minimize culverting. These will reduce the cost of drainage improvement.

G. GROUTED RIPRAP SPILLWAY

Gully scour on rolling and mountainous terrain is taken into consideration since continuous erosion specially on culvert outlet sites creates construction on the roadway width which poses danger to motorists and passers-by.

The use of grouted riprap spillway, is a good and economical remedial measure. It will not only prevent scouring but provision of this type of slope protection would also mean restoration of the eroded slope embankment.

H. HEADWALLS AND WINGWALLS

Different types of headwall and wingwalls are commonly recommended for use on the road projects. Each one is designed to suit the existing local field conditions. Headwalls and wingwalls are usually constructed at the ends of culvert barrels for the following reasons:

1. To retain the fill material and reduce erosion of embankment slopes;
2. To improve hydraulic efficiency;
3. To provide structural stability to culvert ends and serves as a counterweight to offset buoyant or uplift forces; and
4. To inhibit piping which is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a hollow adjacent to the pipe. Fine soil particles are washed out freely along the hollow and erosion inside the fill may ultimately cause failure of culvert or embankment.

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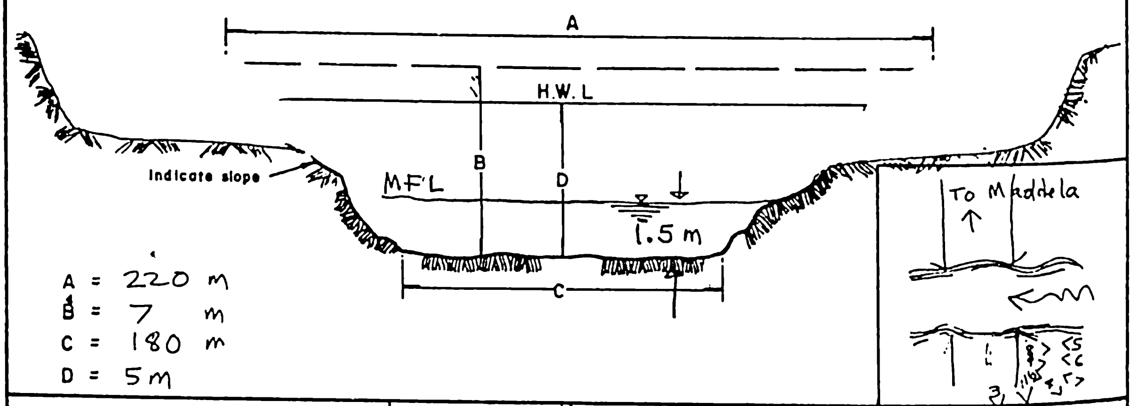
ANNEX 1

Ministry of Public Works and Highways

THIRD ROAD IMPROVEMENT PROJECT

HYDRAULIC INVENTORY OF BRIDGE SITES

Cross-sectional sketch of actual channel with dimensions - (show structure if existing) - take photos.



A = 220 m
B = 7 m
C = 180 m
D = 5 m

ITEM	COMMENTS
Existing structure (if any)	(Yes or No)
Type TIMBER	(RCDG, etc.)
Condition of structure	(good, bad, etc.)
Condition of Abutment 1	(good, bad, etc.)
Condition of Abutment 2	(good, bad, etc.)
Condition of Piers	(good, bad, etc.)
Flooding	(never, much, etc.)
Velocity at flood	(fast, slow, etc.)
Scouring and Erosion - Site MILD	(none, severe)
Normal water level	1.0–1.5 m
Lowest known water level	0.5 m
Highest known flood level	5.0 m
Angle of flood flow to bridge ℄	NORMAL
Is discharge affected by:	
I. Overflow from other rivers	—
II. Escape or ponding upstream	—
Driftwood hazard ? (State size)	No Problem
Estimate of flood discharge and velocity from above data.	
Water surface gradient at flood flow	Very flat
Boat clearance requirements (if any)	—
Tidal river	(Yes or No)
Is this cross-section typical	NO

ITEM	COMMENTS
Is river channel braided, mean during, incised.	INCISED
Is there any possibility of river tending to cut off approaches.	NONE
Is river bed aggrading, degrading or stable. Give results of any measurements.	—
Are banks stable	YES
Give details of any extraction near site, of river bed material.	COARSE GRAVEL
River bed material at bridge site (eg. silt, sand, fine or coarse gravel, etc.	COARSE GRAVEL
Records of local scour measured from general bed level	—
Is depth of scour affected by:	—
i. Direction of flow	—
ii. Local river bed shape	—
iii. Other local effects	—
Are moderate flood conditions critical for scour	—
Preliminary estimate of Manning's "n"	0.035

Other notes: (On catchment areas, etc.)

Catchment Area = 950 km²
Stream Length = 87.0 km
Stream length centroid to outlet = 39.5 km
Highest Point Elevation = 1600 m
middle Point Elevation = 500 m
Lowest Point Elevation = 80 m
Catchment area is lightly forested

No. of film/photo: PTT4 # 3, 4, 5, 6, 7, 8, 9, 10, 11

Surveyed by: Date	Name of River	Road	Location
			Km.

