WORKSHOP ON DESIGN OF SMALL SCALE WATER CONTROL STRUCTURES

VOLUME I

পলি দাশ
উপ-বিভাগীয় প্রকৌশলী
নকশা সার্কেল-২
বাপাউবো, ঢাকা-১২১৫

SPONSORED BY

BANGLADESH WATER DEVELOPMENT BOARD
AND
CANADIAN INTERNATIONAL DEVELOPMENT AGENCY

ORGANISED BY

northwest hydraulic consultants ltd.

1987

DESIGN OF SMALL SCALE WATER CONTROL STRUCTURES.

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উপ-বিভাগীয় প্রকৌশলী
নকশা সার্কেল-১
বাপাউবো, ঢাকা-১২১৫

PREFACE

This publication includes

a. methodologies and details of computations for the design of a non tidal drainage regulator

b. methodologies and computations for hydrologic and hydraulic design of a non tidal and a tidal regulator and

c. hydrologic and hydraulic design of a flushing regulator

The purpose of these volumes are to train engineers of Bangladesh Water Development Board on design of small scale water control structures.

These publications have been produced with financial assistance from Canadian International Development Agency for the Small Scale Water Control Structures (SSWCS) Project.

The SSWCS Project has a training component which includes following categories of training :

1. Field workshops for Junior level field officers and contractors on construction management and O&M.

2. Dhaka Workshops for mid level engineers on planning and design aspects of small scale projects.

3. Advanced applied programs for mid level engineers at the Asian Institute of Technology, Thailand to provide in-depth training on water resource development.

4. Brief Technical Missions by senior officials to Canada and South East Asian countries to visit similar programs.

These volumes have been prepared to meet the partial requirements of training materials for 2nd and 3rd categories of training.

Any suggestions to improve the quality of contents will be greatly appreciated.

H.R. Khan
Water Resource Advisor.

March, 1987

CHAPTER I

BASIC HYDRAULICS

H.R.Khan

NORMAL DEPTH OF FLOW

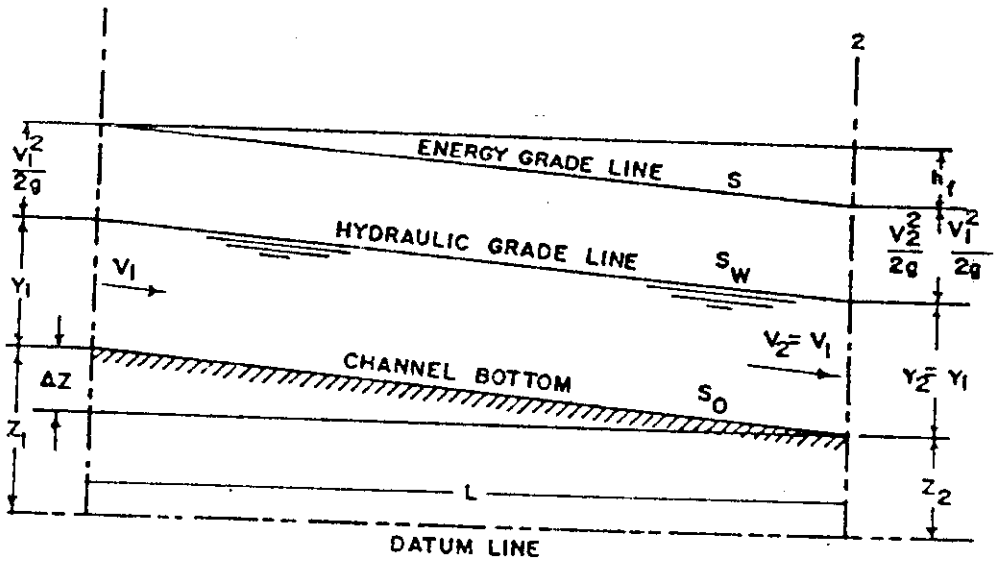

Fig. 1 Flow characteristics for uniform open-channel flow.

expressed as

$$E_{va} = a \frac{V^2}{2g} \qquad (5)$$

Where a is an empirical coefficient that represents the degree of turbulence. Experimental data indicate that a may vary from about 1.03 for prismatic channels. It is, however, normally taken as 1.00 for practical hydraulic work and is evaluated only for precise investigations of energy loss.

The total energy per pound of water relative to the bottom of the channel at a vertical section is called the <u>specific energy head</u> E. It is composed of the depth of flow at any point plus the velocity head at the point. It is expressed in feet as

$$E = Y + \frac{V^2}{2g} \qquad (6)$$

A longitudinal profile of the elevation of the specific energy head is called the <u>energy grade line</u>, or the <u>total-head line</u>. A longitudinal profile of the water surface is called the <u>hydraulic grade line</u>. The vertical distance between these profiles at any point equals the velocity head at that point.

Figure 1 shows a section of uniform open channel for which the slope of the water surface S_w and of the energy grade line S equal the slope of the channel bottom S_o.

Loss of head due to friction h_f in channel length L equals the drop in elevation of the channel Dz in the same distance.

Normal Depth of Flow. The depth of equilibrium flow that exists in the channel of Fig.1 is called the normal depth Y_n. This depth is unique for specific discharge and channel conditions. It may be computed by a trial-and-error process when the channel shape, slope, roughness, and discharge are known. A form of the Manning equation has been suggested for the calculation. (V.T. Chow, "Open-channel Hydraulics," McGraw-Hill Book Company, New York.)

$$AR^{2/3} = \frac{Qn}{1.486\ S^{1/2}} \qquad (7)$$

where A = area of flow, sq ft

R = hydraulic radius, ft

Q = amount of flow or discharge, cfs

n = Manning's roughness coefficient

S = slope of energy grade line or loss of head, ft, due to friction per line ft of channel.

$AR^{2/3}$ is referred to as a section factor.

SPECIFIC ENERGY AND ALTERNATE DEPTHS

We define the specific energy E as the energy referred to the channel bed as datum, i.e.

$$E = Y + \frac{V^2}{2g} \qquad (8)$$

and in this simple concept, introduced by B.A. Batemeteff in 1912 lies the key to even the most complex of open channel flow phenomens.

Since V = Q/A, Eq. (8) may be written as $E = Y + \frac{Q^2}{2gA^2}$

It can be seen that, for a given channel section and discharge Q, the specific energy in a channel section is a function of the depth of flow only.

When the depth of flow is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained. See Fig. 2. The curve has two limbs. The lower limb approaches the horizontal axis asymptotically toward the right. The upper limb approaches the 45° line as it extends upward and to the right.

The curve shows that, for a given specific energy there are two possible depths, for instance, the low stage Y_1, and the high stage Y_2. The low stage is called the alternate depth of the high stage, and vice versa. At point C, the specific energy is a minimum. This is the condition for critical state of flow. Thus, at the critical state the two alternate depths apparently become one, which is known as the critical depth of flow Y_c. When the depth of flow is greater than the critical depth, the velocity of flow is less than the critical velocity for the given discharge and the flow is called subcritical. When the depth of flow is less than the critical depth, the flow is supercritical.

Alternatively we may say that the curve represents two possible regimes of flow - slow and deep on the upper limb, fast and shallow on the lower limb - meeting at the crest of the curve C.

Other curves might be drawn for other values of Q: Since, for a given value of y, E increases with Q, curves having higher values of Q will occur inside and to the right of those having lower values of Q.

Criterion for a Critical State of Flow

Since $V = Q/A$, the equation for specific energy in a channel of small slope may be written

$$E = Y + \frac{Q^2}{2gA^2}$$

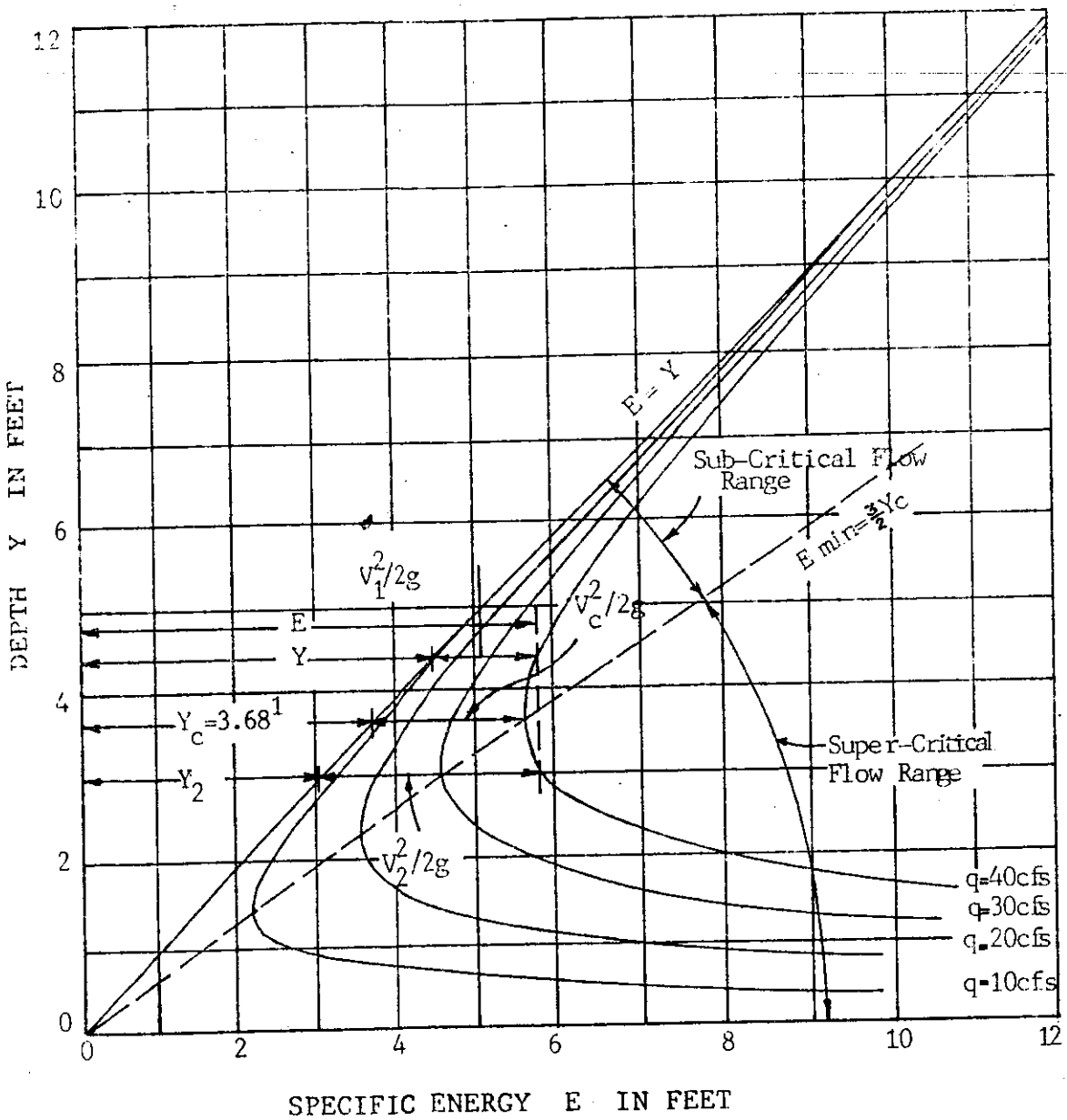

Figure 2. Specific energy and depth of flow.

We know at the critical state of flow $\frac{dE}{dy} = 0$.

Differentiating E with respect to y and equating $\frac{dE}{dy} = 0$, noting Q is a constant, putting $\frac{dA}{dy} = T$, and $D = \frac{A}{T}$, we get for the critical flow condition.

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

So it means that at the critical state of flow, the velocity head is equal to half the hydraulic depth.

The above equation may also be written as $V/(gD)^{1/2} = 1$, which means $F_r = 1$.

For rectangular channels, the above two equations are reduced to the following forms

$$\frac{V^2}{2g} = \frac{Y}{2} \quad \text{and} \quad \frac{V}{(gy)^{1/2}} = 1 \qquad (9)$$

It can also be shown that at the critical state of flow the discharge is maximum for a given specific energy.

For rectangular channels, it can be proved to be

$$Y_c = \left(\frac{q^2}{g}\right)^{1/3} \qquad (10)$$

and

$$Y_c = \frac{2}{3} E_c \qquad (11)$$

Where q is the discharge per foot of channel width

= Q/width

Application of the specific energy concept

Much of the complexity of open channel flow relates to the characteristics of critical, subcritical and supercritical flows. These categories of flow affect the design of conveyance channels and hydraulic structures. This section applies the concept of specific energy to these flow categories and is taken from the National Engineering handbook of the Soil Conservation Service (1971).

Uniform flow at or near critical depth is unstable. This results from the fact that the unique relationship between energy head and depth of flow which must exist in critical flow, is readily disturbed by minor changes in energy. Examine the curve for q = 40 cfs on Figure 2. The critical depth is 3.68 ft and the corresponding energy head is 5.52. If the energy head is increased to 5.60, the depth may be 3.2 or 4.2. Those who have seen uniform flow at or near critical depth have observed the unstable, wavy surface that is caused by appreciable changes in depth resulting from minor changes in energy. In channel design these conditions must be recognized. Variations in channel roughness, cross section, slope, or minor deposits of sediment or debris may cause fluctuations in depth of flow that are significant to channel operation. In many cases it is desirable to base design computations on two or more evaluations of flow

resistance in order to establish the probable range of operating conditions. Because of the unstable flow, channels carrying uniform flow at or near critical depth should not be used unless the situation allows no alternative. The critical flow principle is the basis for the design of control sections at which a definite stage-discharge relation is desired or required.

Two general characteristics of subcritical flow are important. First, at all stages in the subcritical range, except those in the immediate vicinity of critical, the velocity head is small in comparison with the depth of flow. Study of the curves of constant discharge will make this point clear (Figure 2). Second, the velocities are less than wave velocity for the depths involved and backwater curve will result from retardation of velocity. Thus, in the subcritical range we are concerned with cases in which the depth of flow is of greater importance than kinetic energy as represented by velocity head. In practice, this means that changes in channel cross section, slope, roughness, and alignment may be made without the danger of developing seriously disturbed flow conditions so long as the design assures that flow in the supercritical range will not be created for some discharges in the operational range. However, in many cases the latitude in design which may be possible as a result of dealing with subcritical flow will be offset by limited head requiring that friction losses be held to a minimum.
Designing structures to carry supercritical flow requires consideration of some of the most complex problems in hydraulics.

in supercritical flow the velocity head may range from a value approximately equal to depth of flow to many times the depth of flow. Note from the curves on Figure 2 that the velocity head increases very rapidly with decreases in depth throughout the supercritical range. Supercritical velocities exceed the velocities at which gravity waves may be propagated upstream. Any obstruction of flow will result in a standing wave, and there will be no effect upon flow upstream or the obstruction. The fact that kinetic energy predominates in supercritical flow and cannot be dissipated through the development of a water surface curve extending upstream is of primary importance in design.

Channels involving changes of direction, contraction or expansion of cross section, or the joining of two flows at a confluence at which either or both of the flows may be supercritical, require careful consideration. Changes in direction or channel contractions develop disturbances at the walls of the channel which take the form of standing waves reflected diagonally from wall to wall downstream from the disturbance points. The height of these standing waves may be several times the depth of flow immediately upstream of the origin of the disturbance. Confluences at which either flow or both flows may be supercritical also develop disturbances resulting in standing waves. In expanding channels the discharge may be incapable of following the channel walls because of the high velocities involved. This results in no uniform depth and the development of a hydraulic jump which is unstable as to both location and height.

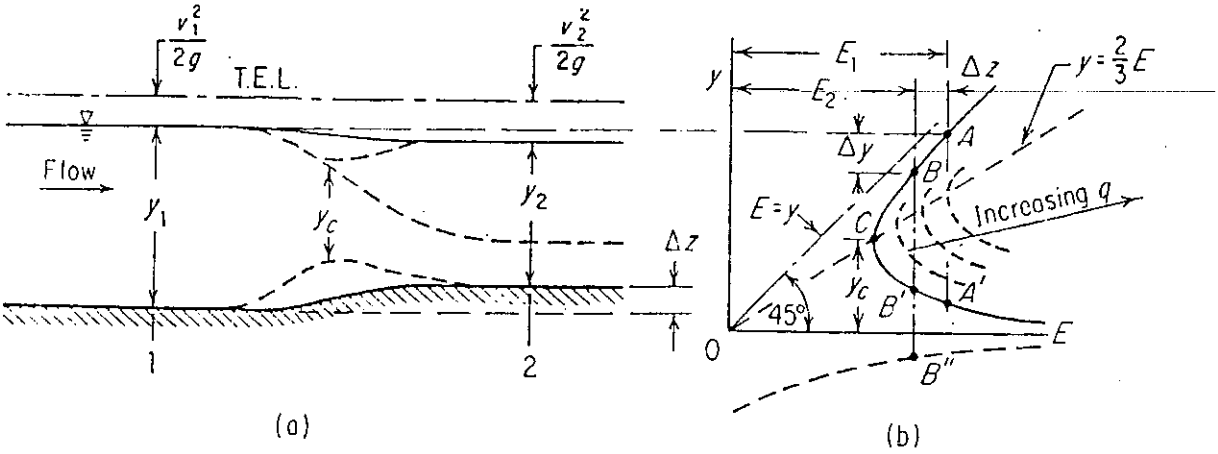

Figure 3. The Specific Energy Curve and Its Use in the Transition Problem

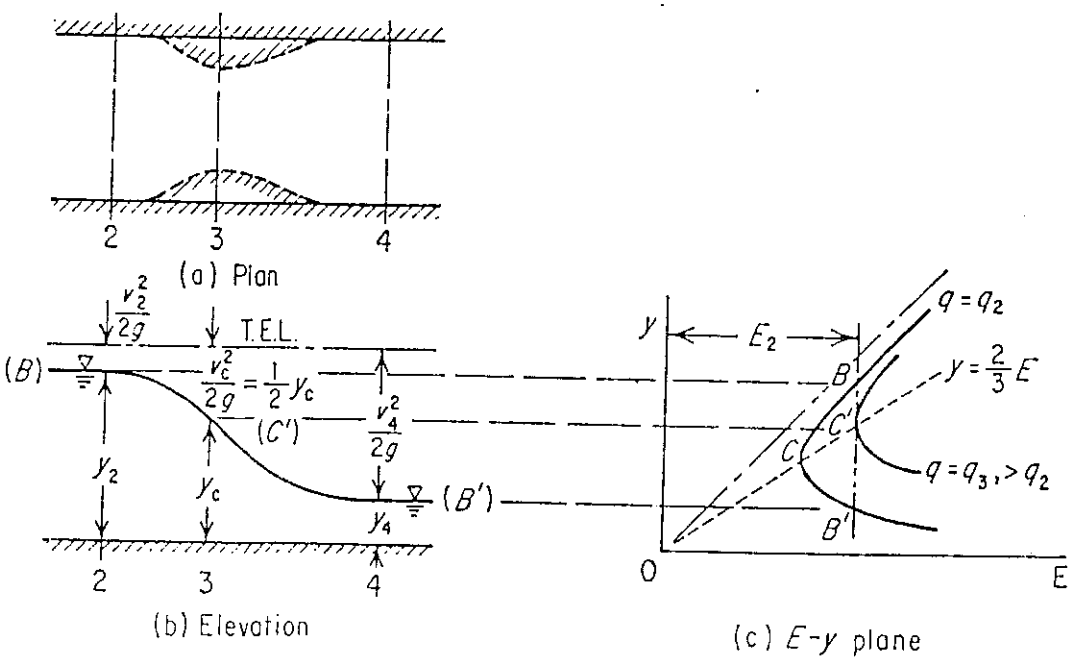

Figure 4. The Change From One State of Flow to Another Via a Contraction in Width

Specific Energy and the Transition Problem

With the aid of specific energy curve it is now possible to see clearly what is involved in the problem shown in Fig. 3. Suppose that the flow upstream of the step (section 1) in the channel bed has such a velocity and depth as to be represented by the point A on the upper (slow and deep) limb of the E-y curve. We assume also that there is no change in channel width over the step, so that value of q (q = Q/width) remain unaltered. Since the curve in Fig. 3 is drawn for a given constant value of q, it follows that the point representing the flow over the step (section 2) will also lie on the curve. Its position on the curve is easily located by recalling that the total energy must be the same at sections 1 and 2 and therefore that the specific energies differ by the height of the step, i.e. $E_1 - E_2 = Dz$.

Having obtained the value of E_2, we can now obtain solution describing the flow at section 2; they are represented by the points where the line $E = E_2$ cuts E-y curve. There are two physically real solutions, represented by the points B and B', the third, represented by the point B", is unreal and need not concern us further.

This two solutions would also apply to the case where the upstream flow was represented by the point A', having the same specific energy E as the point A. The question now arises as to which of the two solutions represented by B and B' is more likely

to occur in reality, and this question really reduces to one of accessibility: given a certain upstream flow (represented by A) are the two possibilities for the downstream flow (represented by B and B') equally accessible? The energy equation in itself appears to have little to tell us about this problem, but the form of the E-y curve is, as we shall see, a most valuable guide.

As the flow moves over the step, the point representing the flow moves from A to either B or B': If we are concerned with the relative accessibility of B and B', it is reasonable to ask what kind of path this flow-point must follow. The answer is quite simple; since the discharge per unit width is unaltered, the flow-point must therefore remain on the E-y curve which, it will be remembered, was drawn for a certain constant value of q. The point cannot jump across the curve straight from B to B', for if it did the value of q would have to change temporarily from its original magnitude and then return to it again, i.e., the width would have to undergo a similar temporary change. It is easily seen that this change in width would take the form of a constriction followed by an expansion as shown in Fig. 4 (For the sake of clarity this figure is drawn without including the step, as if the constriction were in fact a short way downstream of the step).

If the width remains constant the only available path from B to B' is round the curve itself; but if this path were followed, the specific energy would have to drop below E_2 and return to it

again. This could only happen if the bed level temporarily rose above the level of the step (just far enough to bring the point from B to C, the crest of the curve) and then dropped back to the level of the step, as shown in Fig. 3.

The conclusion is that if the width remains unchanged, and if the bed rises steadily to the level of the step without rising above it at any point, the point B' is inaccessible when the flow is represented by the point A. Similarly the point B will be inaccessible if the upstream flow is represented by the point A'. The the point A. Similarly the point B will be inaccessible if the upstream flow is represented by the point A'. The jump from one side of the curve to the other can be accomplished only by inserting a local constriction either in width or in bed level at or near the step. This statement of course relates only to this particular type of problem; there may be other conditions under which a change from one regime to the other is possible.

One last remark may be made about this example. As we go from A to B the depth decreases and the velocity and velocity head must therefore increase. Since the total energy line remains at the same level, the water surface must therefore drop, as in Fig. 3 as the flow moves over the step-a slightly surprising conclusion in view of one's natural expectation that the water surface would have to rise in order to get over the obstacle presented by the step. On the other hand, movement from A' to B' is accompanied by an increase in depth, so that the water surface rise.

Example 1

Water flows at a velocity of 3 ft/sec and a depth of 5 ft in a rectangular channel. There is a smooth upward step of 6 in. in the channel bed; find the depth of water over the step and the change in the absolute level of the water surface.

Describing as section 1 and 2 the sections upstream of the step and over the step respectively, we first calculate the upstream specific energy.

$$E_1 = 5 + \frac{3^2}{2g} = 5.14 \text{ ft}$$

Then the downstream specific energy is equal to

$$E_2 = E_1 - 0.5 = 4.64 \text{ ft}$$

and since q = 15,

$$E_2 = Y_2 + \frac{q^2}{2gY_2^2} = Y_2 + \frac{3.49}{Y^2} = 4.64 \text{ ft}$$

This is a cubic equation, which can nevertheless be readily solved giving the result

$$Y_2 = 4.565 \text{ or } 0.977 \text{ ft}$$

and since, as is easily shown, the upstream flow is subcritical, then only the subcritical value of Y_2 (i.e. 4.465) is possible. The water level is now 4.465 + 0.5 = 4.965 ft above the upstream channel bed, so that the water level has dropped by only 0.033 ft.

Example 2

Upstream conditions are as in Example 1. The width contracts smoothly from 6 ft to 5 ft; find the depth of water within the contraction.

$E_1 = 5.14$ as before, and on this occasion E_2 has the same value. But $q_2 = 15 \times 6/5 = 18$, whence:

$$E_2 = Y_2 + \frac{q^2}{2gy_2^2} = Y_2 + \frac{5.03}{Y_2^2} = 5.14 \text{ ft}$$

yielding the solution

$$Y_2 = 4.93 \text{ or } 1.12 \text{ ft}$$

Again, only the subcritical value of Y_2 (i.e., 4.93) is possible. Also, the change in the absolute level of the water surface is again small because of the low upstream froude number.

Smallest section

There is a certain degree of channel contraction beyond which no solution is possible. The amount of this contraction is readily calculated in the case of the upward step; the maximum height of step permissible is simply equal to the difference between the upstream specific energy and the minimum possible specific energy (that of critical flow), which is easily found from equations derived earlier.

It is important to notice that when the step is of this critical height, and is of limited length the downstream flow may be

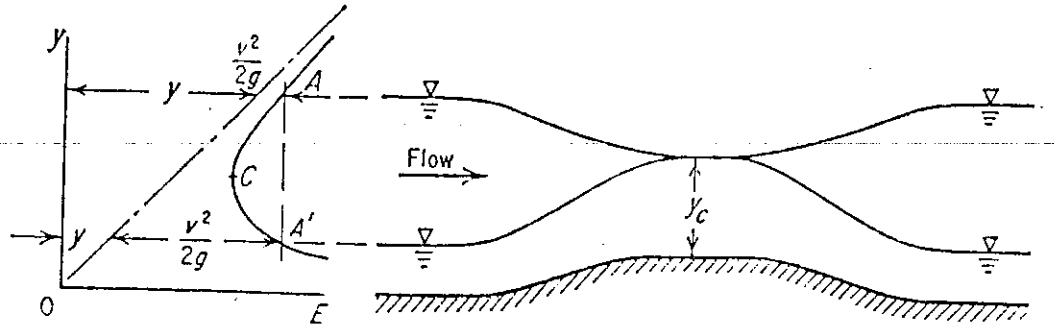

Figure 5. The Effect of a Short Upward Step of "Critical" Height

either supercritical or subcritical, depending on the downstream conditions. Referring back to the earlier discussion centered on Fig. 3 it can be seen that if the point representing flow moves from A to C, it is free thereafter either to move back up the subcritical limb of the curve, or to carry on round to the supercritical. If there is a downstream control such as a sluice gate the flow will tend to become subcritical, if not supercritical. If there is any doubt about downstream conditions, the tendency will be towards superciritical flow; the convergence of flow towards the step will tend to carry on into a further convergence downstream of the step. It seems that the same principle operates when the upstream flow is supercritical. Either of the upstream flow regimes in this figure can pass to either of the downstream regimes. Similar considerations apply in the case of a constriction in width.

The problem dealt with in this section is of considerable practical interest. It often happens that a local constriction must be introduced into a channel. e.g., to reduce bridging cost when the channel is being led under a highway. In such cases it is essential to know just how much constriction can be tolerated without influencing the flow upstream of the constricted section. The problem is simply that of sustaining a certain discharge for a given specific energy; the smallest section that can do this job is the one that will operate at critical flow, which gives maximum discharge for a given specific energy. Further, it has already been shown (for rectangular channels anyway) that if this

smallest section is provided, the flow will automatically take up the critical condition.

The problem is not, of course, confined to rectangular channels; a common case is that in which flow passes from a trapezoidal channel into a circular culvert under a roadway. The principle, however, is still the same: the limiting section is that which can take the required Q at the available E under critical flow conditions.

A constriction which is severe enough to influence the upstream flow becomes a special kind of control.

Example 3

Water flows at a velocity of 8 feet per second and a depth of 6 feet in an open channel of rectangular cross-section. If the channel width is reduced from 10 feet to 7 feet and the bottom elevation is increased 1 foot at a given section, to what extent will the surface elevation be affected by the boundary constriction?

The Froude number of the approaching flow is

$$F_1 = V_1/(gy_1)^{1/2} = 8/(32.2 \times 6)^{1/2} = 0.575$$

Since F_1 is less than unity, the boundary contraction should produce a drop in surface elevation if this does not require a lower specific head at the contracted section than that is

physically possible.

$$E = Y + \frac{V_1^2}{2g} = 6 + \frac{8^2}{2g} = 6.99 \text{ ft}$$

Since Q = VA and q = Q/b, in the contracted section

$$q_2 = \frac{V_1 A_1}{b_2} = \frac{8 \times 10 \times 6}{7} = 68.6 \text{ cfs/ft}$$

Hence, for this rate of flow

$$Yc = (q^2/g)^{1/3}$$

$$= (68.6^2/32.2)^{1/3}$$

$$= 5.26 \text{ ft}$$

and

$$E_{min} = \frac{3}{2} Yc = \frac{3}{2} \times 5.26 = 7.89 \text{ ft}$$

But this evidently requires a greater total head than that of the approaching flow. The contraction must therefore produce a backwater effect upstream, the entire surface rising until flow taken place at the critical depth (i.e.. minimum specific head) at the contraction. Under such circumstances $Y_2 = Y_c = 5.26$ feet and

$$\frac{Q^2}{2gA_1^2} + Y_1 = E_{min} + DZ$$

$$\frac{480^2}{2 \times 32.2(10Y_1)^2} + Y_1 = 7.89 + 1$$

Solution by successive approximation yields $Y_1 = 8.38$ ft, indicating that the contraction produces a 2.38 feet increase in upstream depth.

DISTINCTION BETWEEN THE HYDRAULIC GRADIENT AND ENERGY GRADIENT

The hydraulic grade line, or the hydraulic gradient, in open channel flow is the water surface, and in pipe flow it connects the elevations to which the water would rise in piezometer tubes along the pipe. The energy gradient is at a distance equal to the velocity head above the hydraulic gradient. In both open channel and pipe flow the fall of the energy gradient for a given length of channel or pipe represents the loss of energy by friction. When considered together the hydraulic gradient and energy gradient reflect not only the loss of energy by friction, but also the conversions between potential and kinetic energy.

In the majority of cases the end objective of hydraulic computations relating to flow in open channels is to determine the curve of the water surface. These problems involve three general relationships between the hydraulic gradient and the energy gradient. For uniform flow the hydraulic gradient and the energy gradient are parallel and the hydraulic gradient becomes

an adequate basis for the determination of friction loss, since no conversion between kinetic and potential energy is involved. In accelerated flow the hydraulic gradient is steeper than the energy gradient, and in retarded flow the energy gradient is steeper than the hydraulic gradient. An analysis of flow under these conditions cannot be made without consideration of both the energy gradient and the hydraulic gradient.

STEADY AND UNIFORM FLOW FORMULAS FOR OPEN CHANNELS

Many flow formulas have been proposed to determine the characteristics of flow. However, the Chezy and Manning relations are most commonly used.

Chezy's formula

Chezy's relation is one of the oldest velocity-resistance to flow relations in hydraulic engineering.

$$V = C(RS)^{1/2}$$

Where C is a boundary roughness coefficient.

Manning's formula

In 1889 Manning presented an equation to approximate the average velocity in open channels. This formula defines the resistance to flow. The most usual forms are:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad (12)$$

in metric units and

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

in English units

In the above equation n has the dimension of $TL^{-1/3}$. It is difficult to accept that n is a function of time.

In the above equation

V = Average velocity

R = Hydraulic radius = A/P

P = Wetted perimeter

S = Slope of the energy grade line

Using data from field observations of flows in rivers and large channels, both Gauckler in 1868 and Hagen in 1881 came up independently with the following relationships:

$$V = M R^{2/3} S^{1/2}$$

where M was a coefficient.

This formula was incorrectly attributed to the Irish Engineer, Robert Manning, by the Frenchman Flamant in 1891 when he published the equation, presently known as Manning's Equation in the form given in Eq. 12.

However, n not only changes with Reynolds number but is a function of:

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1) the depth of flow

2) the sediment transport (bed load, suspended load, and wash load)

3) floating debris

4) the geometry of the cross section

5) the geometric pattern of scouring and silting in the channel

6) the size of bed material

7) the type and extent of vegetation on the banks

8) the water temperature

9) wind direction and magnitude

In 1959 V.T. Chow published the detailed list of suggested n values some of which are given in the table below:

ROUGHNESS COEFFICIENT n (after Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
C Excavated or Dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.025	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035

Type of Channel and Description	Minimum	Normal	Maximum
c. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. Natural Streams			
D-1 Minor streams (top width at flood stage 100 ft)			
a. Streams on plain			
1. Clean,straight,full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean,winding,some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and section	0.040	0.048	0.055
6. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

Type of Channel and Description	Minimum	Normal	Maximum
D-2 Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050

CHAPTER I

DESIGN DATA CHECKLIST

H.R. KHAN

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I. Data checklist for the Design of Drainage Sluice/ Regulator/Water Retention Structures. (Standard BWDB Format)

II. Assessment of Discharge in Ungaged Rivers/Khals.

DATA CHECKLIST FOR THE DESIGN OF DRAINAGE SLUICE/ REGULATOR/WATER RETENTION STRUCTURE

While submitting proposal for the design of a Sluice, Regulator or Water Control Structure, this Data checklist shall be filled up and sent to the design office along with other requisites mentioned herein.

A. - Name of the Project/Scheme _____

_____

B. - Name of the Structure _____

_____

C. - Project Area _____

D. - Catchment Area above the Structure. _____

1. PURPOSE OF THE STRUCTURE.

Identify the purpose(s) that has to be served by the structure and put tick mark accordingly.

1.1 Pre-monsoon drainage

1.2 Monsoon drainage

1.3 Post-monsoon drainage

1.4 Prevention of early flood

1.5 Prevention of flood

1.6 Flushing of irrigation water

1.7 Retention of post-monsoon water for irrigation.

2. MAPS.

2.1 Project Index Map :

A project index map showing the project boundary and all project features such as embankment; improvement of drainage channel, irrigation canal; all drainage and irrigation structures; other structures etc.

2.2 Location Map :

A Location map in the scale of 1" = 500 ft showing the main and secondary drainage channel, outfall river, roads, embankments, inhabitations, location of the proposed structure and hydrological stations, position of cross-sections, etc.

2.3 Basin Map :

A Basin Map in the scale of 4" = 1 mile, preferably 8" = 1 mile showing the location of the proposed structure, main & secondary drainage channels and other water bodies (ponds, beels & haor, etc.), basin boundaries of the drainage channels for the proposed structure, internal and pheripherial roads, embankments with all existing regulators, sluices, bridges, culverts, breaches and other structures, spot levels contours, cultivated areas, homesteads, outfall river, location of hydrological station, etc.

2.4 Site Plan :

A plane table survey map in the scale of 1" = 50 feet covering an area of at least 1000 ft x 1000 ft showing spot levels at 25 ft interval, the proposed axis & centre-line of structure, original channel, diversion channels, alignment of embankment and closure, location of bore-holes for sub-soil exploration. The proposed centre-line of the structure and the reference line of survey should be fixed up by permanent pillars and those also should be shown on the site plan.

3. HYDROLOGICAL DATA.

3.1 Rainfall Data :

3.1.1 Is there any rainfall station within the catchment area? If so, specify the name(s) and length of records available :-

YES NO

Station No.	Name of Station	Length of records available	Remarks

3.1.2 Specify the name and length of records of the rainfall station close to the catchment of the proposed structure.

Station No.	Name of Station	Length of records available	Remarks

3.2 Water Level Data :

3.2.1 Is there any water level station on the outfall river at or near the structure site? If so mention the name of the station(s).

YES NO

Station(s) _____

_____

3.2.2 Specify the name of at least one station u/s and one station d/s of the structure site with distances :

U/s station _____

U/s distance _____ miles.

D/s station _____

D/s distance _____ miles.

3.2.3 Is there any water level station on the drainage channel corresponding the proposed structure? If so specify the name.

YES NO

Station(s) _____

3.2.4 Mention the highest flood level (H.F.L.) ever experienced in the basin indicating the source of the record.

H.F.L. (ever experienced) _____ SOB from gauge level at _____ Station/

From average public information.

3.3 Discharge Data :

3.3.1 Is there any record of discharge in the drainage channel? If so, enclose the data as available :

YES NO

Enclosed discharge data for the year of _____ at station _____.

4. MORPHOLOGICAL DATA.

4.1 (a) Are the banks of the outfall river and drainage channel at or near the structure site stable?

YES NO

(b) If not, show the movement of the bank in each year in a map.

Enclosed _____ No. of map.

(c) What is the average rate of erosion in each year?

Average rate of erosion _____ ft./year.

4.2 Cross-Section of the drainage channel for at least 1/2 mile u/s and 1/2 mile d/s of the structure site at an interval of 200 ft.

Enclosed _____ No(s) of Cross-Sections.

4.3 Long section of the drainage channel for at least 2 miles u/s from structure site and upto the outfall river in the d/s :

Enclosed Long Section in _____ No(s)

4.4 Cross-Section of the outfall river for a length of half mile u/s and 1/2 mile d/s from the confluence point of drainage channel and the outfall river.

Enclosed _____ No(s) of cross-section in _____ sheet(s).

5. MISCELLANEOUS DATA.

5.1 Data Related With Embankment/Road Connecting the Structures

5.1.1 Existing or proposed road/embankment profile for at least 500 ft on each side of the proposed structure.

Enclosed a profile in _____ No. of sheet(s)

5.1.2 Existing or proposed top elevation, top width and side slopes of embankment/road at the structure site.

(i) Top elevation _____ SOB

(ii) Top width _____ ft.

(iii) C/S. slope _____

(iv) R/S slope _____

5.1.3 Type of expected traffic loading on road/embankment :

_____ Loading.

5.1.4 General classification of the road :-

(a) R & H Road/District Council Road/
Thana Council Road/U.P. Road/Village Road

(b) Highway/Feeder Road/Local Road

5.2 Data Related With Drainage Aspects :

5.2.1 Are the existing section and bed slope of the drainage channel adequate for complete or desired level of drainage?

YES NO

5.2.2 If not, does the scheme include re-sectioning of the drainage channel? If so, the designed cross-section and long section may be furnished as requirement indicated in para 4.2 and 4.3 :-

YES NO

Enclosed _____ No. of sheet(s)

5.2.3 (a) Is the complete drainage of the basin necessary?

YES NO

(b) If not, mention the drainage level required & distance of such level from the proposed structure site :

Drainage EL. requirement _____ SOB

Distnace from the structure site _____ mile(s)

5.2.4 From the field condition propose the invert level of the structure which can allow desired level of drainage.

Proposed Invert Level _____ SOB

5.2.5 From field condition, what is the maximum level of acceptable flooding on the basin during drainage period?

Acceptable flooding level _____ SOB

5.2.6 Desired Post Monsoon Drainage Level :

Date _____ Level _____ SOB

Date _____ Level _____ SOB

Date_____ Level _____ SOB

5.3 Data Related With Irrigation Aspects :

5.3.1 Specify the total cultivable and irrigable area within the project.

Cultivable Area _____ Acres.

Irrigable Area _____ Acres.

5.3.2 Principal crops with acreage in the basin with present and future cropping pattern.

Enclosed in _____ number of sheet(s)

5.3.3 Proposed retention level of water in the u/s of the drainage channel of the structure for irrigation.

Retention level _____ SOB

5.3.4 For Irrigation by flushing of water, specify the period of such Irriation.

Period from _____ To _____.

6. FOUNDATION DESIGN DATA.

Submit the soil boring report.

CHAPTER II

HYDROLOGIC DESIGN

A. SIMPLE METHOD

- Using Graphs & Tables

B. ELABORATE METHOD

- Using Unit Hydrograph Method
- Using Richard's Method
- Using SCS Method

HYDROLOGIC DESIGN

A. SIMPLE METHOD

- Using Graphs & Tables

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1.0 INTRODUCTION :

The purpose of Hydrologic Design is to 1st determine the peak flow from the catchment area to be handled by a hydraulic structure. The peak flood flow can either be determined by simple method using graphs and tables or by more elaborate computations using unit hydrograph method or by comparatively easier method using Richard's method or by using Soil Conservation Service (SCS) method. For a small catchment area say 2 to 4 sq. mile, where from our previous experience we can say that a one vent regulator of size 5'x6' will be enough to drain the basin efficiently, in that case we may not go for elaborate computations. But for a moderate to large catchment area selection of ventage should be made more precisely by using unit hydrograph method or by Richard's method or by SCS method and finally by flood routing operation the exact size and number of vents required to drain the basin without exceeding the tolerable limit of flooding should be obtained.

Two simple methods have been developed especially for this region for estimating peak flow of a drainage basin. One has been developed for India and the other for Bangladesh. Both are simple to apply and they provide comparable results.

These method can be used to estimate the peak runoff for a certain return period for small catchments. The first has been developed by IECO (for the Karnafuli Irrigation Project Area) and the 2nd is a method proposed by Desai and Thirmalai two Indian Engineers. The methods are described below :

2.0 IECO METHOD :

Three graphs have been prepared for use in estimating flood discharges for return periods from 2 to 500 years. The approach is as follows :

(i) Obtain the total annual precipitation for the project area and enter Figure 1 to obtain the Q_{100} unit peak flow. Multiply this value by the coefficient given in the table on Figure 1 to obtain the peak unit flow for the selected return period (10 years for sluices and water retention structures).

(ii) Knowing the length of the main water course and the slope of the basin compute $(L^2/S)^{0.3}$ and enter Figure 2 to obtain the percentage adjustment to make to the peak flood rate.

(iii) Enter Figure 3 with the basin area and obtain the percentage error as a function of basin area.

(iv) Multiply the unit peak flow by the values obtained in (ii) and (iii) to arrive at the unit peak flow.

(v) Multiply the value obtained in (iv) by the basin area to obtain an estimate of the peak flood flow for the basin.

3.0 SAMPLE EXAMPLE : (IECO METHOD)

Basin Area = 4 sq. mile

Length of water course, L = 2 mile

Slope of water course, S = 0.722 ft/mile

= 0.0001367 ft/ft

Let the basin is located in Jessore district.

From Fig. 1 Total Annual rainfall = 72" (6 ft)

q_{100} = 130 cfs per sq mile.

q_{10} = 130 x 0.73 = 95 cfs per sq mile.

$$\frac{L^2}{S}^{0.3} = \frac{4}{0.0001367}^{0.3} = 21.87$$

From Fig. 2 The adjustment is - 40%

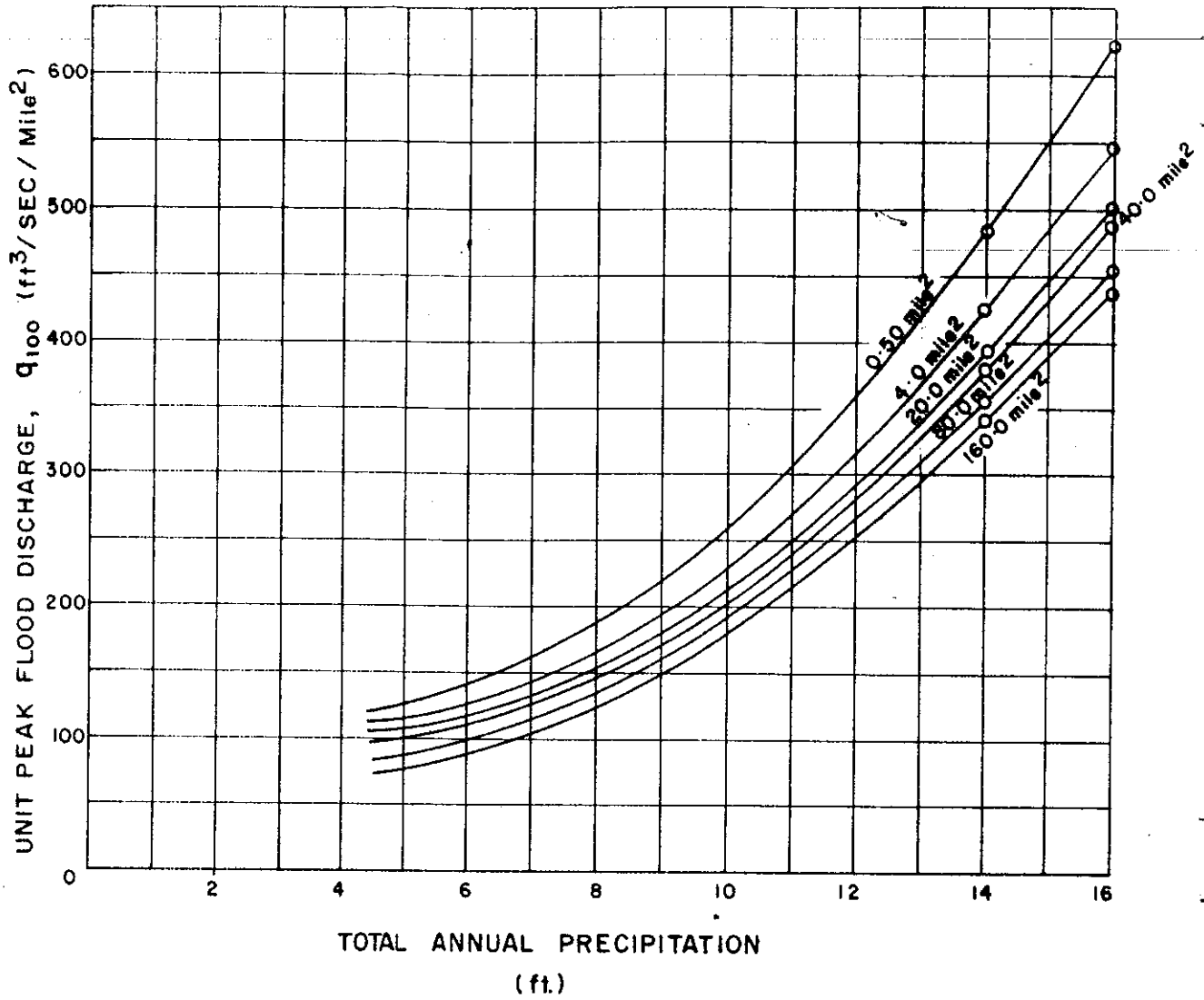
q_{10} = 95 - 38 = 57 cfs/sq mile

From Fig. 3 The correction is + 66%

q_{10} (corrected) = 57 x 1.66 = 95 cfs/sq mile

For 4 sq miles

Q = 95 x 4 = 380 cfs.