Ministry of Public Works and Highways

THIRD ROAD IMPROVEMENT PROJECT

ANNEX 2

CONSULTANT :______________ SHEET ______ OF ______

DATE ______

INVENTORY OF EXISTING CULVERTS.

1	2	3	4	5	6	7		8	9		10	11	12
KILOMETER OR ODOMETER READING	STATION KM.	SIZE	LENGTH	SKEW	TYPE	CONDITION ON HEADWALL.		CONDITION OF CULVERT	EROSION OR VEG. BUILD UP		HYDRAULIC ADEQUACY	FREQ OF FLOODING AND DURATION	IMPROVEMENTS AND GENERAL COMMENTS
						UP STREAM	DOWN STREAM		UP STREAM	DOWN STREAM			
14836.00	338+750	1-610	5		RCPC	N.	N	G	NP	NP			Existing Right cross pipe. Extend 1.0
14836.17	338+900	1-610	12	Normal	RCPC	N	N	G	S/VB	S/VB	A.	N	Cleanout
14837.00	339+750		Existing Oumatata			Bailey Bridge							
14837.60	340+350	1-610	10	Normal	RCPC								Proposed RCPC
14837.75	340+500	1-610	10	Normal	RCPC								Proposed RCPC
14837.95	340+700	1-610	10	Normal	RCPC								Proposed RCPC
14838.05	340+850	1-610	8	Normal	RCPC	N	N	F	S	S	A	N	Cleanout/Extend 2.0
14838.29	341+040	1-610	10	Normal	RCPC								Proposed RCPC
14955.56	341+060	1-610	10	Normal	RCPC								Proposed RCPC
14955.58	341+080	1-610	10	Normal	RCPC								Proposed RCPC
14955.85	341+350	1-610	10	Normal	RCPC								Proposed RCPC
14955.90	341+400	1-610	10	Normal	RCPC								Proposed RCPC
14956.00	341+500	1-910	14	Normal	RCPC	N	N	G	NP	NP	A	N	No Improvement
14956.10	341+600	1-610	10	Normal	RCPC								Proposed RCPC
14956.30	341+800	1-910	20	Normal	RCPC	N	N	G	NP	NP	A	N	No Improvement
14956.70	342+200	1-910	17	Normal	RCPC	N	N	G	NP	NP	A	N	No Improvement
14956.90	342+400	1-610	10	Normal	RCPC								Proposed RCPC

RCB-REINFORCED CONC. BOX
RCP-REINFORCED CONC. PIPE
SP-STEEL PIPE
O-OTHERS

G - GOOD
F - FAIR
B - BAD
VB - VERY BAD
N - NO HEADWALL

G-GOOD
F-FAIR
B-BAD
VB-VERY BAD

NP-NO PROBLEM
E-EROSION
VB-VEGETATION BUILD UP
S-SILTED

I-INADEQUATE
A-ADEQUATE

N-NO FLOODING
O-ONCE A YEAR
F-FREQUENT

ANNEX 3

Ministry of Public Works and Highways

THIRD ROAD IMPROVEMENT PROJECT

CONSULTANT : ____________________

SHEET ______ OF ______

DATE ______________

INVENTORY OF ROADWAY FLOODING AND SIDE ROAD DRAINAGE

1		2			3	4		5	6					
STATION, KILOMETER OR ODOMETER READING		FLOODING			VELOCITY OF FLOW DURING FLOODING	SIGNS OF EROSION		LENGTH OF ROADWAY FLOODING	ROAD SIDE DRAINAGE					
									PARALLEL DITCHES NEEDED				NEW CULVERTS NEEDED	IRRIGATION STRUCTURE
									LEFT		RIGHT			
FROM	TO	DEPTH METER	PRE-QUENCY	TIME HOURS		ROAD SHOULDER	SIDE SLOPE		FROM	TO	FROM	TO		
									348+200	348+300	348+200	348+300		
									348+500	348+900	348+500	348+900		
349+100	Abdalam Bridge								349+200	349+400	349+200	349+400		
350+000	Existing Timber Bridge								350+150	350+200	350+150	350+200		
											350+200	350+300		
									350+350	350+450	350+350	350+450		
											350+450	350+600		
351+300	Existing Timber bridge								350+600	351+000	350+600	350+850		
									351+750	351+900				
									352+300	352+500				
											352+800	353+050		
									353+150	353+250	353+150	353+250		
														333+320
353+600	Existing Bailey Bridge -18 m										353+650	354+ 20		