Factor for Other Flood Frequencies:

$q_{500}/q_{100} = 1.14$
$q_{200}/q_{100} = 1.06$
$q_{50}/q_{100} = 0.93$
$q_{25}/q_{100} = 0.85$
$q_{15}/q_{100} = 0.79$
$q_{10}/q_{100} = 0.73$
$q_{5}/q_{100} = 0.62$
$q_{2}/q_{100} = 0.40$

Region	Rainfall
Rangpur	7.5 feet
Kurigram	7.5 ,,
Chittagong	9.2 ,,
Cox's Bazar	12.5 ,,
Dacca	6.7 ,,
Khulna	5.8 ,,
Sylhet	9.2 to 16.7
Mymensingh	8.0 ,,
Barisal	7.5 ,,
Jessore	6.0 ,,

FIGURE I. UNIT PEAK DISCHARGES FOR 100 YEAR RETURN FREQUENCY

Source: Karnafuli Irrigation Project
International Engineering Co. Inc. (May 1980)

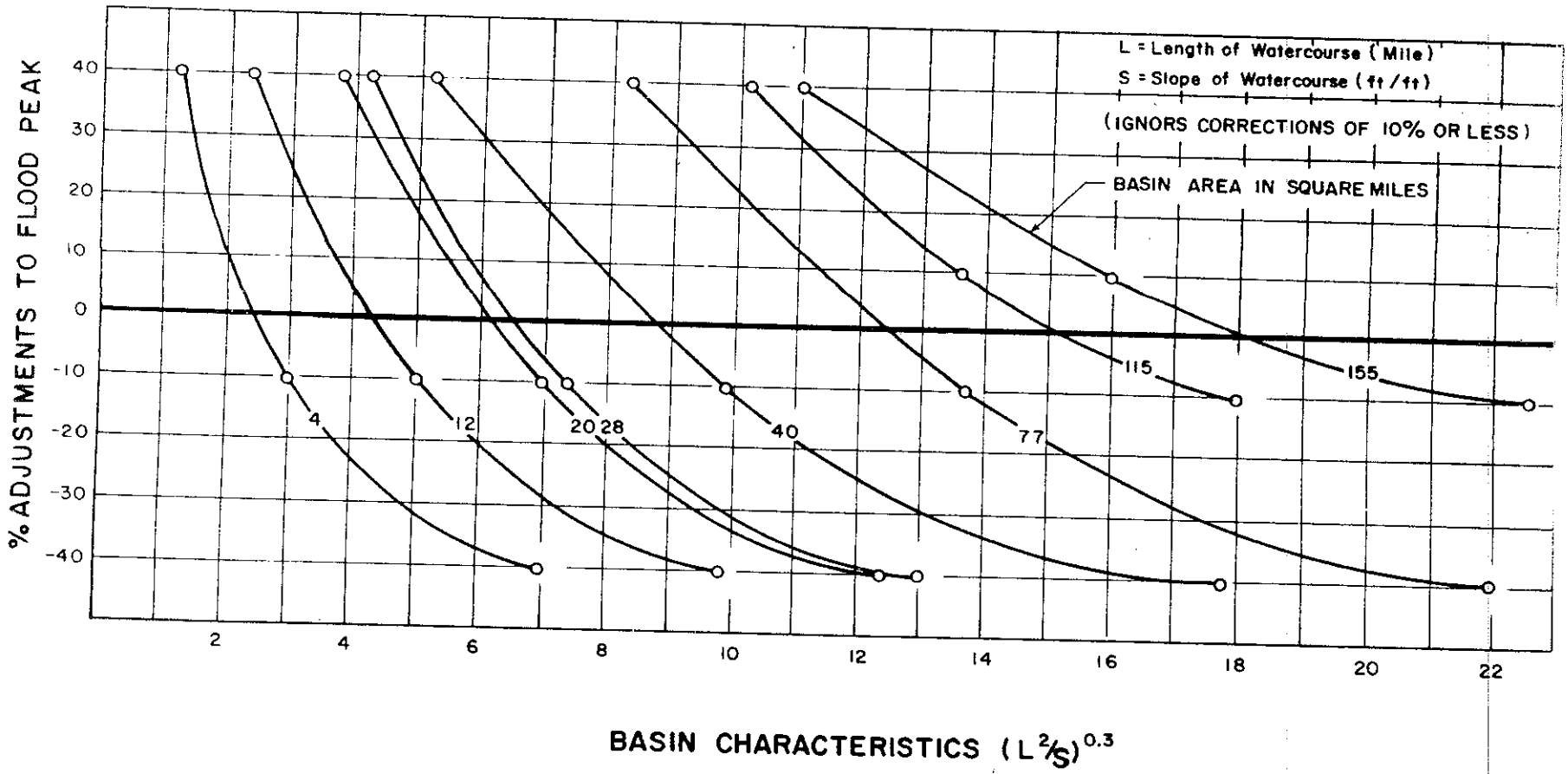


FIGURE 2. ESTIMATED PEAK ADJUSTMENTS FOR BASIN SLOPE AND SHAPE CHARACTERISTICS.

Source: Karnafuli Irrigation Project
International Engineering Co. Inc. (May 1980)

$$\text{PERCENT ERROR} = \frac{Q_{25} \text{ from stream Records} - Q_{25} \text{ Regional analysis}}{Q_{25} \text{ Regional analysis}} \times 100$$

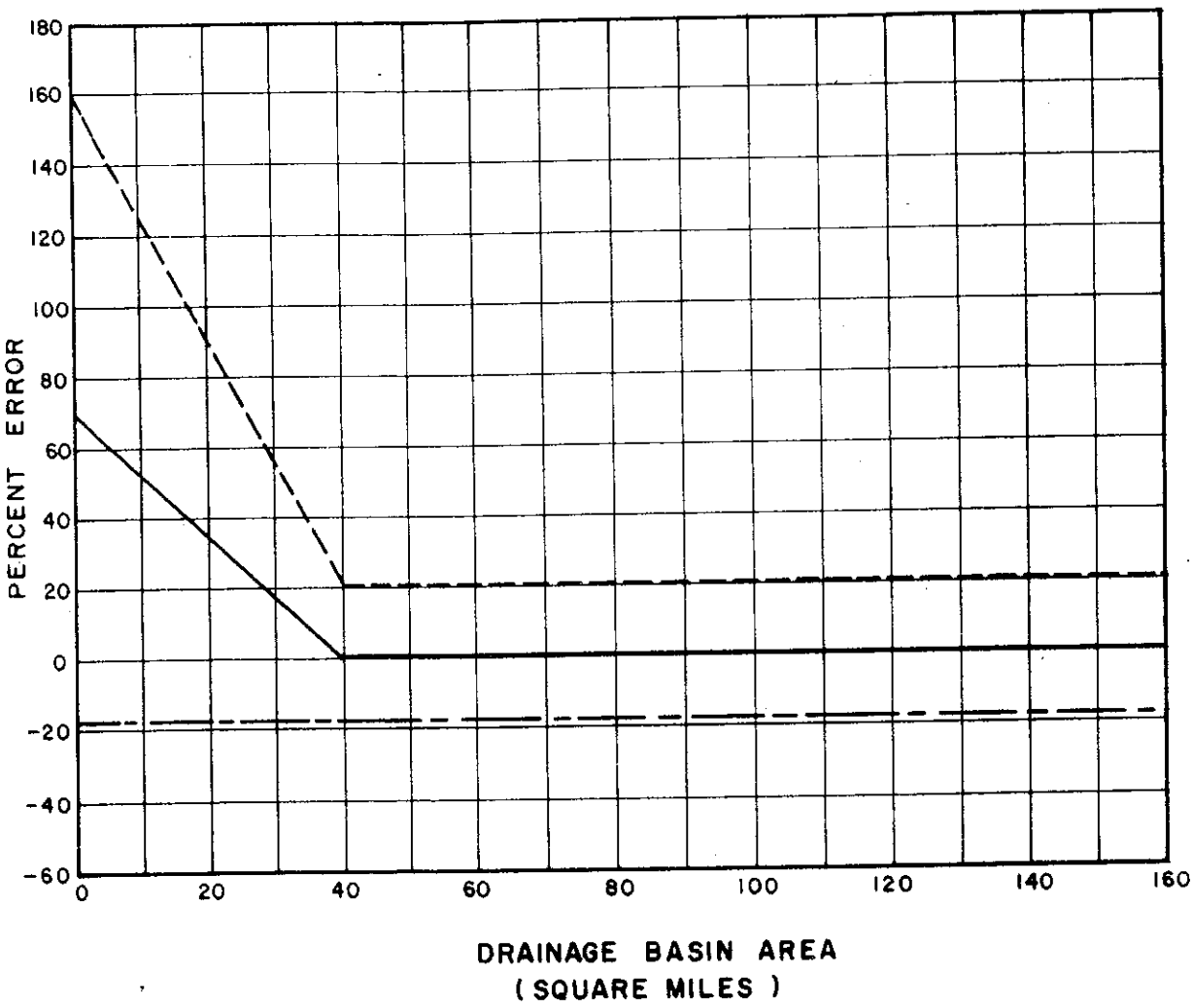


FIGURE 3. ERROR AS A FUNCTION OF DRAINAGE AREA

Source: Karnafuli Irrigation Project
International Engineering Co. Inc. (May 1980)

4.0 DESAI'S AND THIRMALAI'S METHOD :

A simple approach for determining the design floods for small catchments has been proposed by M.W. Desai and S. Thirmalai two engineers of the Research Design and Standards Organization, Ministry of Railways in India. A set of graphs and charts have been developed using data from 92 bridge locations throughout India. With the aid of rating curves established for each site the highest flood level has been obtained by extra-polation. This discharge is then related to the 50 year 1 hour point rainfall, the average land slope and the catchment area and a series of curves produced. The authors state that the graphs, although prepared with limited data, give fairly satisfactory results when compared with more elaborate methods and provide a means of obtaining a first estimation of design floods for small catchments.

The method has been developed for use in India but because climatic and topographic conditions in Bangladesh are quite similar to those in India the method can be used here, either separately or in conjunction with other methods, to estimate the 5,10,25 and 50 year flood discharges of small watersheds.

The step to be taken to arrive at the design floods for various return periods are as follows :

STEPS

1. Define the catchment area and planimeter its perimeter to obtain the drainage basin area.

2. Estimate the 2 year 1-hour rainfall for the basin from Figure 4.

3. Select the zone in which the basin lies (Figure 4). If the basin lies outside the boundaries of the established zones then choose the zone which most closely represents climatic and precipitation condition in the basin.

4. Obtain the multipliers for the various n-year 1-hour rainfall (Figure 4).

5. Enter Table 1 and obtain an estimate for K.

6. Select from Figure 5 the approximate K value curve and obtain the discharge for the basin area.

5.0 SAMPLE EXAMPLE : (Desai's and Thirmalai's Method)

Basin Area = 4 sq mile

Let the basin is located in Jessore district.

Extending the rainfall Isohyetal lines & the Zonal boundary in Figure 4. We get 2-year 1-hour rainfall of the basin = 61 mm & the basin falls in Zone II.

From Table of Fig. 4

10-year 1-hour rainfall = 61 x 1.46 = 89 mm (3.5")

From Table I

K = 0.25

From Fig. 5 for K = 0.25 & Basin Area = 4 sq miles

Q = 430 cfs.

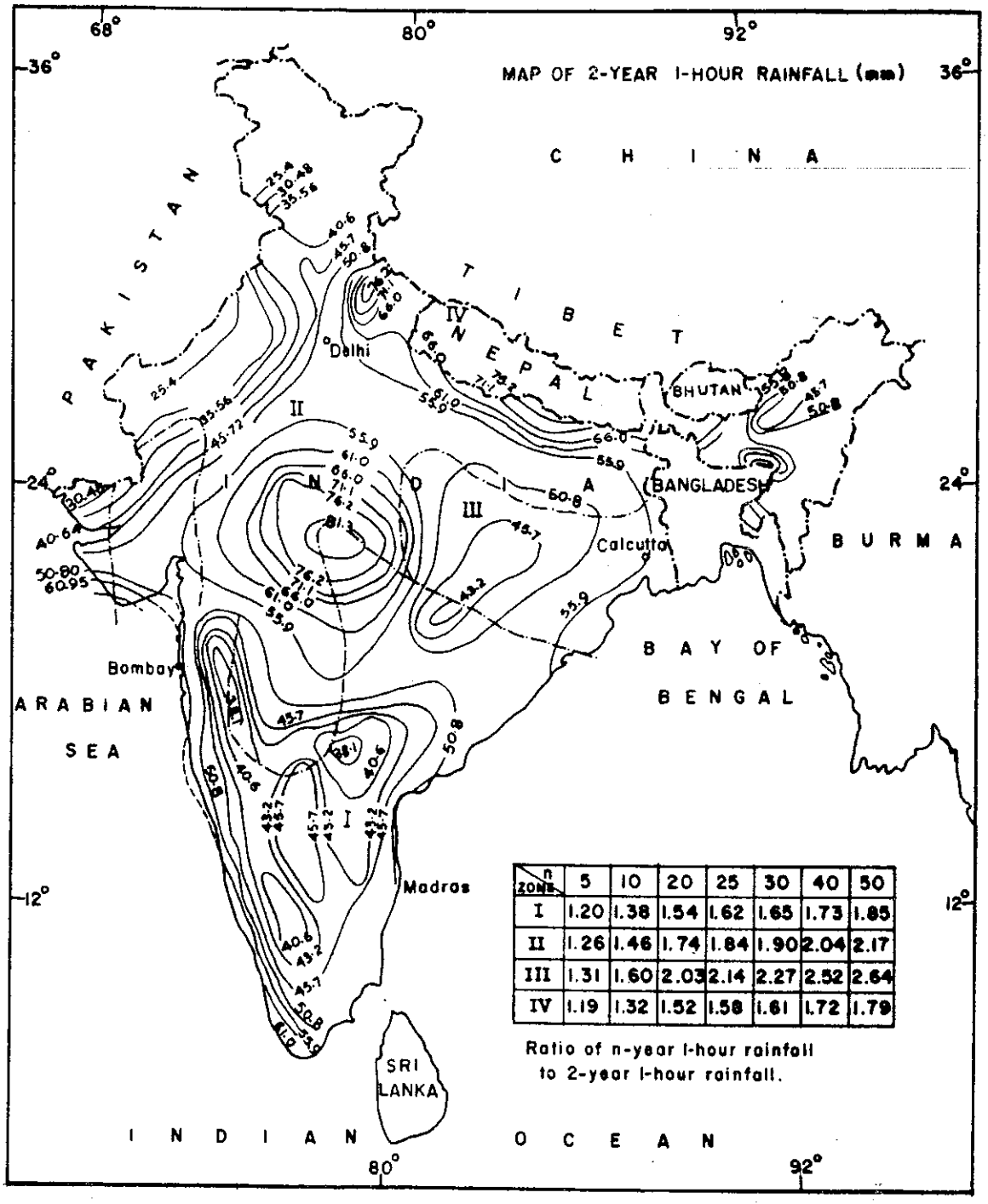
TABLE 1.

"K" VALUES FOR VARIOUS TERRAIN

n-year 1-hour Rainfall		Statistical Slope					Ahmadnagar-Vikrabad plateau and Ajanta range and for similar areas		
		Hilly 10% and above	Rough hilly 5% - 10%	Rolling 2% - 5%	Fall to 1-2%	Fall to 0-1%	5% - 10%	2% - 5%	0 - 1%
(mm)	(inches)								
64	2.5	-	-	0.80	0.30*	0.15*	1.10	-	-
76	3.0	0.92	0.90	0.85	0.50*	0.20*	1.15	1.0	-
89	3.5	0.93	0.92	0.90	0.60*	0.25*	1.18	1.1	-
102	4.0	0.95	0.94	0.92	0.70*	0.30	1.21	1.2	1.10
114	4.5	1.10	0.96	0.95	0.90*	0.35	1.22	-	1.20
127	5.0	1.20	1.06	1.00	0.95	0.40*	-	-	-
140	5.5	1.30	1.28	1.10	1.00		-	-	-
152	6.0	1.35	1.32	1.15			-	-	-
165	6.5	1.34	-	-	-	-	-	-	-

*Values suggested

Source : Desai, M.W., Thirumali, S., A Simplified Approach for Determination of Design Floods for Small Catchments.



ZONE \ n	5	10	20	25	30	40	50
I	1.20	1.38	1.54	1.62	1.65	1.73	1.85
II	1.26	1.46	1.74	1.84	1.90	2.04	2.17
III	1.31	1.60	2.03	2.14	2.27	2.52	2.64
IV	1.19	1.32	1.52	1.58	1.61	1.72	1.79

Ratio of n-year 1-hour rainfall to 2-year 1-hour rainfall.

FIGURE 4. MAP OF 2-YEAR 1-HOUR RAINFALL (mm).

Source: A Simplified Approach for Determination of Desig floods for Small Catchments. by M.W. Desai & Thirumali

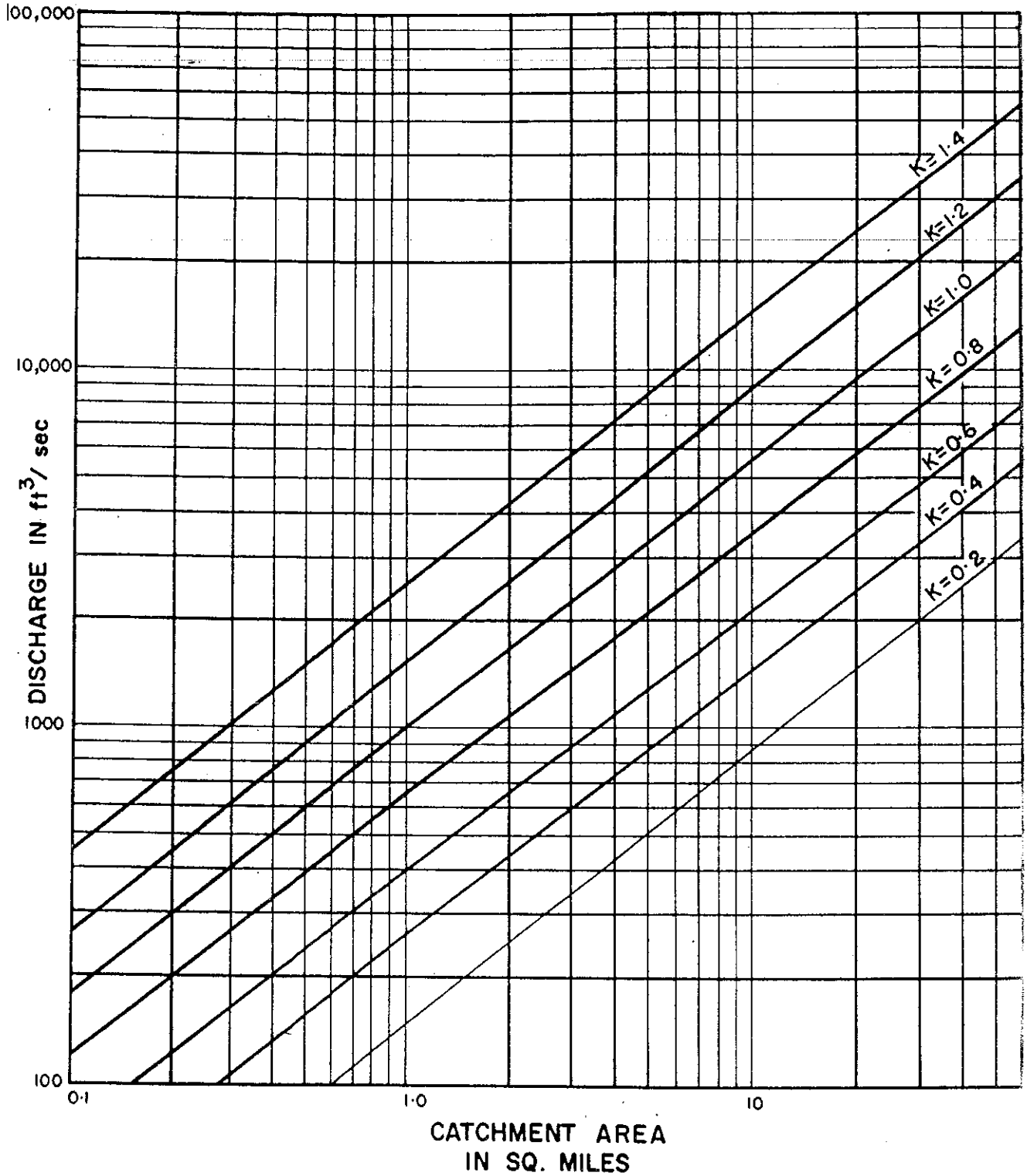


FIGURE 5. DISCHARGES FOR VARIOUS 'K' VALUES.

Source: A Simplified Approach for Determination of Design Floods for Small Catchments, by M. W. Desai & Thirumali

6.0 DRAINAGE MODULUS :

Drainage Modulus or the drainage rate of the basin expressed in inch or mm. per day is used to determine the design discharge of the drainage channel. During monsoon the main & seconday drainage canals sometimes even the tertiary canals remain flooded by the rainwater stored in the basin & the monsoon drainage is not effective due to high river stage unless drainage pumps are installed. During monsoon only this channels lying above basin inside storage level will be effective. These channels are mostly farm ditches or minor channels which are usually developed by the farmers and do not require much detail in design.

During pre-monsoon period i.e, in May and June the low stage of the outfall river will permit gravity drainage and the drainage will be effective through main and secondary drains. As such the pre-monsoon storm i.e, the storm for May or June will be used to determine the drainage modulus of a basin.

7.0 SAMPLE EXAMPLE : (Drainage Modulus)

Rainfall station : Narail

1 in 10 year Design Rainfall (in inch)

Days ---->

1	2	3	4	5	6	7	8	9	10
5.12	1.18	1.14	0.91	0.91	0.75	0.63	0.63	0.47	0.39

Rainfall losses due to evapotranspiration & deep percolation = 0.2"/day.

Rainfall Excess

Days ---->

1	2	3	4	5	6	7	8	9	10
4.92	0.98	0.94	0.71	0.71	0.55	0.43	0.43	0.27	0.19

Cumulative

4.91	5.90	6.84	7.55	8.26	8.81	9.24	9.67	9.94	10.13

Assumptions :

1) Pre-monsoon storm occured in the month of May when paddy height is 12"
2) Allowable initial paddy storage = 2"
3) Maximum tolerable depth of flood for paddy in the month of May = 6"

In Figure 6. the excess rainfall is plotted over allowable paddy storage and the 1 in 10 year Flood curve is drawn. Now to keep the basin water level within tolerable depth of flooding for paddy, the excess rainfall runoff over 6" in the basin have to be drained. To find the design discharge of a drainage channel, the drainage modulus of the basin is found by drawing a tangent to the flood curve from the 6" point of the rainfall axis. The slope of the tangent line represents the drainage modulus of the basin. To find the value of the drainage modulus a line parallel to the tangent line passing through the origin is drawn. The parallel line to the tangent shows that 1" of rain has to be drain in 1 day i.e, the drainage modulus of the basin under the influence of Narail rainfall station is 1 inch per day.

In absense of detail rainfall records the drainage modulus or the design runoff of a basin to determine the design discharge of a drainage channel can be readily obtained from the isohyetal map presented in Fig. 7. The map has been prepared by LDL based on rainfall intensity, duration and frequency data published in the Master Plan by IECO.

The drainage modulus thus obtained may reduce to some extent if the maximum rainfall frequency duration is considered over an increasingly larger area. A study on drainage modulus was conducted in the Chao Phya delta in Thailand. The results of the study showed that the drainage modulus reduces for larger areas. (Ref. Design & Management of Drainage systems - Drainage of Rice Fields by G.A.W. Vande Goor). The percentage of reduction of drainage modulus will vary from basin to basin due to variation of vagitational growth, which has to be determined based on the experimental results. However for small areas the drainage modulus can be directly used to determine the design discharge of a drainage channel.

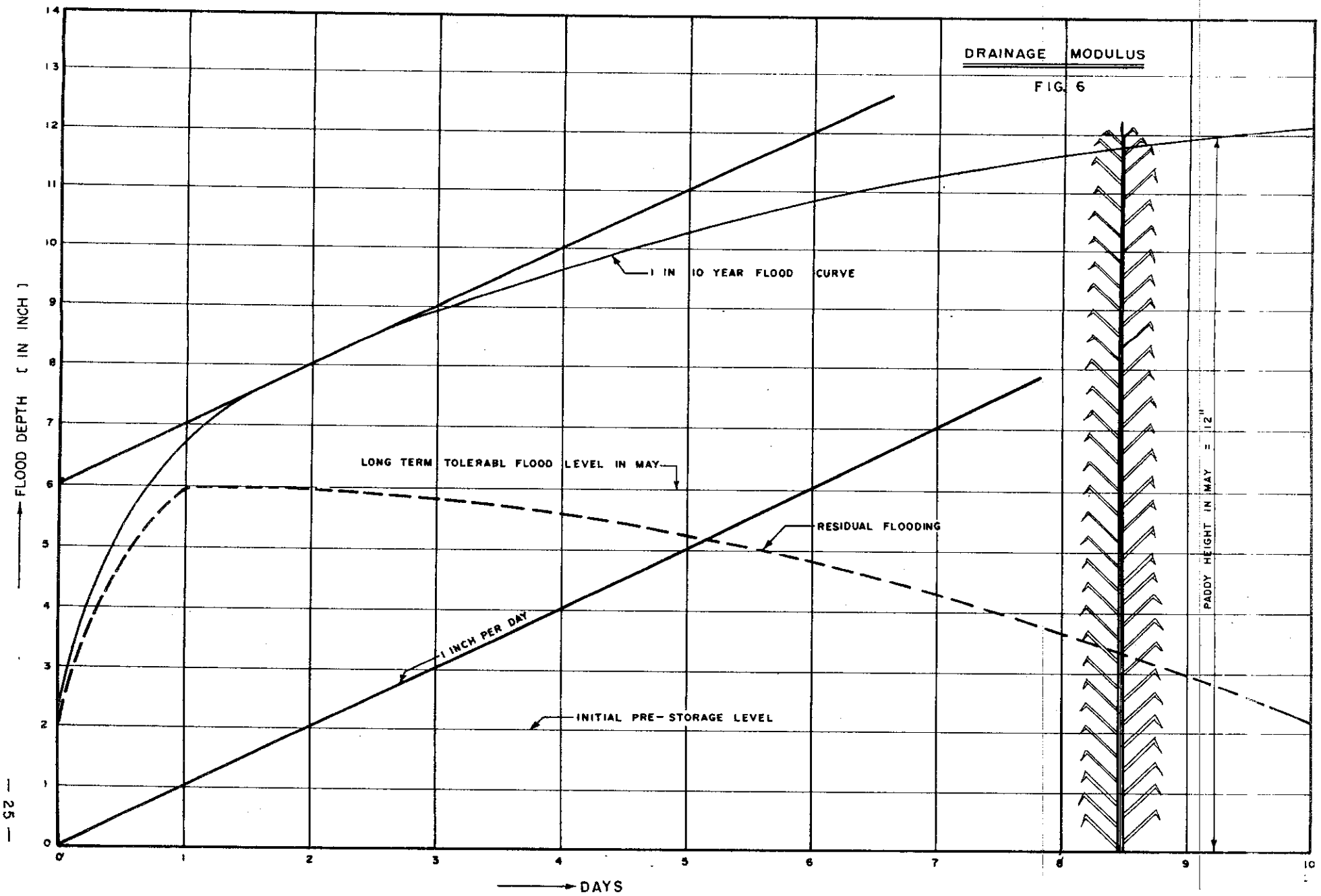
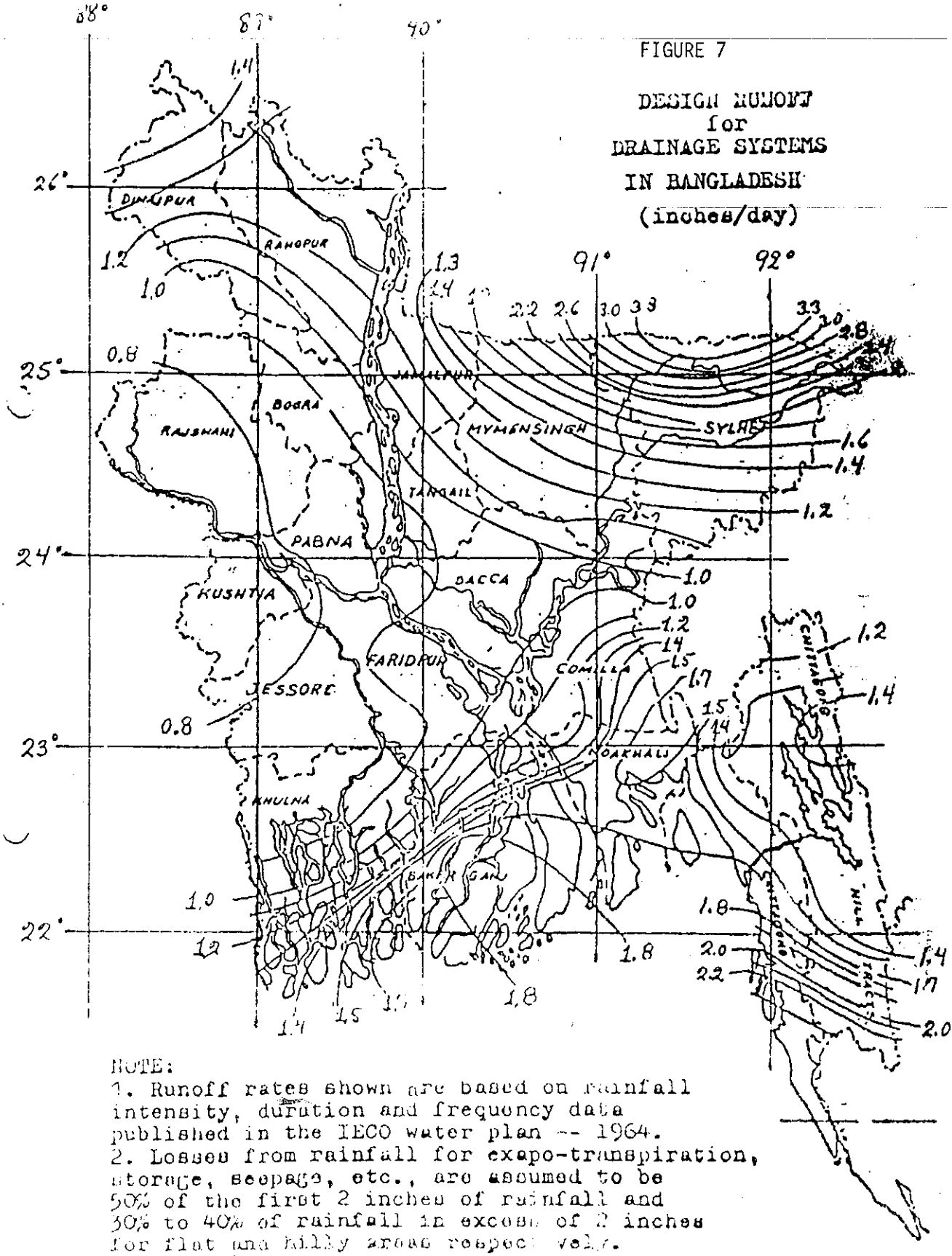

DRAINAGE MODULUS
FIG. 6
1 IN 10 YEAR FLOOD CURVE
LONG TERM TOLERABL FLOOD LEVEL IN MAY
RESIDUAL FLOODING
1 INCH PER DAY
INITIAL PRE-STORAGE LEVEL
PADDY HEIGHT IN MAY = 12"
FLOOD DEPTH [IN INCH]
DAYS
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14

FIGURE 7

DESIGN RUNOFF
for
DRAINAGE SYSTEMS
IN BANGLADESH
(inches/day)



NOTE:
1. Runoff rates shown are based on rainfall intensity, duration and frequency data published in the IECO water plan -- 1964.
2. Losses from rainfall for exapo-transpiration, storage, seepage, etc., are assumed to be 50% of the first 2 inches of rainfall and 30% to 40% of rainfall in excess of 2 inches for flat and hilly areas respectively. (ADL 1968)

HYDROLOGIC DESIGN

B. ELABORATE METHOD

- Using Unit Hydrograph Method
- Using Richard's Method
- Using SCS Method

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1.0 INTRODUCTION

The flood wave moving through a short reach of channel of regular section will undergo little change in its configuration. However, if the flow is obstructed in any way the flood wave configuration may be modified appreciably. The determination of this modification is called flood routing. Flood routing in our case will consist of predicting an outflow hydrograph below a drainage structure from a known inflow hydrograph above the structure.

Flood routing serves several purposes.

a) The maximum discharge under the outflow hydrograph is used as a basis for designing the structural elements that pass the flow. In the case of sluices this include the size and number of vents, and the dimensions and proportions of the stilling basin elements.

b) Because the inflow discharge during the flood peak is greater than the outflow discharge, a portion of runoff becomes storage. Flood routing will determine the accumulated storage volume and by using a basin capacity curve the storage water surface elevation can be found. This information is used to compute crop damages due to flooding.

c) The structure is designed for percolation and uplift using the head differential across the structure measured from the storage level to the tail water level as determined by the outflow hydrograph.

Generally a routing will be made for a pre-monsoon storm, when the river stage is low, as criteria for designing the structural elements.

Before the flood routing process can begin, the designer must estimate a preliminary size of structure required. In a non-tidal area roughly for each 5 to 6 sq. mile of the catchment area a single vent of size 5'x6' may be assumed. The routing procedure will determine whether the assumed structure is adequate after computing the extent of flood damage. Additional routing may be required through an enlarged or reduced structure depending upon the results of flood damage computed from the first routing. The range of tolerable submergence of area may be as high as 10% or even more of the gross project area depending on the topography of the basin. The designer will apply his judgement after studying the basin topography and importance of land, that how great an area can be flooded and to what depth. In general the tolerable range of submergence of area for not more than 12" for a period of 72 hours may be considered upto an incremental area of 5% of the total project area in addition to the existing low lands that can not be drained by gravity.

2.0 COMPUTATION OF DESIGN STORM

MRIGI KHAL BASIN

The Mrigi Khal basin is located in Fig. 1 to find the 4 month rainfall index of the project area which is 50". The 4 month index is converted to total rainfall over a selected number of days and the index frequency is again converted to a selected frequency with the help of two charts represented by Fig. 2 and Fig. 3. For our study we require a rainfall duration no longer than 5 days and frequencies of either 10-years or 25-years. These two charts are combined and the results are given in a tabular form in Table-1.

TABLE-1

COMBINED RAINFALL INDEX
FOR SELECTED FREQUENCIES

DAYS	STORM FREQUENCIES	
	10 YEARS	25 YEARS
1	0.128	0.153
2	0.192	0.228
3	0.230	0.272
4	0.257	0.303
5	0.276	0.326

A postmonsoon routing, when the discharge is controlled by the tail water level due to high river stage, will also be required to check the size of the structure selected by pre-monsoon routing. The criteria for checking the adequacy of the size is to observe that the difference of water level between R/s and C/s cannot be visually identified. This difference of water level is considered to be 6" to 9" which is not easily detectable in the field to a maximum of 12" for three consecutive days. The farmer will then be satisfied with the design provision and will not insist for extra ventage. The tendency for cutting off the embankment for quicker drainage of the area will also be stopped.

The flood routing procedure have been illustrated with the following example on Mrigi Khal regulator under Mrigi drainage and water conservation scheme, Faridpur. Computations of pre-monsoon flood routing have been made with basin runoff hydrographs generated both by Unit Hydrograph Method, Richard's Method & SCS Method. The computed results are discussed in Art. 16.0.

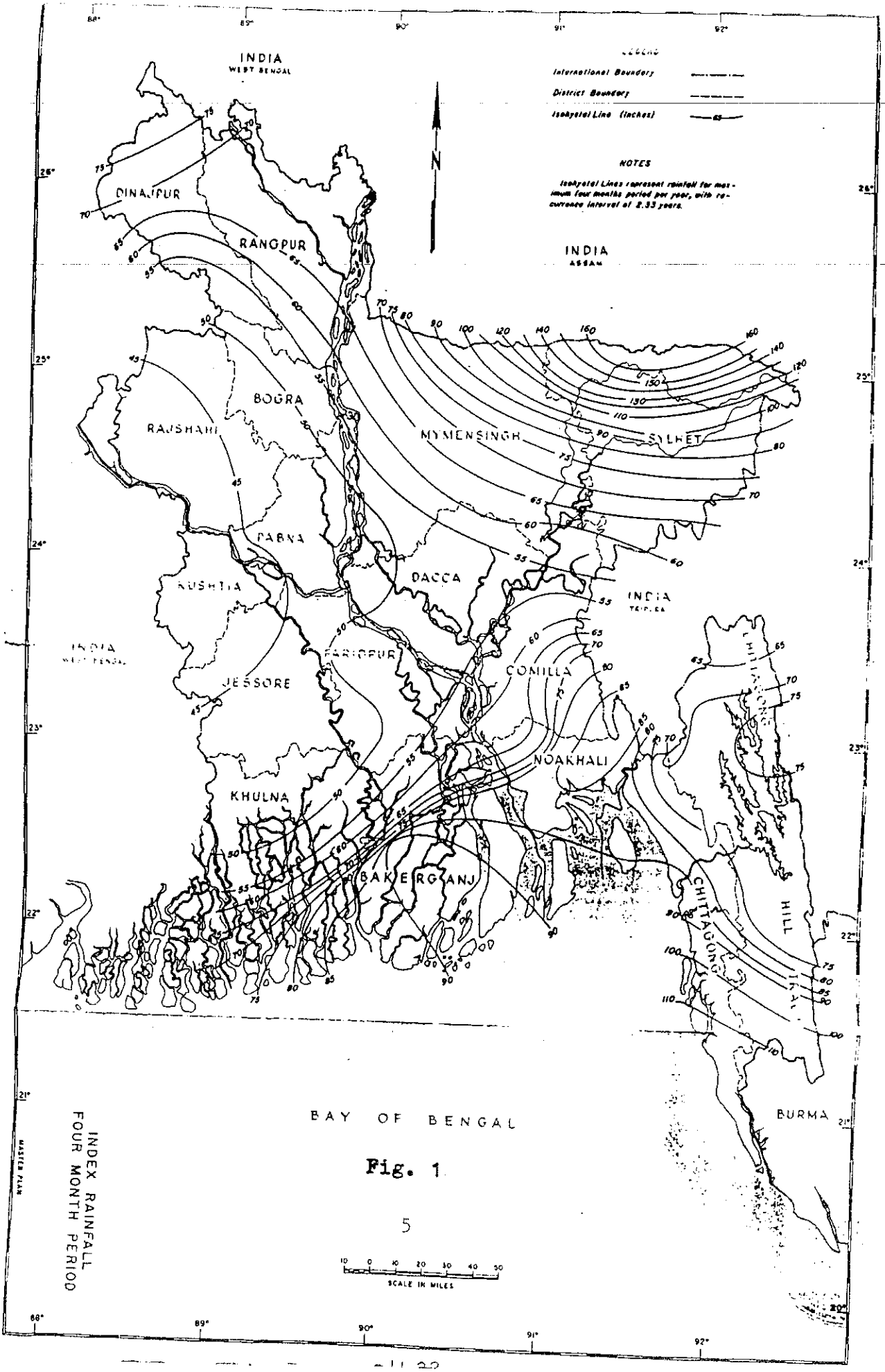

INDIA
WEST BENGAL
LEGEND
International Boundary
District Boundary
Isohyetal Line (Inches)
NOTES
Isohyetal Lines represent rainfall for maximum four months period per year, with recurrence interval of 2.33 years.
N
INDIA
ASSAM
DINAJPUR
RANGPUR
BOGRA
RAJSHAHI
MYMENSINGH
SYLHET
PABNA
KUSHTIA
DACCA
INDIA
TRIPURA
INDIA
WEST BENGAL
FARIDPUR
COMILLA
JESSORE
NOAKHALI
KHULNA
BAKERGANJ
CHITTAGONG
HILL
TRACT
BURMA
BAY OF BENGAL
INDEX RAINFALL
FOUR MONTH PERIOD
MASTER PLAN
SCALE IN MILES

Fig. 1

Figure 2.

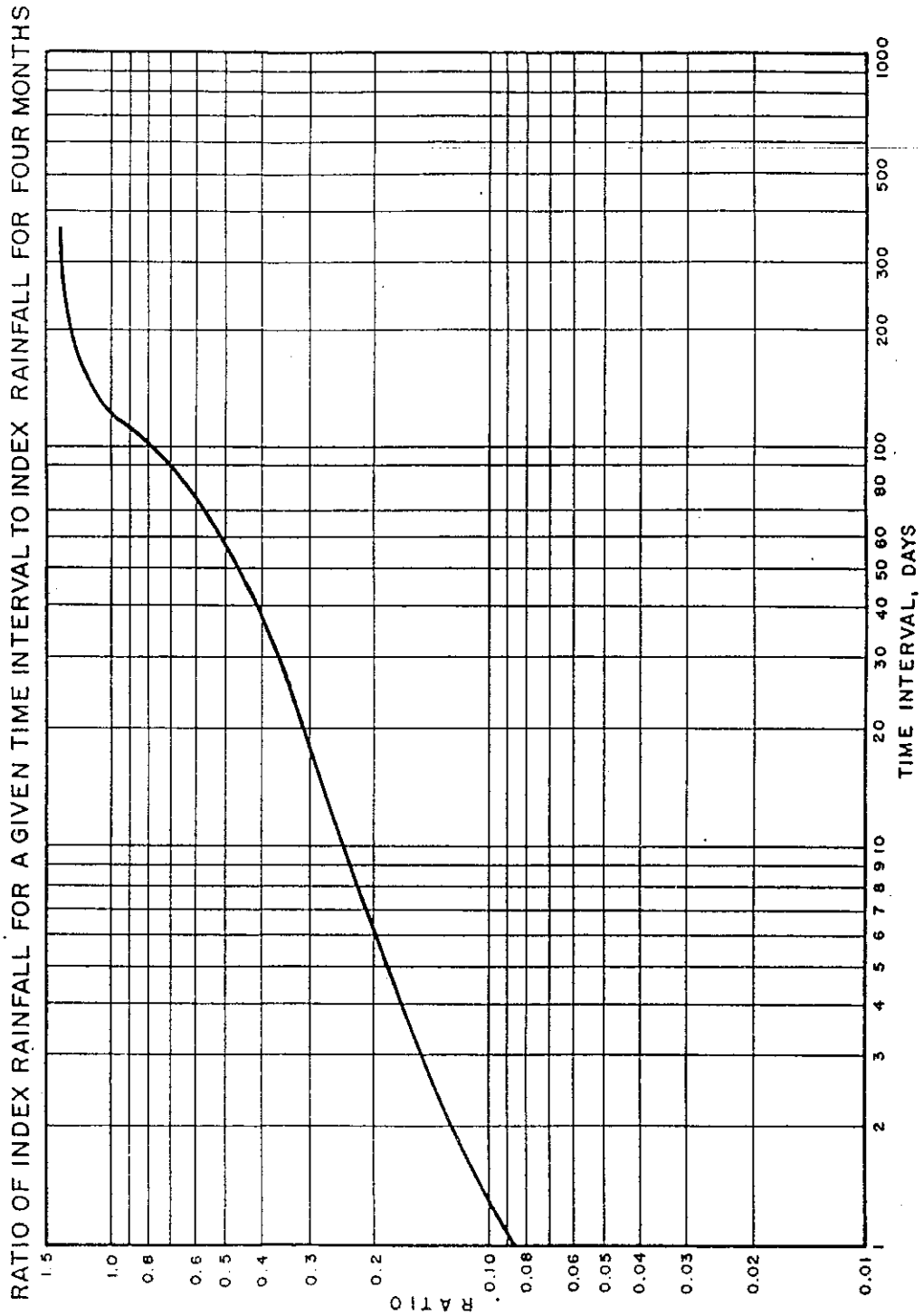
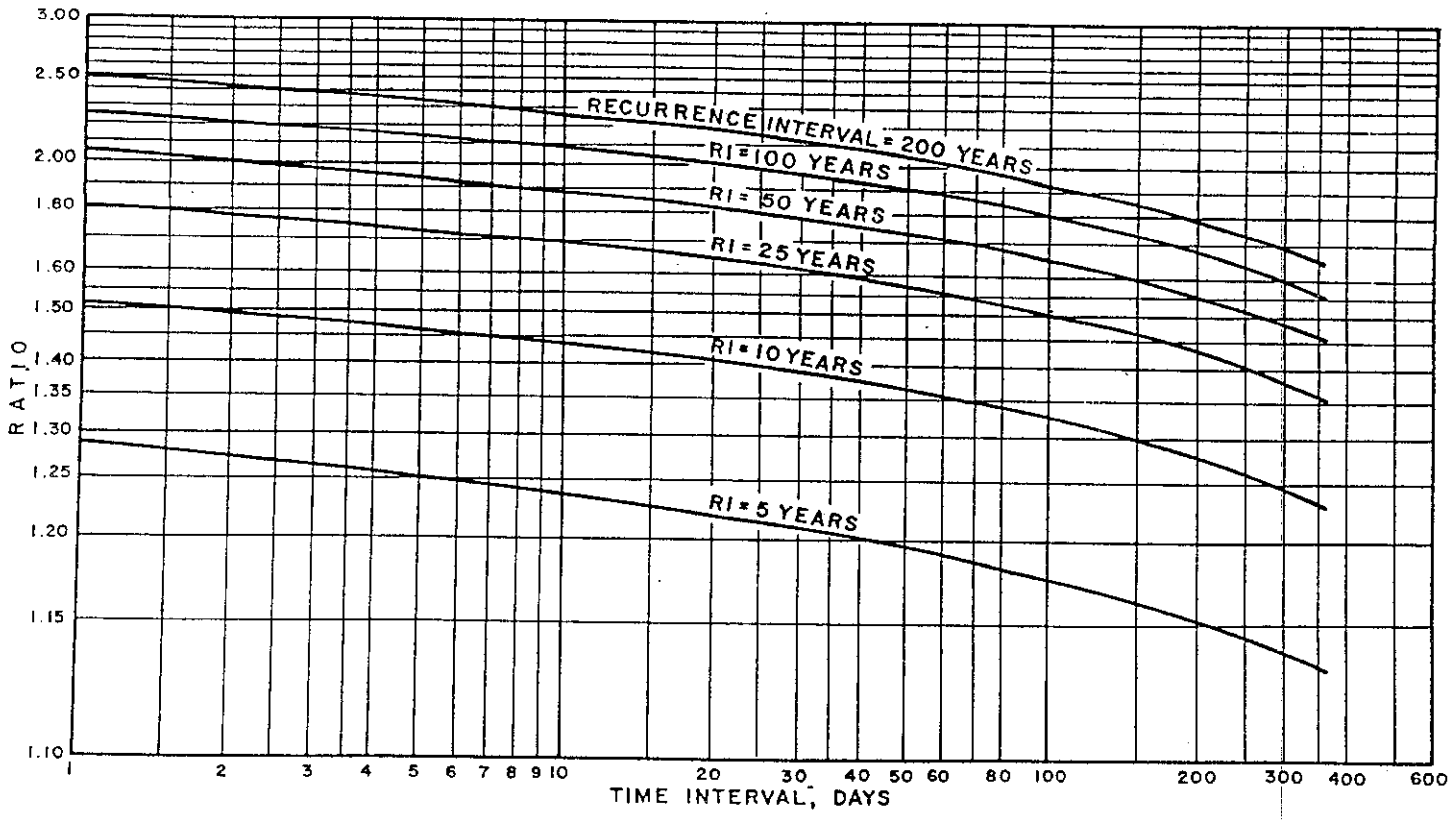

RATIO OF INDEX RAINFALL FOR A GIVEN TIME INTERVAL TO INDEX RAINFALL FOR FOUR MONTHS
RATIO
1.5
1.0
0.8
0.6
0.5
0.4
0.3
0.2
0.10
0.08
0.06
0.05
0.04
0.03
0.02
0.01
TIME INTERVAL, DAYS
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
80
100
200
300
500
1000

Figure 3.

RATIO OF RAINFALL FOR SELECTED RECURRENCE INTERVALS
TO INDEX RAINFALL FOR INDICATED TIME INTERVAL



2.1 RAINFALL VARIABILITY

The rate of decrease of storms intensity away from the centre of storm for Bangladesh has been shown in Fig. 4 whereas Table 2 represents the relationship in the tabular form.

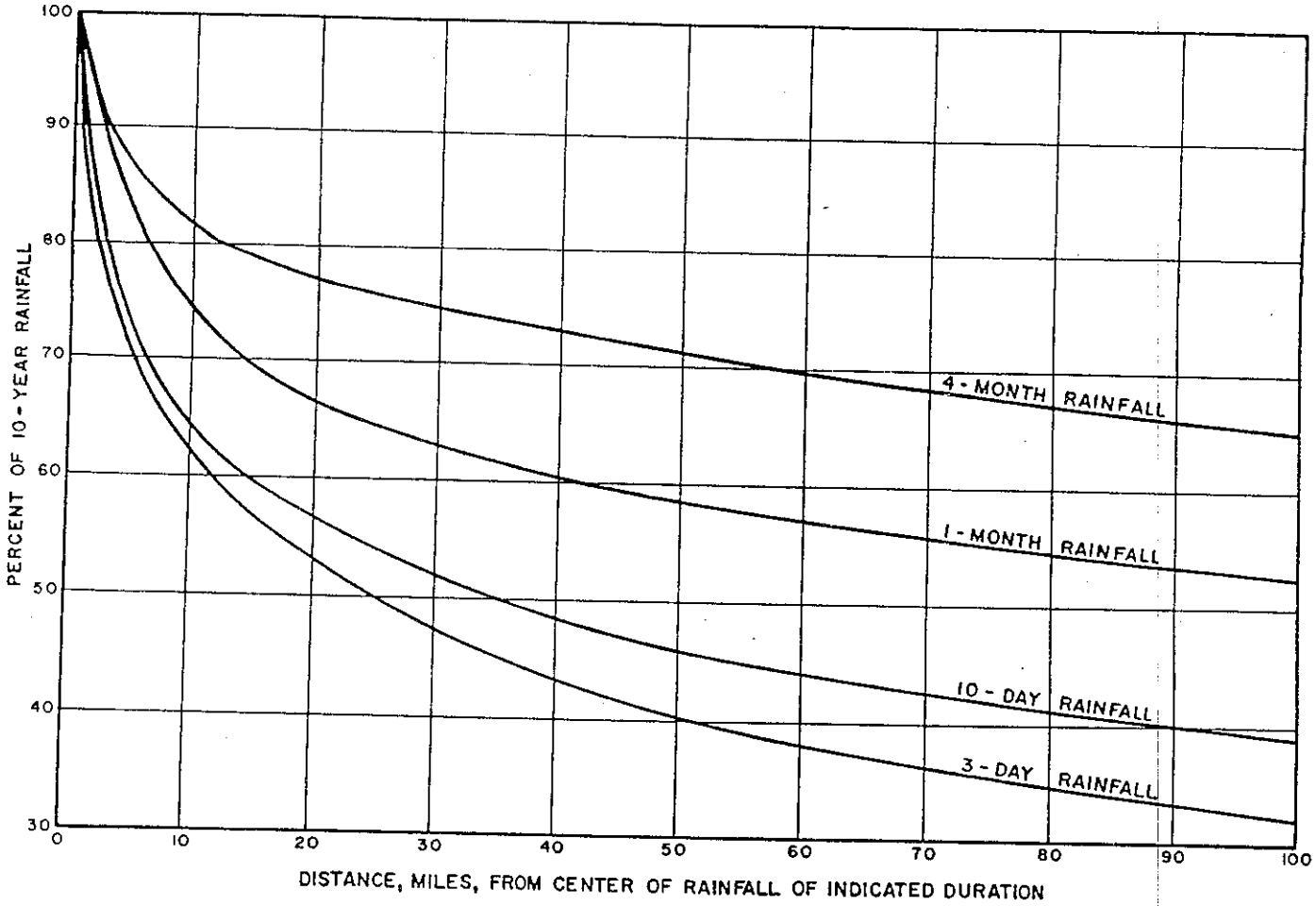
TABLE-2

RAIN FALL VARIABILITY

DISTANCE FROM STORM CENTRE (IN MILES)	5-DAY STORM PERCENTAGE OF POINT RAINFALL
1/4	100
1	88.0
2	81.7
3	77.5
4	74.2
5	72.0
6	69.8
7	67.8

Figure 4.