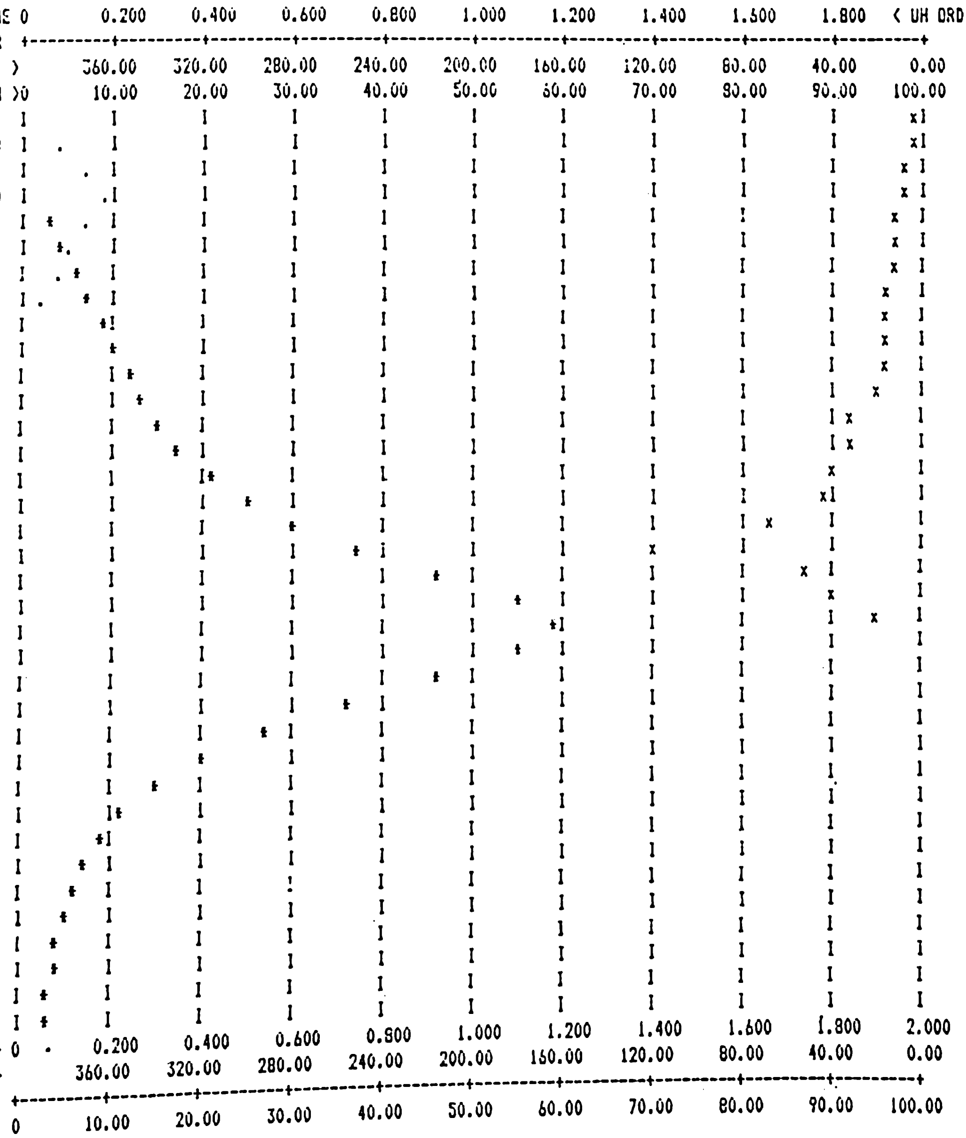
E-Erodible
N-Non-erodible

N - None
M - Mild
S - Severe

ANNEX 4 Sample Output of Hydrology Computer Program

DISCHARGE FLOOD HYDROGRAPH
HIGHLAND AGRICULTURAL DEVELOPMENT PROJECT
KEKEIG SALAT BOGKONG ROAD
BRIDGE A

UH ORD M3/SEC	ROFF MM	DISCH M3/SEC	TIME HOUR
0.026		0.60	1.30
0.034	2.52	0.74	2.60
0.151	5.58	1.11	3.90
0.182	7.66	1.89	5.20
0.159	9.22	3.12	6.50
0.113	10.49	4.65	7.80
0.080	11.61	6.31	9.10
0.055	12.65	7.92	10.40
0.036	13.67	9.45	11.70
0.027	14.73	10.89	13.00
0.018	15.85	12.28	14.30
0.011	18.30	13.68	15.60
0.008	29.25	15.39	16.90
0.005	31.92	17.82	18.20
0.004	36.48	21.16	19.50
0.002	41.68	25.21	20.80
	64.08	30.06	22.10
	116.94	37.35	23.40
	48.77	47.00	24.70
	38.22	55.92	26.00
	18.17	59.58	27.30
		55.34	28.60
		46.45	29.90
		36.52	31.20
		27.32	32.50
		20.25	33.80
		15.41	35.10
		11.80	36.40
		9.24	37.70
		7.57	39.00
		6.30	40.30
		5.42	41.60
		4.75	42.90
		4.23	44.20
		3.94	45.50
		3.74	46.80



RP	#UH	RP DDF	RP BFP	REC K	RGN CF	D AREA	STR LN	LN CTD	EL MAX	EL CTD	EL MIN	CN	HM SLP	LAG CT	TT PEK	UH DUR	MAX Q	MAX RO
25	16	0.0100	0.1000	0.9750	20.000	4.50	3.80	1.75	2400	1700	1320	83.0	0.2702	2.539	5.20	1.30	59.58	116.94