RAINFALL VARIABILITY



2.2 PERCENTAGE OF POINT RAINFALL

The centre of the storm is located at the geographical centre (centroid) of the basin. Concentric circles or isohytels are drawn at one mile interval around the centre starting one-half mile from the centre (Fig 5). Respective areas included in each isohytels are found out by means of planimeter and entered in Table-3. The basin centroid can be found by vertically suspending a card board cutout of the basin shape successively from two or more points and finding the intersection of plumb lines from each point.

TABLE-3

PERCENTAGE OF POINT RAINFALL

Average distance from storm centre	Area in sq.miles	Area times percentage
1/4 mile	0.79	0.79
1 mile	6.21	5.47
2 mile	7.83	6.40
3 mile	1.78	1.38
4 mile	0.12	0.09
	16.73	14.13

Percentage of point rainfall that forms the equivalent uniform depth of rainfall.

$$\frac{14.13}{16.73} \times 100 = 85\%$$

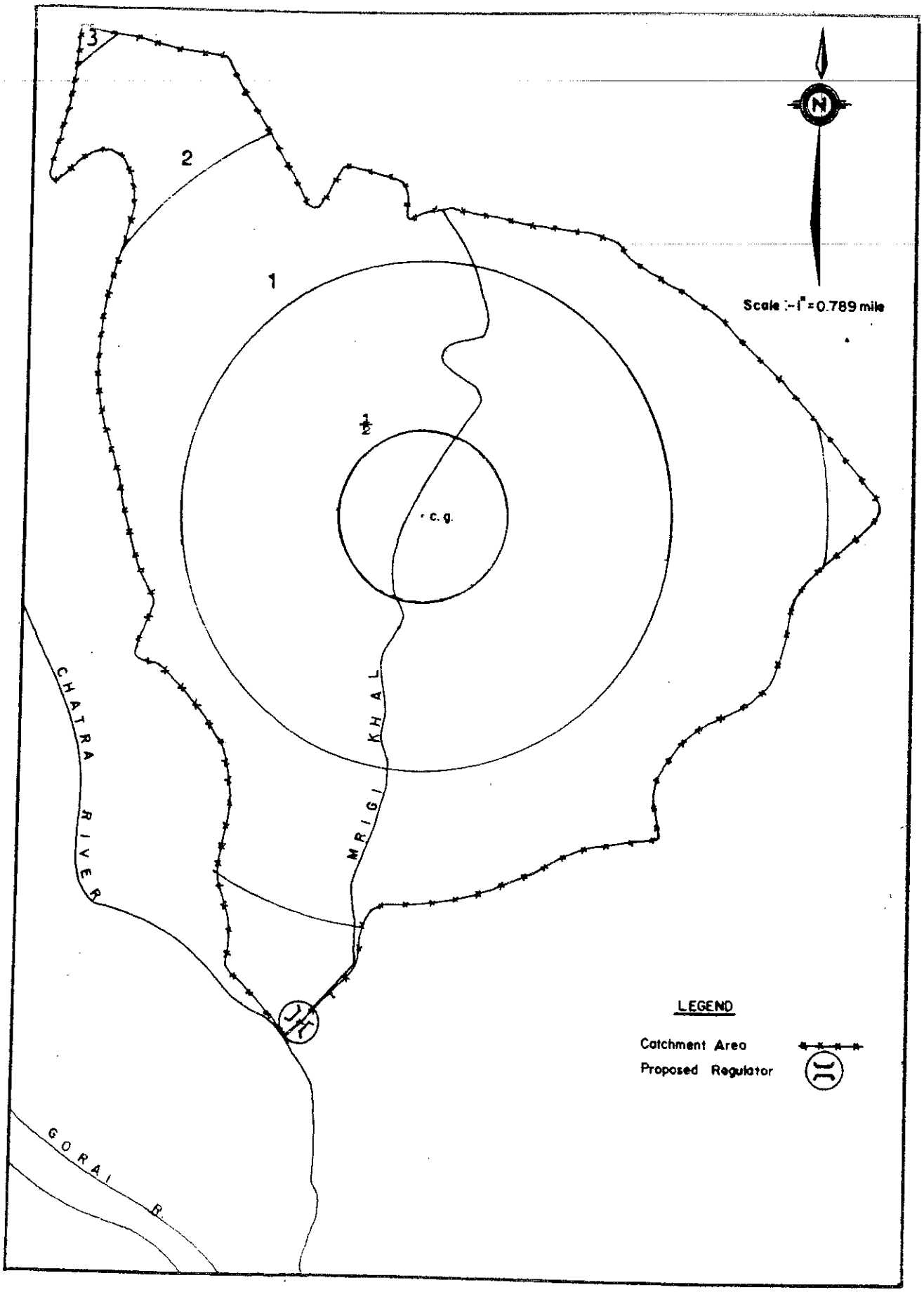

3
2
1
1/2
c.g.
Scale :- 1" = 0.789 mile
CHATRA RIVER
MRIGI KHAL
GORAI R.
LEGEND
Catchment Area
Proposed Regulator

Fig. 5

2.3 POINT RAINFALL

4 months rainfall index of project area = 50" probable 5-DAYS rainfall from a storm with a 10 year recurrence frequency of the project area is presented in Table 4.

TABLE-4

ACCUMULATIVE POINT RAINFALL

DAYS (1)	10 YEAR STORM FREQUENCIES (2)	ACCUMULATIVE POINT RAINFALL (3)=(2)X50"
1	0.128	6.4
2	0.192	9.6
3	0.230	11.5
4	0.257	12.9
5	0.276	13.8

2.4 EQUIVALENT UNIFORM DEPTH OF RAINFALL

Daily uniform depth equivalent are obtained by multiplying accumulative point rainfall of table-4 by 85%.

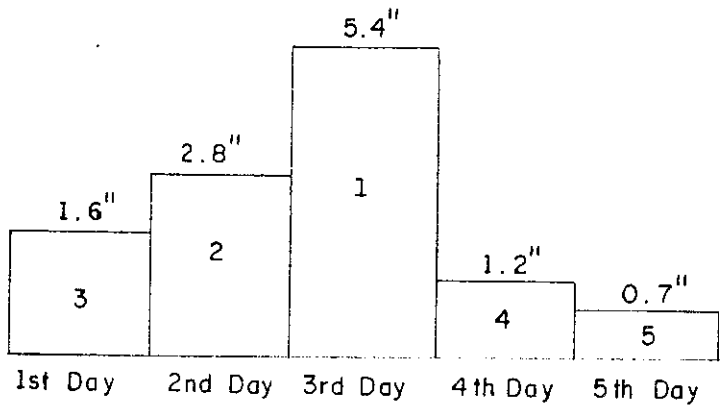
TABLE-5

EQUIVALENT UNIFORM DEPTH OF RAINFALL

DAYS	EQUIVALENT UNIFORM DEPTH		
	ACCUMULATIVE		DAILY INCREMENTS (IN INCH)
(1)	POINT RAINFALL (IN INCH) (2)	UNIFORM DEPTH (3)=(2)x0.85	(4)
1	6.4	5.4	5.4
2	9.6	8.2	2.8
3	11.5	9.8	1.6
4	12.9	11.0	1.2
5	13.8	11.7	0.7

2.5 DAILY RAINFALL SEQUENCE OF EQUIVALENT UNIFORM DEPTH

The daily increments of rainfall shown in Table 5 can occur in any order we will arbitrarily arrange a sequence of 3,2,1,4,5 in order to satisfy the losses that will take place early in the storm. A graphical arrangement of this sequence is presented in Fig. 6.



DAILY RAINFALL SEQUENCE UNIFORM EQUIVALENT DEPTH

FIGURE 6

2.6 RAINFALL TIME DISTRIBUTION

For further calculation we will wish to know the quantity of rain falling during certain hourly intervals. As an aid for breaking down rainfall time distribution into intervals less than 24-hours, Table-6 has been prepared from IECO Master Plan rainfall data. A reasonable 24-hour rainfall time distribution can be made for any basin by using the particular rainfall data closest in distance to the basin being studied and adjusting it to fit the calculated 24-hour basin rainfall. The smallest time

interval to be used will depend on the length of the main drainage course within the basin. The following intervals are recommended:

1. Less than 2 miles : one hour
2. 2 to 6 miles : 3 hours
3. Over 6 miles : 6 hours

TABLE-6

24 HR POINT RAINFALL TIME DISTRIBUTION

	4-Month Index	Storm Duration in Hours Accumulative rainfall in Inches						
		1	2	3	6	12	18	24
Barisal	65	2.33	2.77	3.11	3.82	5.36	6.85	8.35
Jessore	46	2.37	3.00	3.31	3.80	4.43	5.15	5.90
Cox's Bazar	115	2.33	2.88	3.31	4.40	6.65	10.50	14.10
Chittagong	86	2.43	3.07	3.58	4.56	6.67	9.00	11.00
Dacca	55	3.10	4.00	5.50	5.25	6.10	6.60	7.06
Bogra	58	3.40	4.23	4.70	5.47	6.30	6.90	7.40
Sylhet	120	2.85	4.15	5.00	6.16	8.20	12.00	15.30

The length of natural drainage channel = 9 miles > 6 miles. Therefore the 24-hour time distribution is to be made 6 hourly. The rainfall data located closest to project area is Jessore district, so an adjustment must be made. The adjustment will be made on the basis of the ratio of the respective 4 month rainfall index, that is, 50/46. The following table shows the adjustment procedure.

TABLE-7

MRIGI BASIN 24 HR RAINFALL TIME DISTRIBUTION

	Hours			
	6	12	18	24
1. Jessore Maximum Accumulative point rainfall	3.80	4.43	5.15	5.9
2. Project Area Max^{m} Accumulative point rainfall (1x50/46)	4.13	4.82	5.60	6.41
3. Project Area Max^{m} Incremental point rainfall	4.13	0.69	0.78	0.81
4. Project Area uniform Equivalent depth (3x0.85)	3.47 (1)	0.58 (2)	0.66 (3)	0.69 = 5.4" (4)

As it has been pointed out for rainfall time distribution with 24 hours increments, the increments within a 24 hour period also can not be predicted. Therefore a sequence of 3,4,1,2 is arranged arbitrarily. Thus during the maximum 24 hour rainfall period the 6 hour incremental rainfall is 0.66", 0.69", 3.47" and 0.58" which is the sequence for the 3rd day. The sequence for the other days are established in the following way.

1) Moving to the 2nd day use the same increments for 1st, 2nd and 4th periods as were used for the 3rd day.

2) For the 3rd period of the 2nd day we will assign the remainder of the total 2nd day rianfall or 0.87".

3) The ist day is similarly handled with the remainder of 0.00" in the 3rd period.

4) The 4th and 5th days are similarly handled and the design storm is complete.

TABLE-8

MRIGI BASIN DESIGN STORM

Day	6 Hour Increments
1	0.66
	0.69
	0.00
	0.25
2	0.66
	0.69
	0.87
	0.58
3	0.66
	0.69
	3.47
	0.58
4	0.66
	0
	0
	0.54
5	0.66
	0
	0
	0.04

3.0 DETERMINATION OF PERCENTAGE OF PADDY AND NON-PADDY LAND

Determine land classification as percentage of area in paddy land and non-paddy land. Whenever possible these percentages should be determined by field inspection. This is particularly important for small basins where the proportion of paddy land is disproportionately high. In absence of a survey the percentages on Table 9 can be used as an average.

TABLE-9

AGRICULTURAL LAND CLASSIFICATION

District	Percentage of Total Land Area	
	Available for Cultivation	Cultivated as paddy land
Dhaka	70	33
Mymensingh	75	40
Faridpur	84	50
Bakerganj	69	57
Chittagong	42	39
Noakhali	66	41
Comilla	79	61
Sylhet	56	35
Rajshahi	74	45
Dinajpur	69	46
Rangpur	66	42
Bogra	84	56
Pabna	81	37
Kushtia	75	26
Jessore	73	35
Khulna	36	25
Average	67	42

4.0 RAINFALL LOSSES

4.1 Soil moisture initial loss:

The U.S. soil conservation service has recommended an initial abstraction of Ia = 0.25 be made to cover initial interception, infiltration surface storage losses for this group. 'S' is the difference between storm rainfall and direct runoff. Utilizing the SCS rainfall runoff curves for land use classification and soil groups similar to those in Bangladesh. 'S' has been found to equal 2.5 and Ia = 0.5"

i) For paddy land no initial loss will be used.

ii) For non-paddy land an initial soil moisture loss of 0.5 inch will be considered.

4.2 Soil moisture subsequent loss

During the storm period infiltration continues to take place at a slower rate than the initial soil moisture repletion. Infiltration studies have shown that all but impervious clay soils have a maximum constant infiltration rate after saturation that may range from 0.05"/hour to greater than 1"/hr.

We will assume an average infiltration rate of 1"/day or 0.04"/hour for the relatively impervious soils or 0.25"/6 hour.

4.3 Depression storage

4.3.1 Paddy land

Paddy land is bounded by small earthen bund to retain water. This may be as high as 6" or more. We will assume that the first 4" of rain falling upon paddy land serves to fill up the paddy basin. Rainfall in excess of 4" becomes runoff.

4.3.2 Non paddy land

We will assume that the constant rate of retention is approximately 0.033"/hour or 0.2" per 6 hour during periods of rainfall until a maximum of 1.00" is stored.

5.0 RAINFALL EXCESS - WEIGHTED BASIN AVERAGE

5.1 Paddy Land

Separate the rainfall losses from the rainfall time distribtuion determined in Table-8 by assuming the entire basin area consists of paddy land. Computations are shown in Table 10.

5.2 Non-paddy Land

Separate the rainfall losses from the rainfall time distribution determined in Table-8 by assuming the entire basin area consists of non-paddy land.

Computations ar shown in Table 11.

5.3 Weighted Basin Average

Compute the weighted basin runoff distribution by multiplying the net runoff determined in Table 10 and Table 11 by the respective land classification percentage. Computations are shown in Table 12.

TABLE-10

RAINFALL EXCESS PADDY LAND

Days	Hours	Rainfall inches	Losses in inches: Soil Moisture: Initial	Losses in inches: Soil Moisture: Subsequent	Losses in inches: Depression storage	Available paddy storage in inches	Net Run off in inches
1	0-6	0.66	0	-0.25	-4.00	-3.59	0
	6-12	0.69		-0.25		-3.15	0
	12-18	0.00		-0.25		-3.40	0
	18-24	0.25		-0.25		-3.40	0
2	0-6	0.66		-0.25		-2.99	0
	6-12	0.69		-0.25		-2.55	0
	12-18	0.87		-0.25		-1.93	0
	18-24	0.58		-0.25		-1.60	0
3	0-6	0.66		-0.25		-1.19	0
	6-12	0.69		-0.25		-0.75	0
	12-18	3.47		-0.25		0	2.47
	18-24	0.58		-0.25		0	0.33
4	0-6	0.66		-0.25		0	0.41
	6-12	0.00		-0.25		-0.25	0
	12-18	0.00		-0.25		-0.50	0
	18-24	0.54		-0.25		-0.21	0
5	0-6	0.66		-0.25		0	0.20
	6-12	0.00		-0.25		-0.25	0
	12-18	0.00		-0.25		-0.50	0
	18-24	0.04		-0.25		-0.71	0

TABLE-11

RAINFALL EXCESS - NON PADDY LAND

Days	Hours	Rainfall inches	Losses in inches: Soil Moisture: Initial	Losses in inches: Soil Moisture: Subsequent	Losses in inches: Depression storage	Available non-paddy storage in inches	Net Run off in inches
1	0-6	0.66	-0.50	-	-0.16	0	0
	6-12	0.69		-0.25	-0.20	0	0.24
	12-18	0.00		-0.25	0	-0.25	0
	18-24	0.25		-0.25	0	-0.25	0
2	0-6	0.66		-0.25	-0.16	0	0
	6-12	0.69		-0.25	-0.20	0	0.24
	12-18	0.87		-0.25	-0.20	0	0.42
	18-24	0.58		-0.25	-0.08	0	0.25
3	0-6	0.66		-0.25	0	0	0.41
	6-12	0.69		-0.25	0	0	0.44
	12-18	3.47		-0.25	0	0	3.22
	18-24	0.58		-0.25	0	0	0.33
4	0-6	0.66		-0.25	0	0	0.41
	6-12	0.00		-0.25	0	-0.25	0
	12-18	0.00		-0.25	0	-0.50	0
	18-24	0.54		-0.25	0	-0.21	0
5	0-6	0.66		-0.25	0	0	0.20
	6-12	0.00		-0.25	0	-0.25	0
	12-18	0.00		-0.25	0	-0.50	0
	18-24	0.04		-0.25	0	-0.71	0

TABLE-12

RAINFALL EXCESS - WEIGHTED BASIN AVERAGE

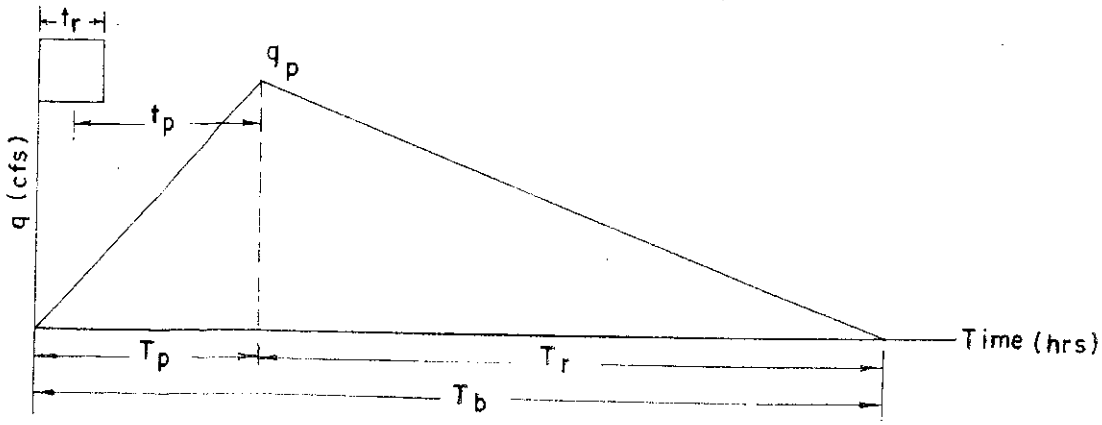
Days	Hours	Paddy land (50%)		Non paddy land (50%)		Basin weighted runoff
		Net runoff	Weighted runoff	Net runoff	Weighted runoff	
1	0-6	0	0	0	0	0
	6-12	0	0	0.24	0.12	0.12
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
2	0-6	0	0	0	0	0
	6-12	0	0	0.24	0.12	0.12
	12-18	0	0	0.42	0.21	0.21
	18-24	0	0	0.25	0.13	0.13
3	0-6	0	0	0.41	0.21	0.21
	6-12	0	0	0.44	0.22	0.22
	12-18	2.47	1.24	3.22	1.61	2.85
	18-24	0.33	0.17	0.33	0.17	0.34
4	0-6	0.41	0.21	0.41	0.21	0.42
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
5	0-6	0.20	0.10	0.20	0.10	0.20
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
						4.82

6.0 UNIT HYDROGRAPH

The unit hydrograph or unit-graph is a graphical presentation of discharge with respect to time at the drainage basin outlet. The unit hydrograph may be defined as the hydrograph of a unit volume of storm runoff produced by a uniform intensity storm of unit duration. The unit volume corresponds to one inch of rainfall excess spread uniformly over the total basin area. The unit duration is the same time interval we selected for our rainfall time duration Using our Mrigi Basin example, the unit-graph is formed from one inch of rainfall excess uniformly over the entire basin area during a unit time interval of 6 hours.

6.1 THE UNIT HYDROGRAPH SHAPE

Most of the basins we will encounter will be ungauged with scanty hydrological data available. Therefore, rather than using the more complicated and classical curvilinear hydrograph shape we will use the simpler triangular approximation. The shape of the hydrograph and its nomenclature is as follows:



q_p = Peak rate of runoff in cfs.

T_p = time of rise from beginning of rainfall excess to peak rate in hours.

T_r = Time of recession from peak rate to end of hydrography in hours.

T_b = time base of hydrograph in hours.

t_r = duration of rainfall excess in hours.

t_p = Lag time in hours from centre of rainfall excess to peak.

6.2 DEVELOPING THE UNIT HYDROGRAPH

The following data describing the basin characteristics have been obtained from topographic maps.

1) Basin area, A = 16.73 sq. mile = 10707.20 Acres

2) Length of the principle drainage channel : L = 9 miles

3) Distance to basin centroid: L = 3.6 miles

4) Average overland slope of basin measured between contours = 1.25 ft/mile.

5) Straight line length of basin = 5.2 miles

6) Average channel slope : $S = \frac{1.25 \times 5.2}{9} = 0.722$ ft/mile

7) Channel Roughness Factor : n = 0.055

6.3 The Elements of the unit Hydrograph

1. Time of concentration $T_c = 31 \left(\frac{L^2 n^2}{S}\right)^{0.3}$

$$= 31 \left(\frac{9^2 \times 0.055^2}{0.722}\right)^{0.3} = 22.5 \text{ hrs.}$$

2. Lag time to peak : $t_p = 0.6\ T_c = 0.6 \times 22.5 = 13.5$ hrs.

3. Time of rise : $T_p = t_p + 1/2\ t_r = 13.5 + 1/2 \times 6 = 17$ hrs.

4. Coefficient Ct :

$$C_t = \frac{t_f}{(L \times L)^{-0.3}} = \frac{13.5}{(9 \times 3.6)^{0.3}} = 4.76$$

5. Coefficient Cp = 0.43, obtained from Fig.7

6. Unit-graph Base Length:

$$T_b = \frac{1.2\ Tc}{Cp} = \frac{1.2 \times 22.5}{0.43}$$

$$= 63 \text{ hours}$$

7. Peak Discharge :

$$q_p = \frac{2A}{T_b} = \frac{2 \times 10707.2}{63}$$

$$= 340 \text{ cfs.}$$

The completed hydrograph is shown in Fig.8

7.0 THE RUNOFF HYDROGRAPH: (UNIT HYDROGRAPH METHOD)

The runoff hydrograph represents the time discharge relationship of the total design storm runoff at the basin outlet. The computations are shown in Table 13 while Fig.9 represents the shape of the runoff hydrograph.

TABLE-13

RUNOFF HYDROGRAPH COMPUTATIONS (UNIT HYDROGRAPH METHOD)

Unit Hydrograph Ordinates	Hours	Days	Rainfall Excess - 6 hours duration										Runoff Hydrograph
			0.12"	0.12"	0.21"	0.13"	0.21"	0.22"	2.85"	0.34"	0.42"	0.2"	
0	6		0										0
98	12	1	12	0									12
218	18		26	12	0								38
340	24		41	26	21	0							88
295	6		35	41	46	13	0						135
250	12	2	30	35	71	28	21	0					185
203	18		24	30	62	44	46	22	0				228
160	24		19	24	53	38	71	48	279	0			532
115	6		14	19	43	33	62	75	621	33	0		900
70	12		8	14	33	26	53	65	969	74	41	0	1283
28	18	3	3	8	24	21	43	55	841	116	92	20	1223
0	24		0	3	15	15	33	45	713	100	143	40	1111
	6			0	6	9	24	35	579	85	124	68	930
	12	4			0	4	15	25	456	69	105	59	733
	18					0	6	15	328	54	85	50	538
	24						0	6	200	39	67	41	353
	6							0	80	24	48	32	184
	12	5							0	10	29	23	62
	18									0	12	14	26
	24										0	6	6
	6	6										0	0
													8567

A check on the computations:

1. Total volume of water under the hydrograph : $V = \frac{It}{12}$, I = from last col. t = 6-hour time interval

$$V = \frac{8567 \times 6}{12} = 4283.5 \text{ acre feet}$$

2. Total rainfall excess, $i = \frac{12V}{A} = \frac{12 \times 4283.5}{10707.2} = 4.8"$, This should equal the summation of weighted rainfall excess (Table 12)

Figure 7.

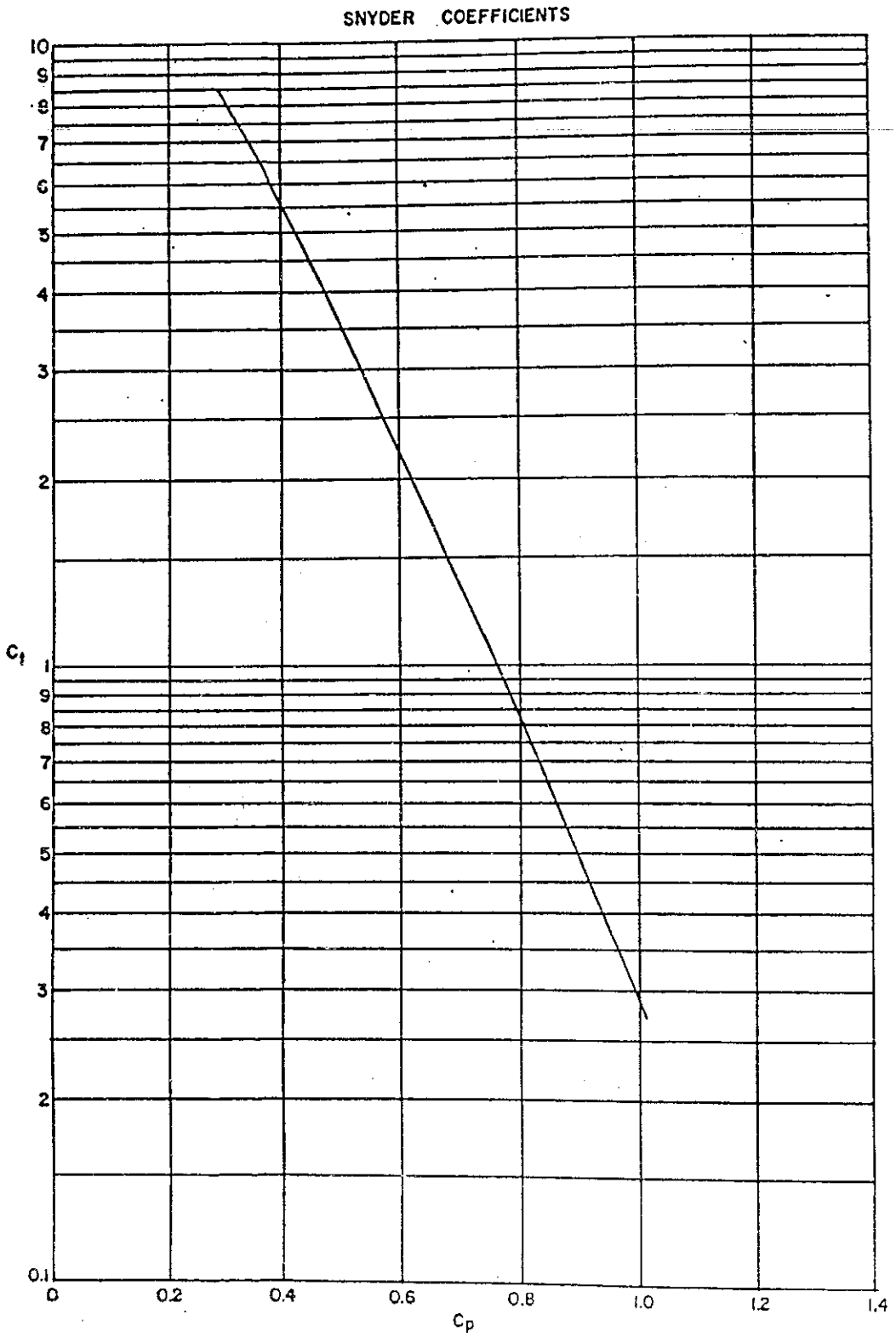

SNYDER COEFFICIENTS
10
9
8
7
6
5
4
3
2
C_t
1
9
8
7
6
5
4
3
2
0.1
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
C_p

Figure 8.

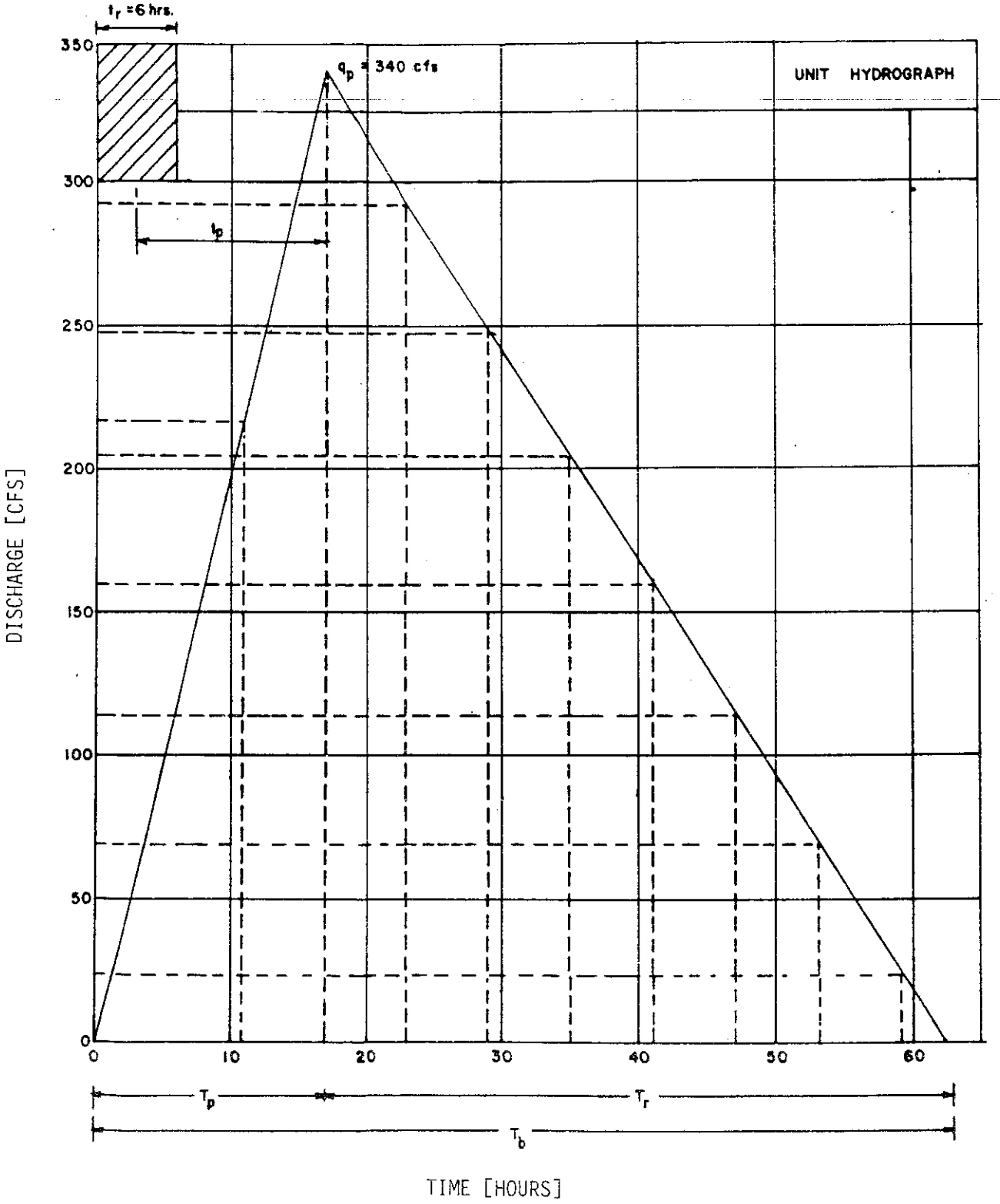
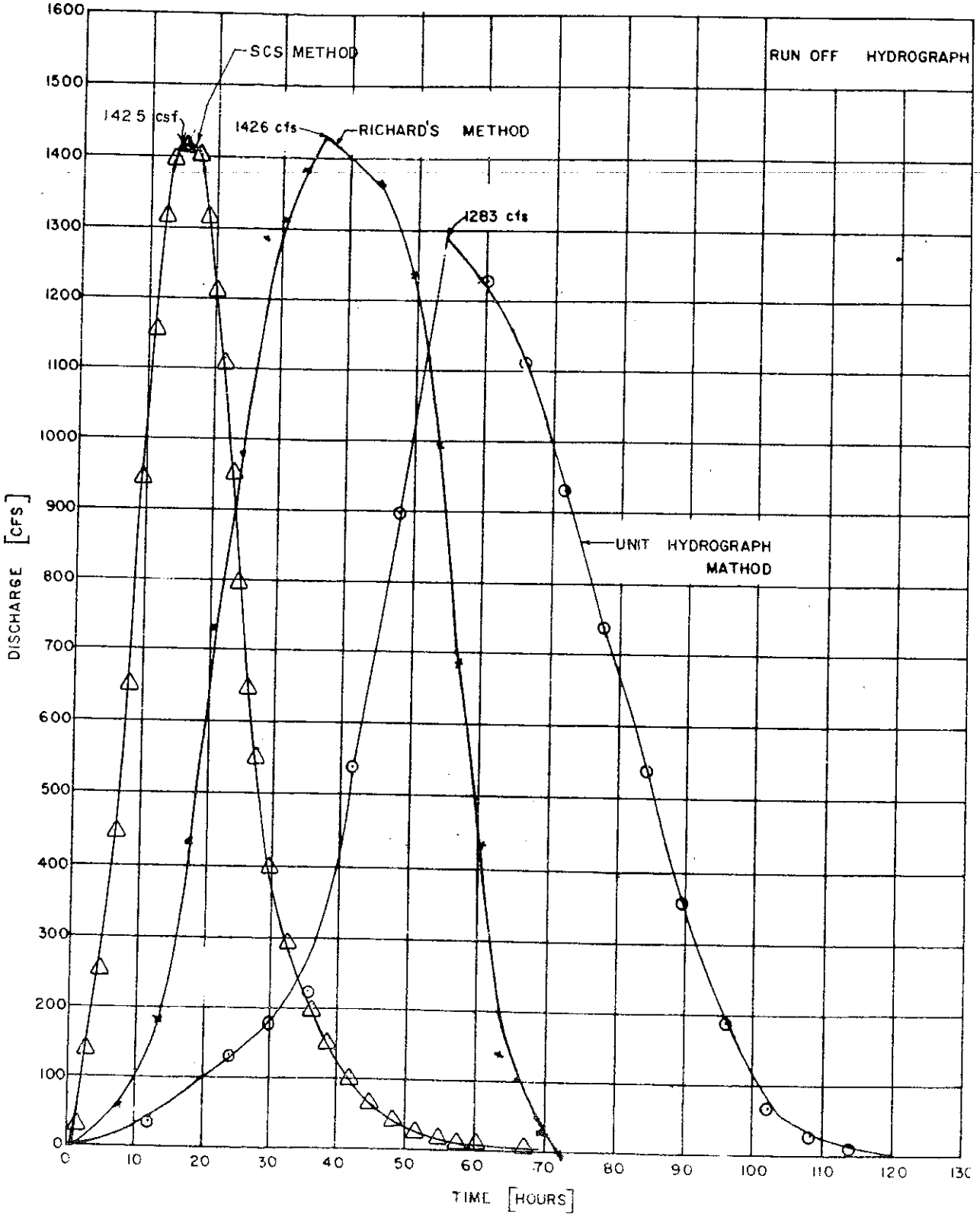

t_r = 6 hrs.
q_p = 340 cfs
UNIT HYDROGRAPH
t_p
350
300
250
200
150
100
50
0
DISCHARGE [CFS]
0
10
20
30
40
50
60
T_p
T_r
T_b
TIME [HOURS]

Figure 9.



পলি দাশ
উপ-বিভাগীয় প্রকৌশলী
নকশা সার্কেল-২
বাপাউবো, ঢাকা-১২১৫

8.0 THE RUNOFF HYDROGRAPH RICHARD'S METHOD

8.1 RICHARD'S EQUATIONS

The assumptions made by Richards are that the storm covered the catchment and that its duration was equal to the period of concentration of the flood. These assumptions implied that the whole catchment contributed to produce the maximum intensity of flood.

On this basis, the following principal equations were deduced by Richards:

(i) $i = \frac{R.f(a)}{t + 1}$ or $R = \frac{i(t+1)}{f(a)}$

(ii) $\frac{t3}{t+1} = \frac{N.C.L^2}{K.S.R.f(a)}$

(iii) $Q = k\ i\ a$

where, t denotes the period of concentration in hrs.

Q	"	maximum intensity of flood in cusec
N	"	storm shape factor with range of value from 0.72 to 1.72 (usually 1.1)
i	"	the average intensity of rainfall over the catchment in inches per hour
a	"	the area of the catchment in acres
L	"	the distance in miles from the point of concentration to the furthest point on the catchment
s	"	the slope of the catchment
k	"	the run-off co-efficient

f(a) denotes a function of the area for the adjustment of the average intensity of the rainfall.

C " a co-efficient

R " a co-efficient of rainfall

The intermediate points on the curve of rising flood are given by $Q_1 = Qm. \frac{a_1}{a}$ and $t_1 = t \sqrt[3]{\left(\frac{r}{L}\right)^2}$

and the falling flood curve is the reverse of the rising flood curve. r is radius from point of concentration along L for the area a, bounded by catchment boundary and arc with radius r.

8.2 DESIGN COMPUTATIONS & PROCEDURE

a) Use 24-hour point rainfall with required frequency and determine i.

b) Use topomap and determine a, L & S

c) Determine f(a) from Fig.10 for the area of the catchment

d) Compute R using Equation (i)

e) Select K for the catchment using Table 14 and compute K.R.

f) Determine C from Fig.11 using the computed value of K.R.

g) Find t by using Equation (ii)

h) Use t in nearest days and take point rainfall for this duration.

i) Repeat necessary steps to obtain t at the end of the operation as same in the beginning of the operation

j) Compute i for this determined t

k) Use Equation (iii) and compute Q.

l) Draw arcs with 'O' as centre from the point of concentration along L with equal intervals say 1,2,2,4,5 .....miles

m) Compute area of the catchment intercepted by each arc

n) Compute intermediate points on the hydrograph by using equations for intermediate points.

TABLE-14

VALUES OF RUNOFF CO-EFFICIENT, K

Type of catchment	Large	Small & Steep
Rocky and impermeable	0.8	1.0
Slightly permeable, bare	0.6	0.8
Slightly permeable, partly cultivated or covered with vegetation	0.4	0.6
Cultivated absorbent soil	0.3	0.4
Sandy absorbent soil	0.2	0.3
Heavy forest	0.1	0.2

Example:

a = 16.73 sq. miles = 10707.2 acres

L = 9 miles

S = 0.722 ft/mile = 0.0001367

24 hour rainfall = 6.4 inch (from Table-4)

$$i = \frac{6.4}{24} = 0.267 \text{ in/hour}$$

f(a) = 0.825 from Fig 10 for a = 10707.2 acres

$$R = \frac{i(t + 1)}{f(a)} = \frac{0.267 \times 25}{0.825} = 8.09 \text{ inches}$$

K = 0.6 selected from Table 14

KR = 0.6x8.09 = 4.854

C = 0.009 from Fig 11 for KR = 4.854

$$\frac{t^3}{t+1} = \frac{N.C.L^2}{K.S.R.\ f(a)} = \frac{1.1 \times 0.009^2 \times 9}{0.6 \times 0.0001367 \times 8.09 \times 0.825} = 1465$$

By trial, t = 39 hours ≈ 2 days.

2 - day rainfall = 9.6"

$$i = \frac{9.6}{48} = 0.2 \text{ inch/hour}$$

$$R = \frac{0.2 \times 49}{0.825} = 11.88$$

KR = 0.6 x 11.88 = 7.128

C = 0.0075

$$\frac{t^3}{t+1} = \frac{1.1 \times 0.0075 \times 9^2}{0.6 \times 0.0001367 \times 11.88 \times 0.825} = 831.28$$

t = 30 hours ≈ 1 1/2 day

1 1/2 day rainfall = 8" [From Fig. 1,2, & 3]

$$i = \frac{8}{36} = 0.222 \text{ inch/hour}$$

$$R = \frac{0.222 \times 37}{0.825} = 9.956$$

KR = 0.6 x 9.956 = 5.97

C = 0.0085

$$\frac{t^3}{t+1} = \frac{1.1 \times 0.0085 \times 9^2}{0.6 \times 0.0001367 \times 9.956 \times 0.825} = 1124$$

t = 34 hours ≈ 36 hours

i = 0.222 inch/hours

Q = Kia = 0.6 x 0.222 x 10707.2 = 1426 cfs.

To compute intermediate points arcs are drawn with 0 as centre from the point of concentration along L with radii 1,2,3,4,5,6,7 & 8 (Fig 12) and the area of the catchment intercepted by each are computed. The intermediate points on the curve of raising flood are given by

$$Q_1 = Q_m \frac{a_1}{a} \quad \& \quad t_1 = t\sqrt[3]{\left(\frac{r}{L}\right)^2}$$

and the falling curve is the reverse of the raising curve. The computations are shown in Table 15 whereas the runoff hydrograph is presented in Fig.9.

TABLE-15

RUNOFF HYDROGRAPH COMPUTATIONS (RICHARDS METHOD)

Distance from outlet in miles	$\frac{a_1}{a}$	$\sqrt[3]{\left[\frac{r}{L}\right]^2}$	Rising Flood		Falling Flood	
			Q_R (in cfs)	t_R (in hour)	Q_F (in cfs)	t_F (in hour)
(1)	(2)	(3)	(4) (1285xcol.2)	(5) (48xcol.3)	(6) (1285-col.4)	(7) (48+col.5)
0	0	0	0	0	1426	36.00
1	0.043	0.2311	61	8.32	1365	44.32
2	0.132	0.3669	188	13.21	1238	49.21
3	0.302	0.4807	431	17.31	995	53.31
4	0.517	0.5824	737	20.97	689	56.97
5	0.693	0.6758	988	24.33	438	60.33
6	0.899	0.7631	1282	27.47	144	63.47
7	0.925	0.8457	1319	30.45	107	66.45
8	0.973	0.9245	1388	33.28	38	69.28
9	1.000	1.0000	1426	36.00	0	72.00

Figure 11.

Ratio of Rainfall Intensity f (a)

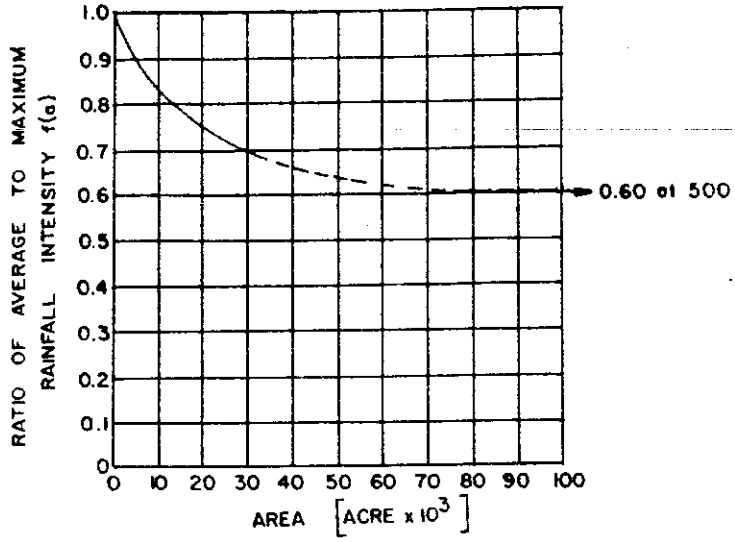


Fig. 10

Co-efficient, C

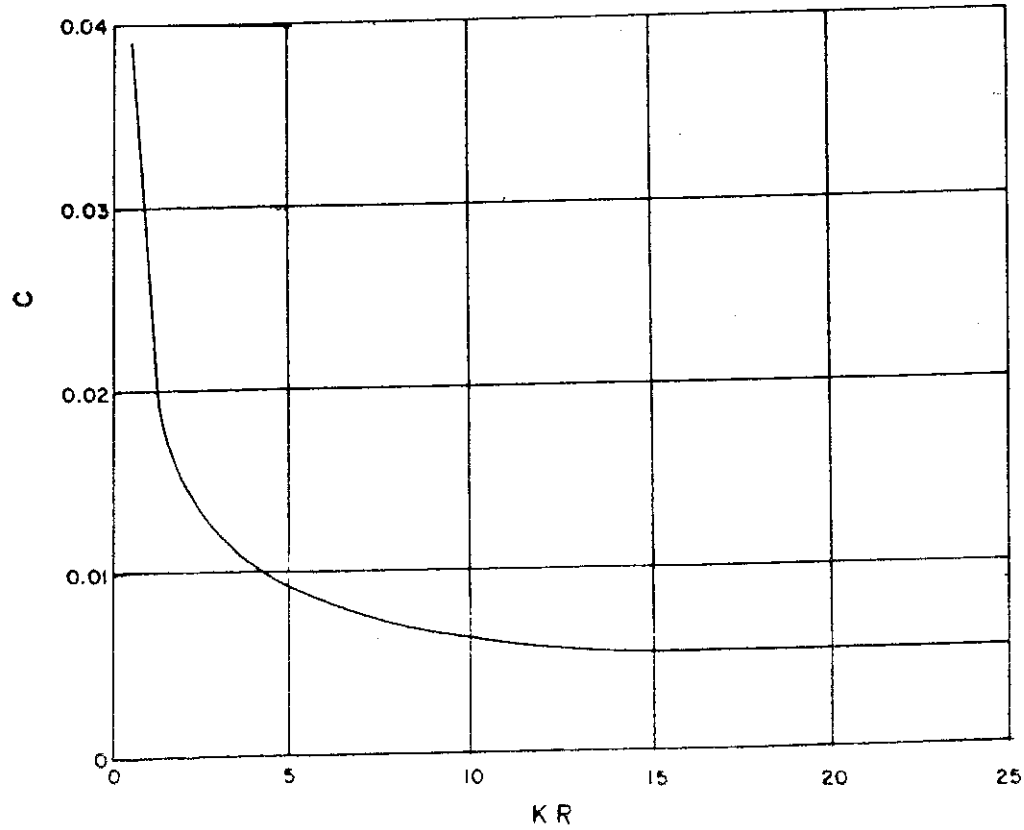
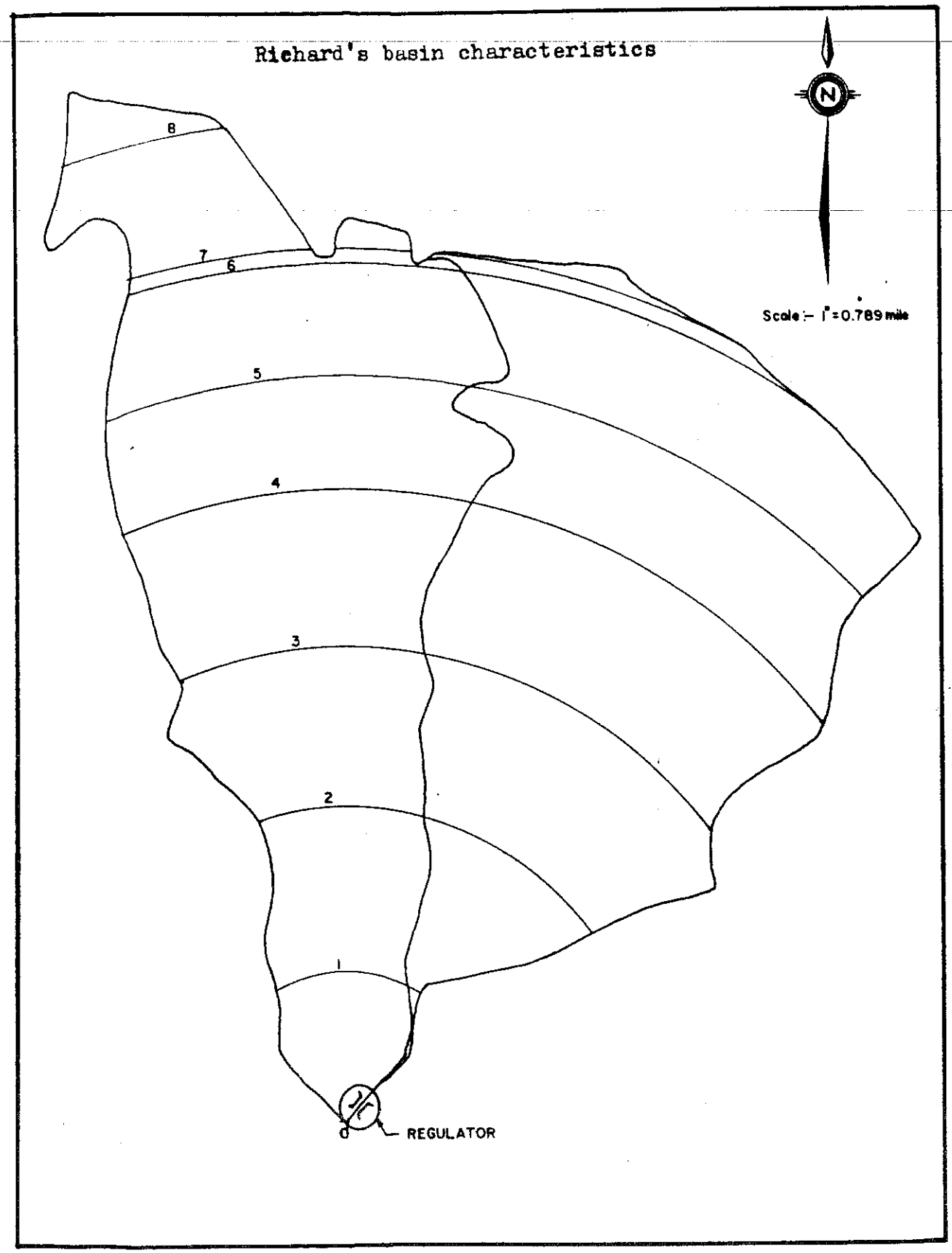


Figure 12.


Richard's basin characteristics
N
Scale :- 1" = 0.789 mile
8
7
6
5
4
3
2
1
0
REGULATOR

9.0 THE RUNOFF HYDROGRAPH (SCS METHOD)

9.1 DESIGN COMPUTATION AND PROCEDURE

a) Estimate Curve Number (CN) according to hydrologic soil group & soil-cover complexes using Table 17 & Table 18.

b) Estimate $Tc = 31 \left(\frac{L^2 n^2}{S}\right)^{0.3}$ (in hours)

where, L = Length of the principle drainage channel in miles

n = Channel Roughness Factor

S = Average channel slope in ft/mile.

c) Find Duration of Rainfall, $D = 0.133\ Tc$ (in hours)

d) Find Time to Rise, $Tp = \frac{D}{2} + 0.6\ Tc$ (in hours)

e) Find Direct Runoff, Q (Excess rainfall) using the following equation ;

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \text{(in inch)}$$

where, P = Potential maximum rainfall

$$S = \frac{1000}{CN} - 10$$

g) Find Peak Discharge

$$q_p = \frac{484\ AQ}{Tp} \quad \text{in cfs}$$

where, A = Basin area in sq. mile

Take T_p & Q from steps (d) & (e)

h) Estimate total actual hydrograph from

SCS dimensionless Unit hydrograph (q/qp Vs t/Tp)

Values are presented in Table 19.

where, q = Discharge at time t

q_p = peak discharge

t = A selected time

Tp = Time from begining of Rise to the Peak.

<u>Example (SCS Method)</u>

Location of Basin = Faridpur Dist.

Type of soil = clay

Land use = Cultivated Land

From Table 17, Hydrologic soil Group = D

From Table 18, Curve Number = 91

$$Tc = 31 \left(\frac{L^2 n^2}{S}\right)^{0.3} = 22.5 \text{ hrs}$$ [From page 51]

$$D = 0.133\ Tc = 0.133 \times 22.5$$

$$= 2.99 \text{ hrs.}$$

Say 3 hrs.

$$Tp = \frac{D}{2} + 0.6\ Tc$$

$$= \frac{3}{2} + 0.6 \times 22.5$$

$$= 15 \text{ hours}$$

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

From Table 6, P for 3 hrs = 3.31 for Jessore Dist.

For Faridpur Dist.

$$P = 3.31 \times \frac{50}{46} = 3.6"$$

$$S = \frac{1000}{CN} - 10$$

$$= \frac{1000}{91} - 10 = 0.99$$

$$Q = \frac{(3.6 - 0.2 \times 0.99)^2}{3.6 + 0.8 \times 0.99}$$

$$= 2.64"$$

$$q_p = \frac{484\ AQ}{Tp}$$

$$A = 16.73 \text{ sq. mile}$$

$$q_p = \frac{484 \times 16.73 \times 2.64}{15}$$

$$= 1425 \text{ cfs.}$$

TABLE-16

RUNOFF HYDROGRAPH COMPUTATION (SCS METHOD)

Time Ratio t/Tp	Time t (in hours)	Discharge Ratio q/qp	Discharge q (in cfs)
0	0	0.000	0.00
0.1	1.5	0.030	42.75
0.2	3.0	0.100	142.50
0.3	4.50	0.190	270.75
0.4	6.0	0.310	441.75
0.5	7.5	0.470	669.75
0.6	9.0	0.660	940.50
0.7	10.5	0.820	1168.50
0.8	12.0	0.930	1325.25
0.9	13.5	0.990	1410.75
1.0	15.0	1.000	1425.00
1.1	16.5	0.990	1410.75
1.2	18.0	0.930	1325.25
1.3	19.5	0.860	1225.50
1.4	21.0	0.780	1111.50
1.5	22.5	0.680	969.00
1.6	24.0	0.560	798.00
1.7	25.5	0.460	655.50
1.8	27.0	0.390	555.75
1.9	28.5	0.330	470.25
2.0	30.0	0.280	399.00
2.2	33.0	0.207	294.98
2.4	36.0	0.147	209.48
2.6	39.0	0.107	152.48
2.8	42.0	0.077	109.73
3.0	45.0	0.055	78.38
3.2	48.0	0.040	57.00
3.4	51.0	0.029	41.33
3.6	54.0	0.021	29.93
3.8	57.0	0.015	21.38
4.0	60.0	0.011	15.68
4.5	67.50	0.005	7.13
5.0	75.0	0.000	0

The runoff hydrograph is ploted with the values of t & q from Table 16 & presented in Fig. 9.

TABLE-17.

HYDROLOGIC SOIL GROUPS.

A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

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

TABLE-18.

RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USE (ANTECEDENT MOISTURE CONDITION II, AND $I_a = 0.2 S$) (FROM SCS TECHNICAL RELEASE NO. 55, P.2-5)

Land Use Description	Hydrological Soil Group A	B	C	D
Cultivable land 1/: Without conservation treatment	72	81	88	91
with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood of forest land: thin scand, poor cover, no mulch	45	66	77	83
good cover 2/	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% or 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential 3/				
Average lot size — Average % impervious 4/				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc. 5/	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers 5/	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

1/ For a more detailed description of agriculture land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972

2/ Good cover is protected from grazing and litter and brush cover soil.

3/ Curve number are computed assuming the runoff from the house & driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur

4/ The remaining pervious areas (Lawns) are considered to be in good pasture condition for these curve numbers.

5/ In some warmer climate of the country a curve number of 95 may be used.

TABLE-19.

RATIOS FOR DIMENSIONLESS UNIT HYDROGRAPH.

Time Ratios (t/Tp)	Discharge Ratios (q/qp)
0	.000
0.1	.030
0.2	.100
0.3	.190
0.4	.310
0.5	.470
0.6	.660
0.7	.820
0.8	.930
0.9	.990
1.0	1.000
1.1	.990
1.2	.930
1.3	.860
1.4	.780
1.5	.680
1.6	.560
1.7	.460
1.8	.390
1.9	.330
2.0	.280
2.2	.207
2.4	.147
2.6	.107
2.8	.077
3.0	.055
3.2	.040
3.4	.029
3.6	.021
3.8	.015
4.0	.011
4.5	.005
5.0	.000

10.0 PRE-MONSOON FLOOD ROUTING

Flood routing is basically a system of book keeping involving inflow, outflow and storage. During a definite time interval (assuming there are no losses other than those already extracted).

$$I_v - D_v = S$$

Where, I_v = total volume of inflow during the period.

D_v = total volume of outflow " " "

S = total volume going to storage " "

The eqn. written for a definite time period and expressing inflow and outflow in cfs is

$$\frac{S}{t} = I - D$$

t = time of period in sec.

I = Average inflow during the period in cfs.

D = Average outflow " " " "

S = total volume going to storage during the period measured in cft.

This equation forms the basis for a hydrological procedure in routing in which 't' is known the routing period.

10.1 FLOOD ROUTING EQUATION

$$(I - D)\ t = S$$

Let $s_2 - s_1 = S$ Storage differential during period

$$\frac{I_1 + I_2}{2} = I \quad \text{average inflow} \qquad " \qquad "$$

$$\frac{D_1 + D_2}{2} = D \quad \text{average outflow} \qquad " \qquad "$$

Substituting the terms.

$$\left(\frac{I_1 + I_2}{2} - \frac{D_1 + D_2}{2}\right) t = S_2 - S_1$$

$$I_1 + I_2 - D_1 - D_2 = \left(\frac{2S_2}{t}\right) - \frac{2S_1}{t}$$

$$(I_1 + I_2) + \left(\frac{2S_1}{t} - D_1\right) = \left(\frac{2S_2}{t} + D_2\right)$$

To solve the equation 4 curves are required

1. The inflow hydrograph - prepared already
2. Area - capacity curve
3. The stage discharge relation for condition 3 & 5.
4. The 2S/t + D curve.

11.0 AREA-CAPACITY CURVES

To draw these curves a contour map of the project area is collected and the area between each contour level is planimetered. The area curve is plotted as ground elevation Vs total accumulated area at any elevation. The capacity curve is the accumulation of volume below a particular elevation. Capacity between two contour intervals is calculated as the average of the area at the two levels multiplied by the contour interval. For Mrigi Khal Basin the calculations are shown in Table 20 while the area capacity curves are presented in Fig.13.

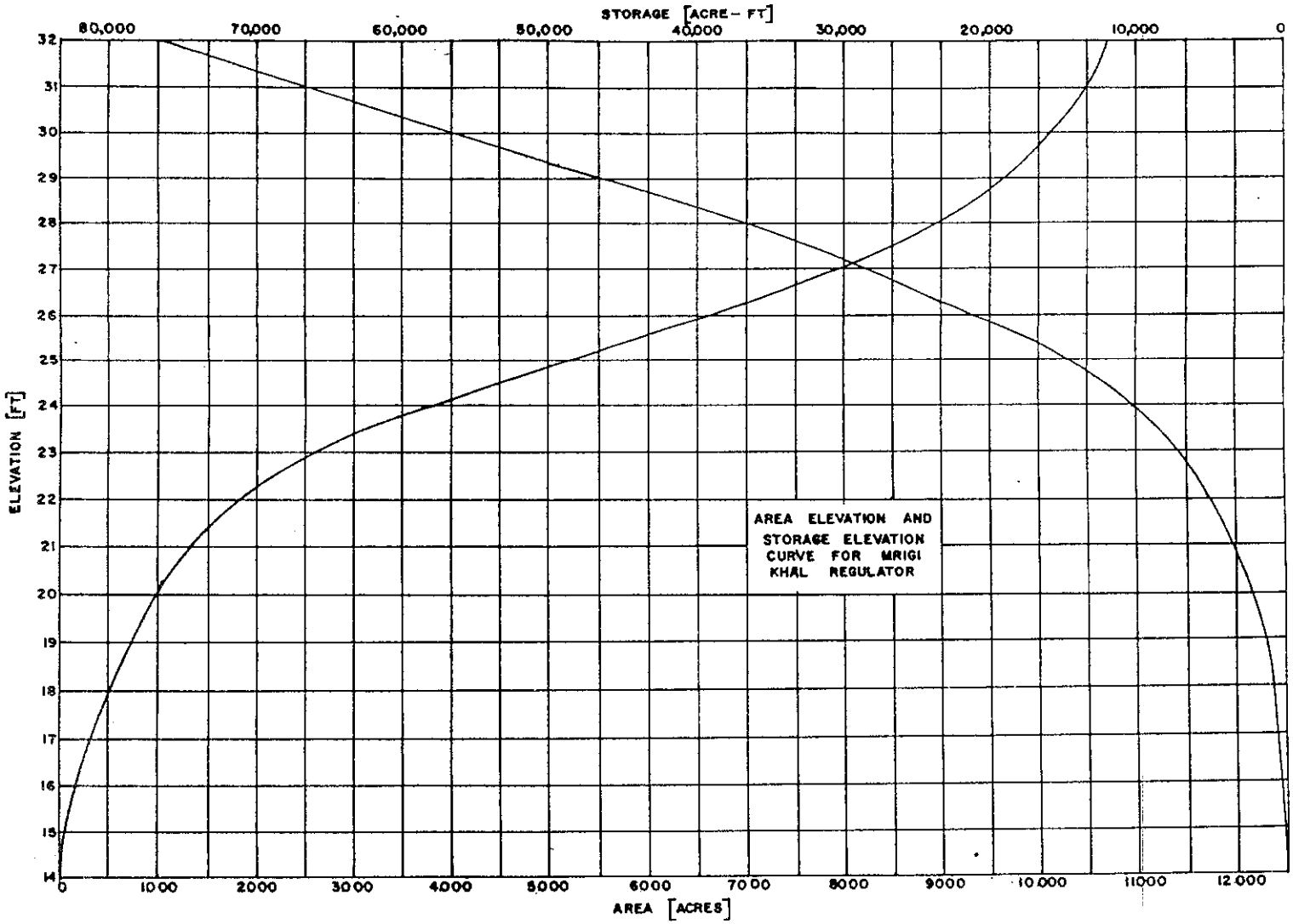
TABLE-20

AREA CAPACITY COMPUTATIONS

Ground Elevation	Area in Acres	Σ Area in Acres	Δ Volume in Acre Ft	Σ Volume in Acre Ft
14	0	0	0	0
15	50.56	50.56	25.28	25.28
16	101.12	151.68	101.12	126.40
17	151.68	303.36	227.52	353.92
18	202.24	505.60	404.48	758.40
19	252.80	758.40	632.00	1390.40
20	252.80	1011.20	884.80	2275.20
21	134.40	1145.60	1078.40	3353.60
22	510.40	1656.00	1400.80	4754.40
23	933.60	2589.60	2122.50	6876.90
24	1183.20	3772.80	3181.20	10058.10
25	1365.60	5138.40	4455.60	14513.70
26	1444.00	6582.40	5860.40	20374.10
27	1294.40	7876.80	7229.60	27603.70
28	1104.00	8980.80	8429.80	36033.50
29	537.60	9518.40	9249.60	45283.10
30	564.00	10082.40	9850.40	55133.50
31	483.20	10565.60	10374.00	65507.50
32	141.60	10707.20	10636.4	76143.90

= 16.73 sq. miles

Figure 13

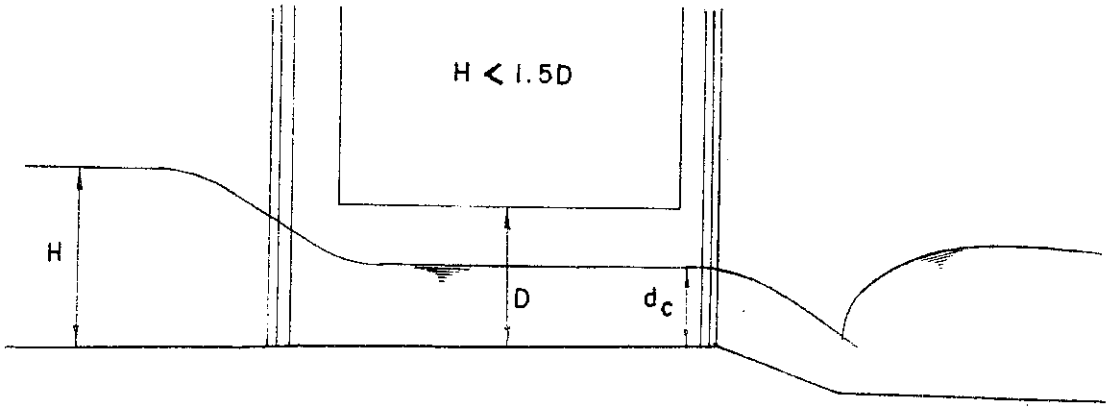

STORAGE [ACRE-FT]
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
ELEVATION [FT]
32
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10,000
11000
12,000
AREA [ACRES]
AREA ELEVATION AND STORAGE ELEVATION CURVE FOR MRIGI KHAL REGULATOR

12.0 STAGE DISCHARGE CURVE

For premonsoon routing condition i.e. when discharge is a function of storage level only the flow condition changes between condition 5 to 3 when the head water depth is 1.5 times the vent height.

12.1 Characteristic of condition 5

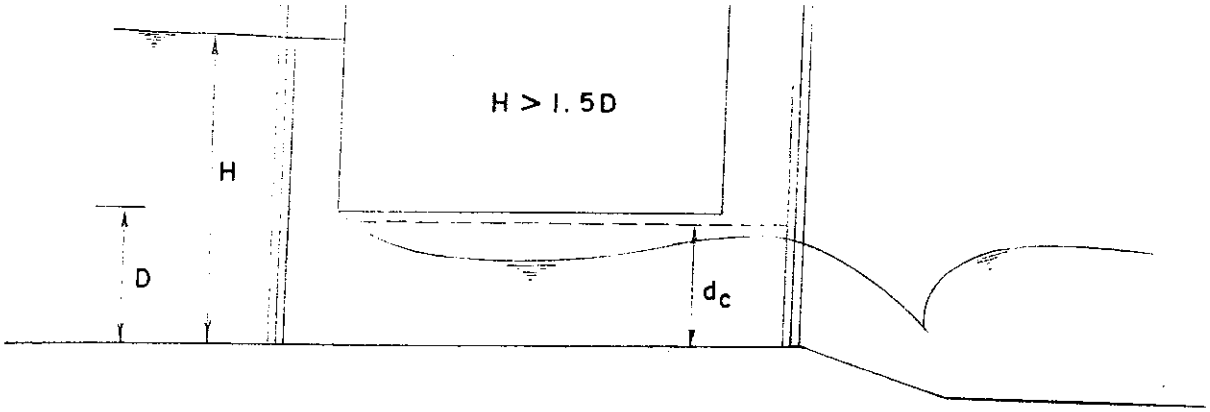
Inlet not submerged and low tail water. When the head water depth is less than 1.5 times the vent height the entrance is not sealed by water and the regulator acts as a broad crested weir. The tail-water depth is less than critical flow depth through the vent. This type will usually occur when a stilling pool or downstream drop, is part of the structure and the flow is made to pass through critical depth. A hydraulic jump will usually occur down stream. The control is at the out let and the discharge co-efficient varies from 2.46 to 3.47.



CONDITION 5

12.2 Characteristics of condition 3

Inlet submerged outlet not submerged vent hydraulically short. The vent is short and will not flow full. It is the usual condition occuring in regulators when the head water depth is greater than 1.5 times the vent height. The control is at entrance and is similar to orifice discharge with the discharge co-efficient varying from 0.45 to 0.75.



CONDITION 3

Invert level of the structure is fixed at RL + 14.00. The discharge under condition 5 & 3 are computed for different elevation and entired in Table 21 while the stage discharge curve is presented in Fig. 14.

12.3 **Discharge formula for conditin 5**

$$q = C_2 H^{3/2} \text{ per foot-width}$$

$$C_2 = 2.68$$

For 3 vents (5'x6')

$$Q = 15 \times 2.68 H^{3/2}$$

$$= 40.2H^{3/2}$$

12.4 **Discharge formula for condition 3**

$$q = CqD \sqrt{2gH} \text{ per foot width}$$

$$= 48.3 \text{ Cq } H^{1/2}$$

For 3 vents (5'x6')

$$Q = 3x5x48.3 \text{ Cq } H^{1/2}$$

$$= 724.5 \text{ Cq } H^{1/2}$$

Cq is obtained from Fig.15 for the corresponding value of D/H.

TABLE-21

STAGE DISCHARGE TABLE (3 VENTS 5'X6')

R. L.	H in ft.	Flow condition	Q in cfs
14	0	-	0
15	1	5	40
16	2	5	114
17	3	5	209
18	4	5	322
19	5	5	450
20	6	5	591
21	7	5	745
22	8	5	910
23	9	5	1086
24	10	3	1134
25	11	3	1194
26	12	3	1267
27	13	3	1322
28	14	3	1383
29	15	3	1440
30	16	3	1495

Figure 14

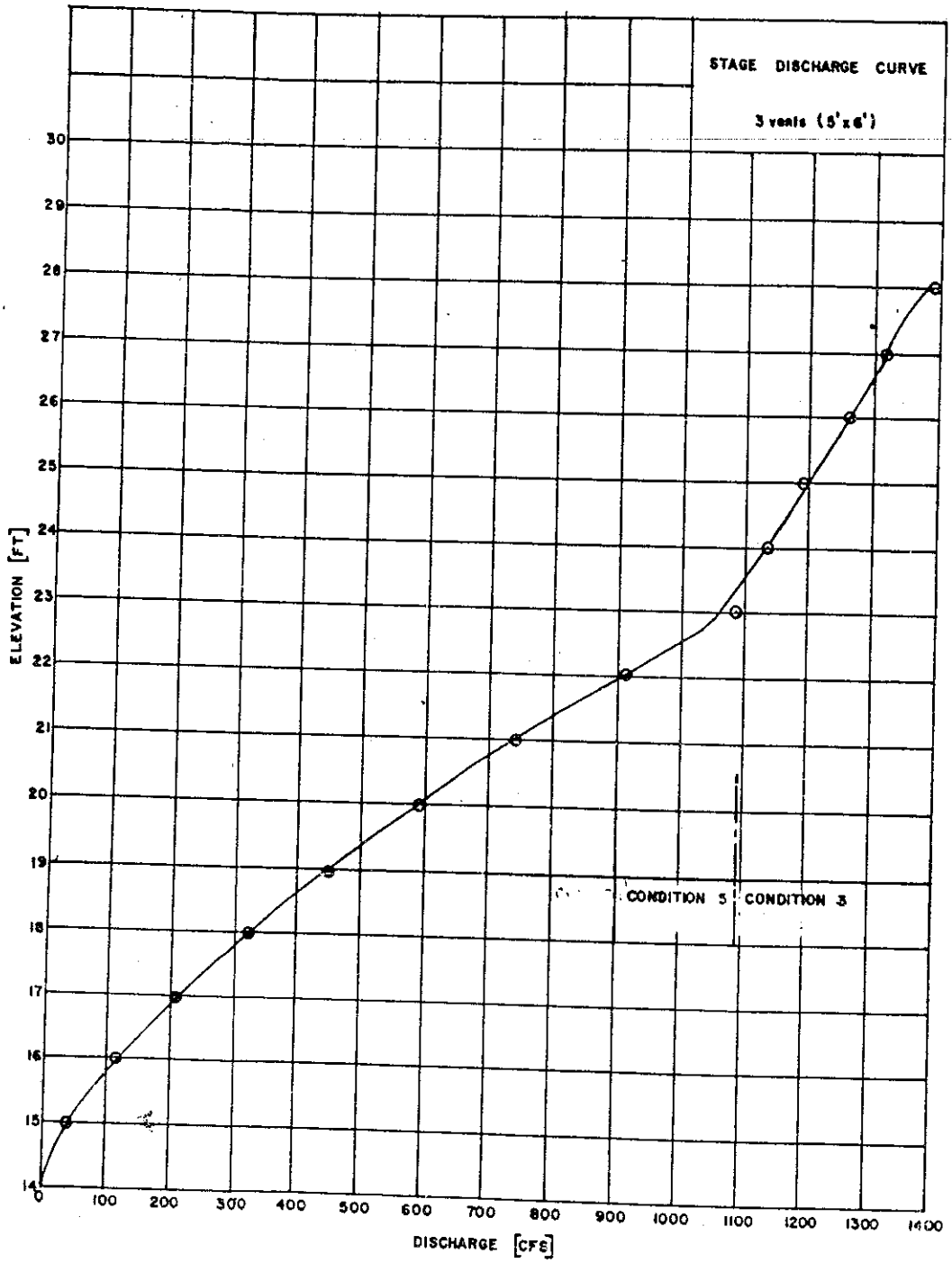
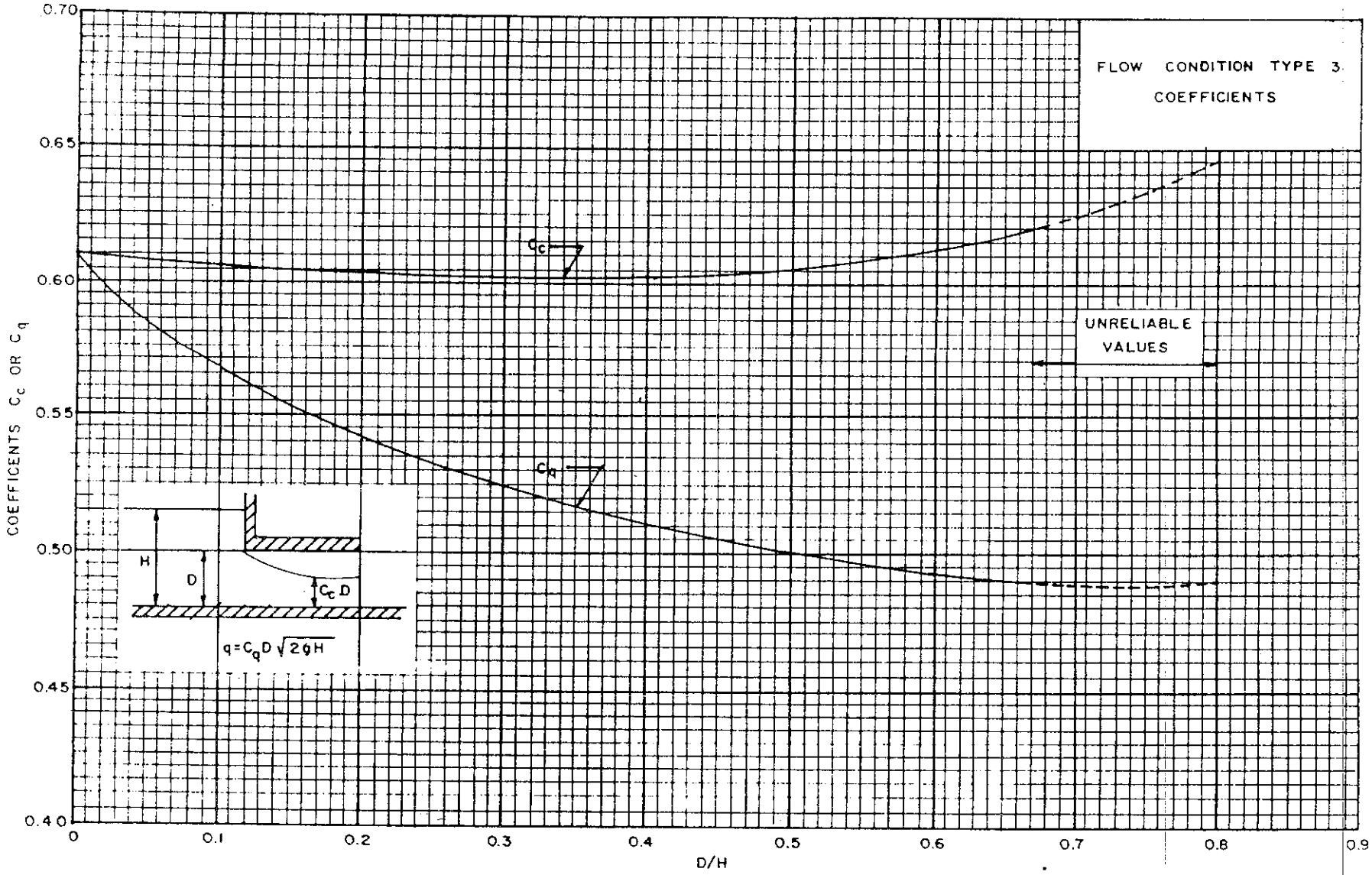

STAGE DISCHARGE CURVE
3 vents (5'x6')
ELEVATION [FT]
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
DISCHARGE [CFS]
CONDITION 5
CONDITION 3

Figure 15.


FLOW CONDITION TYPE 3
COEFFICIENTS
UNRELIABLE VALUES
C_c
C_q
H
D
$C_c D$
$q = C_q D \sqrt{2gH}$
COEFFICENTS C_c OR C_q
D/H
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9

13.0 2s/t + D CURVE

The computations for this curve are performed in Table 22. Column (2) is the storage capacity in acre ft taken from Fig.13. Column 3 is stoarage capacity in cubic-ft (1 acre ft = 43560 cu ft) divided by time. We have selected a time interval of 21,600 seconds (6 hours). Discharge in column (4) is taken from the stage discharge curve Fig. 14. Column (4) Vs column (5) is plotted on Fig. 16.

TABLE-22

COMPUTATIONS FOR 2s/t + D CURVE

Elevation in ft.	Storage volume(s) in Acre ft	2s/t in cfs	D in cfs	2s/t + D in cfs
(1)	(2)	(3)	(4)	(5)
14	0	0	0	0
15	25	101	40	141
16	126	508	114	622
17	254	1428	209	1637
18	758	3057	322	3379
19	1390	5606	450	6056
20	2275	9197	591	9788
21	3354	13528	745	14273
22	4754	19175	910	20085
23	6877	27745	1086	28831
24	10058	40567	1134	41701
25	14514	58540	1192	59732
26	20374	82176	1263	83438
27	27604	111336	1319	112655
28	36034	145337	1391	146728
29	45283	182641	1440	184081
30	55134	222374	1504	223878

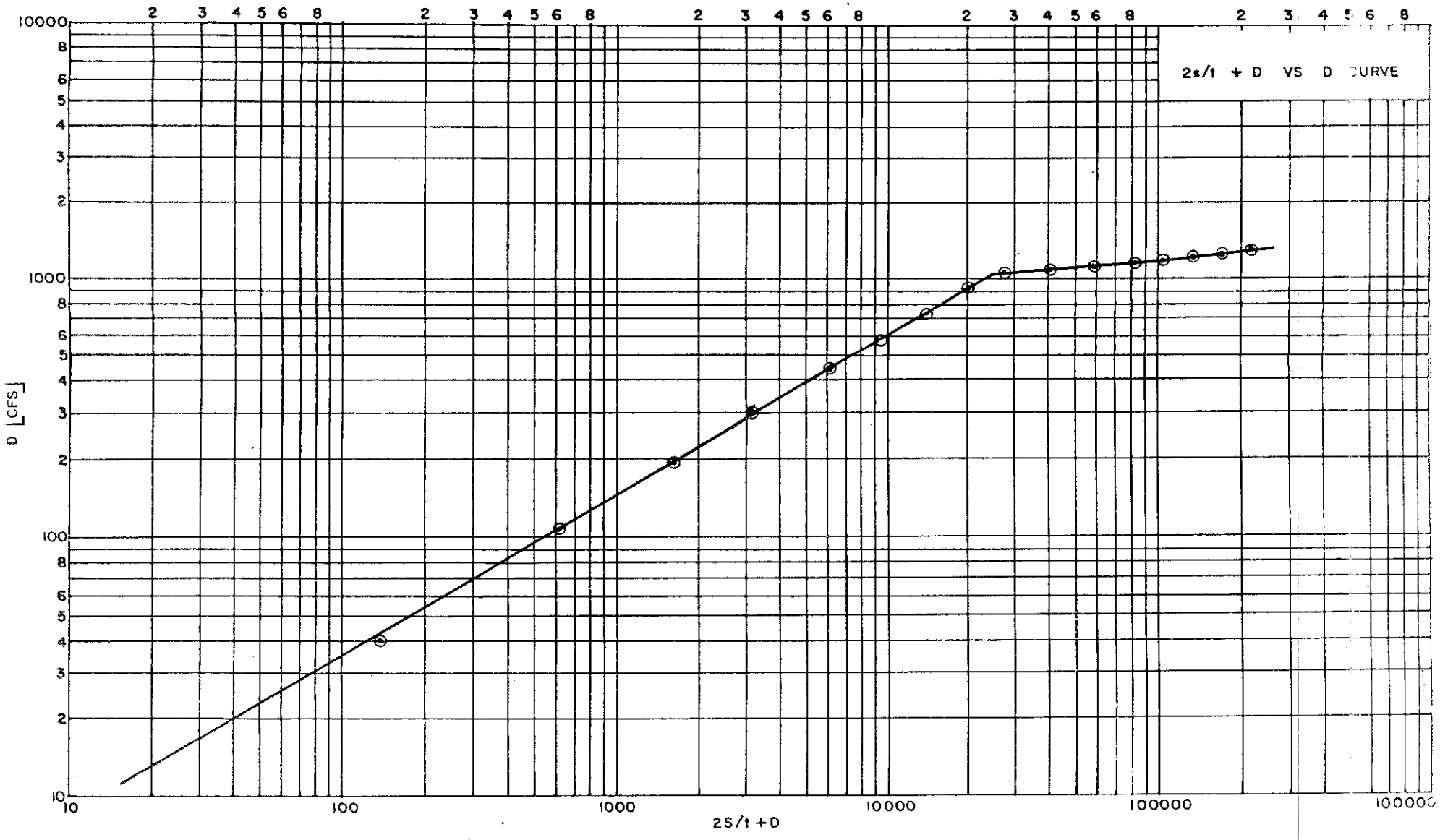


Fig. 16

14.0 PRE-MONSOON FLOOD ROUTING EXAMPLE (3 vents 5'x6')

The design flood, represented by the runoff hydrograph in Figure-9 will now be routed through a regulator containing 3 vents, each 5 ft. wide and 6 ft. high. The computations are done in Table 23, 24 and 25. The procedure is as follows:

1) The first value in column (6) and (8) are known or assumed.

2) The first value in column (5) is the value of 2s/t + D taken from Fig-16 corresponding to the known value of D in this case zero.

3) In subsequent lines column (5) is column (4) plus column (7) from the line above.

4) Column (7) is column (5) minus twice column (6) in each case throughout the computations.

5) After the first line, value of D in column (6) are taken from Fig-16 corresponding to the value of 2s/t + D on the same line.

6) Column (8) is the basin water surface elevation taken from Fig-14 for the corresponding value of D in column (6).

After completing the computations the outflow hydrograph i.e. discharge Vs time is plotted & presented in Fig-17. To assist in determining the

extent of crop damage, a basin water surface elevation Vs time is also plotted and presentd in Fig-18. This completes the flood routing procedure.

Comparison of inflow and outflow hydrographs for unit hydrograph method, Richard's method and SCS method are presented in Fig. 19, Fig. 20 & Fig. 21 respectively.

Premonsoon flood routing computations by Unit Hydrograph method, Richard's method and SCS method are presented in Table 23, Table 24 & Table 25 respectively while the results of the computations are discussed in Article 16.0.

TABLE-23

PRE-MONSOON FLOOD ROUTING COMPUTATION BY UNIT HYDROGRAPH METHOD
(3 vents 5'x6')

Day	Hour	I (cfs)	$I_1 + I_2$	2s/t + D (4)+(7)	D (cfs)	2s/t - D (5)-2x(6)	Basin Water Surface Elevation
1	2	3	4	5	6	7	8
	6	0	0	0	0	0	14.00
1	12	12	12	12	10	-8	14.30
	18	38	50	42	21	0	14.50
	24	88	126	126	40	46	15.00
	6	135	223	269	63	143	15.25
2	12	185	320	463	90	283	15.60
	18	228	413	696	115	466	16.05
	24	532	760	1226	160	906	16.45
	6	900	1432	2338	250	1926	17.30
3	12	1283	2183	4109	350	3409	18.20
	18	1223	2506	5915	430	5055	18.85
	24	1111	2334	7389	505	6379	19.40
	6	930	2041	8420	540	7340	19.60
4	12	733	1663	9003	560	7883	19.75
	18	538	1271	9154	565	8024	19.80
	24	353	891	8915	550	7815	19.70
	6	184	537	8352	535	7282	19.58
5	12	62	246	7528	510	6508	19.42
	18		88	6596	460	5676	19.05
	24	6	32	5708	425	4858	18.80
	6	0	6	4864	380	4104	18.45
6	12	0	0	4104	350	3404	18.20
	18	0	0	3404	325	2754	18.05
	24	0	0	2754	260	2234	17.45
	6	0	0	2234	235	1764	17.20
7	12	0	0	1764	210	1344	17.05
	18	0	0	1344	170	1004	16.50
	24	0	0	1004	145	714	16.30

TABLE-21

PRE-MONSOON FLOOD ROUTING COMPUTATION BY RICHARD'S METHOD

Day	Hour	I (cfs)	$I_1 + I_2$	$\frac{2s}{t}+D$ (4)+(7)	D (cfs)	$\frac{2s}{t}-D$ (5)-2x(6)	Basin Water Surface Elevation
1	2	3	4	5	6	7	8
	6	0	0	0	0	0	14.00
1	12	45	45	45	22	1	14.50
	18	140	185	186	52	82	15.15
	24	470	610	692	115	462	16.05
	6	940	1410	1872	210	1452	17.05
2	12	1300	2240	3692	320	3052	18.00
	18	1426	2726	5778	425	4928	18.80
	24	1380	2806	7734	515	6704	19.45
	6	1270	2650	9354	580	8194	19.90
3	12	930	2200	10394	610	9174	20.10
	18	430	1360	10534	615	9304	20.15
	24	110	540	9844	585	8674	19.90
	6	0	110	8784	545	7694	19.65
4	12	0	0	7694	510	6674	19.42
	18	0	0	6674	460	5754	19.05
	24	0	0	5754	425	4904	18.80
	6	0	0	4904	385	4134	18.46
5	12	0	0	4134	350	3734	18.20
	18	0	0	3734	325	3084	18.05
	24	0	0	3084	290	2504	17.70
	6	0	0	4904	385	4134	18.46
6	12	0	0	1994	220	1554	17.10
	18	0	0	1554	190	1174	16.80
	24	0	0	1174	160	854	16.45
	6	0	0	854	120	614	16.08
7	12	0	0	614	105	404	15.80
	18	0	0	404	82	240	15.55
	24	0	0	240	60	120	15.20

TABLE-25

PRE-MONSOON FLOOD ROUTING COMPUTATION BY SCS METHOD

Day	Hour	I (cfs)	$I_1 + I_2$	$\frac{2S}{t} + D$ (4) + (7)	D (cfs)	$\frac{2S}{t} - D$ (5) - 2x(6)	Basin Water Surface Elevation
1	2	3	4	5	6	7	8
	3	0	0	0	0	0	14.00
	6	143	143	143	41	61	15.00
	9	442	585	646	110	426	15.90
1	12	941	1383	1809	215	1379	17.10
	15	1325	2266	3645	310	3025	17.90
	18	1425	2750	5775	425	4925	18.80
	21	1325	2750	7675	510	6655	19.42
	24	1112	2437	9092	562	7968	19.75
	3	798	1910	9878	580	8718	19.90
	6	556	1354	10072	600	8872	20.00
	9	399	955	9827	585	8657	19.90
2	12	295	694	9351	580	8196	19.90
	15	210	505	8696	542	7612	19.60
	18	153	363	7975	520	6935	19.50
	21	110	263	7198	505	6188	19.40
	24	78	188	6376	455	5466	19.05
	3	57	135	5601	422	4757	18.80
	6	41	98	4855	380	4095	18.45
	9	30	71	4166	355	3456	18.25
3	12	21	51	3507	320	2867	18.00
	15	16	37	2904	290	2324	17.70
	18	12	28	2352	250	1852	17.30
	21	9	21	1873	210	1453	17.05
	24	6	15	1468	175	1118	16.55
	3	3	9	1127	160	807	16.45
	6	0	3	810	118	574	16.05
	9	0	0	574	102	370	15.70
4	12	0	0	370	70	230	15.45
	15	0	0	230	60	110	15.20
	18	0	0	110	35	40	14.90
	21	0	0	40	20	0	14.50
	24	0	0	0	0	0	14.00

Fig. 17

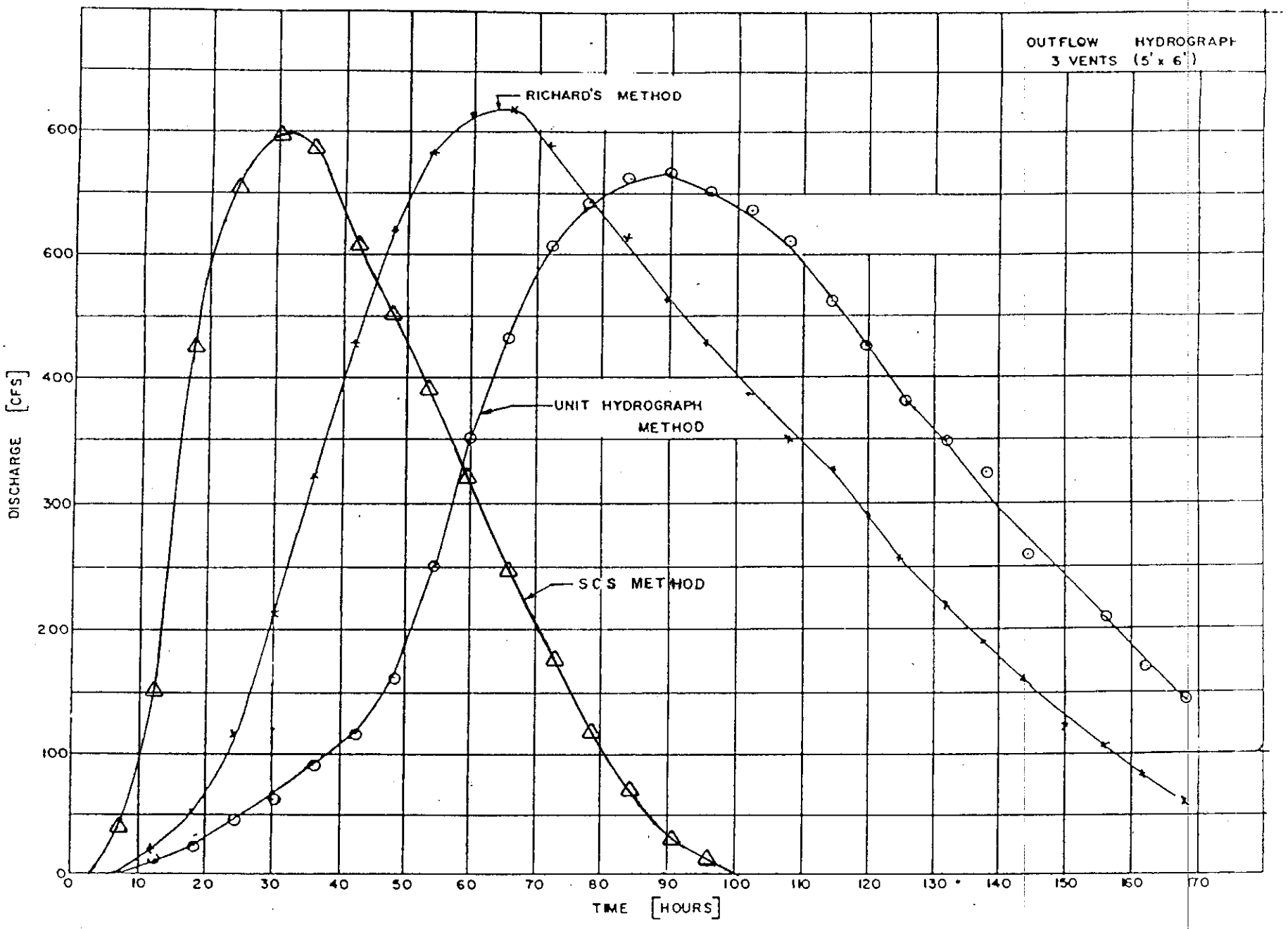
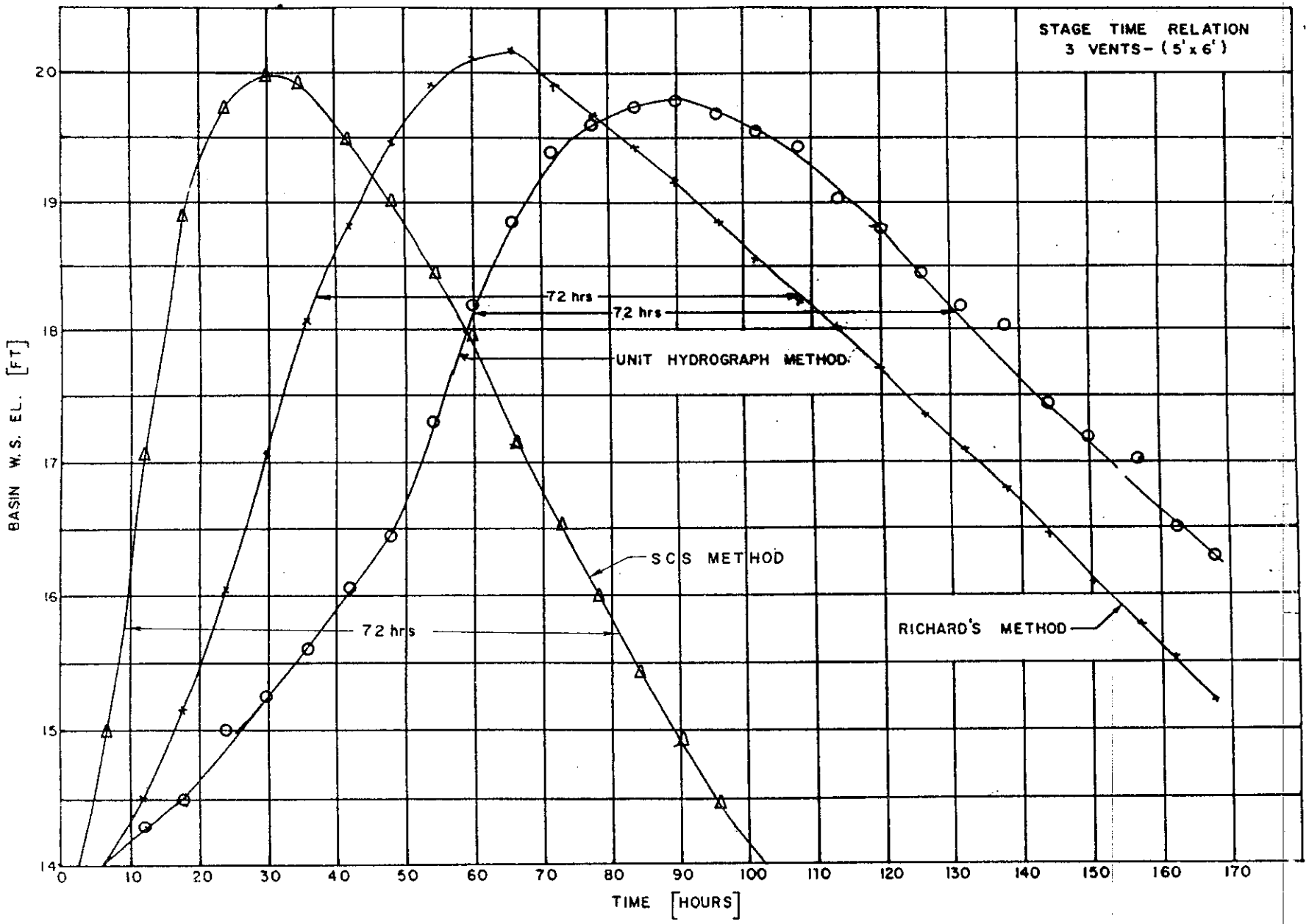

OUTFLOW HYDROGRAPH
3 VENTS (5' x 6')
RICHARD'S METHOD
UNIT HYDROGRAPH METHOD
SCS METHOD
DISCHARGE [CFS]
TIME [HOURS]
0
100
200
300
400
600
600
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170

Figure 18.


STAGE TIME RELATION
3 VENTS- (5' x 6')
72 hrs
72 hrs
UNIT HYDROGRAPH METHOD
SCS METHOD
72 hrs
RICHARD'S METHOD
BASIN W.S. EL. [FT]
TIME [HOURS]
20
19
18
17
16
15
14
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170

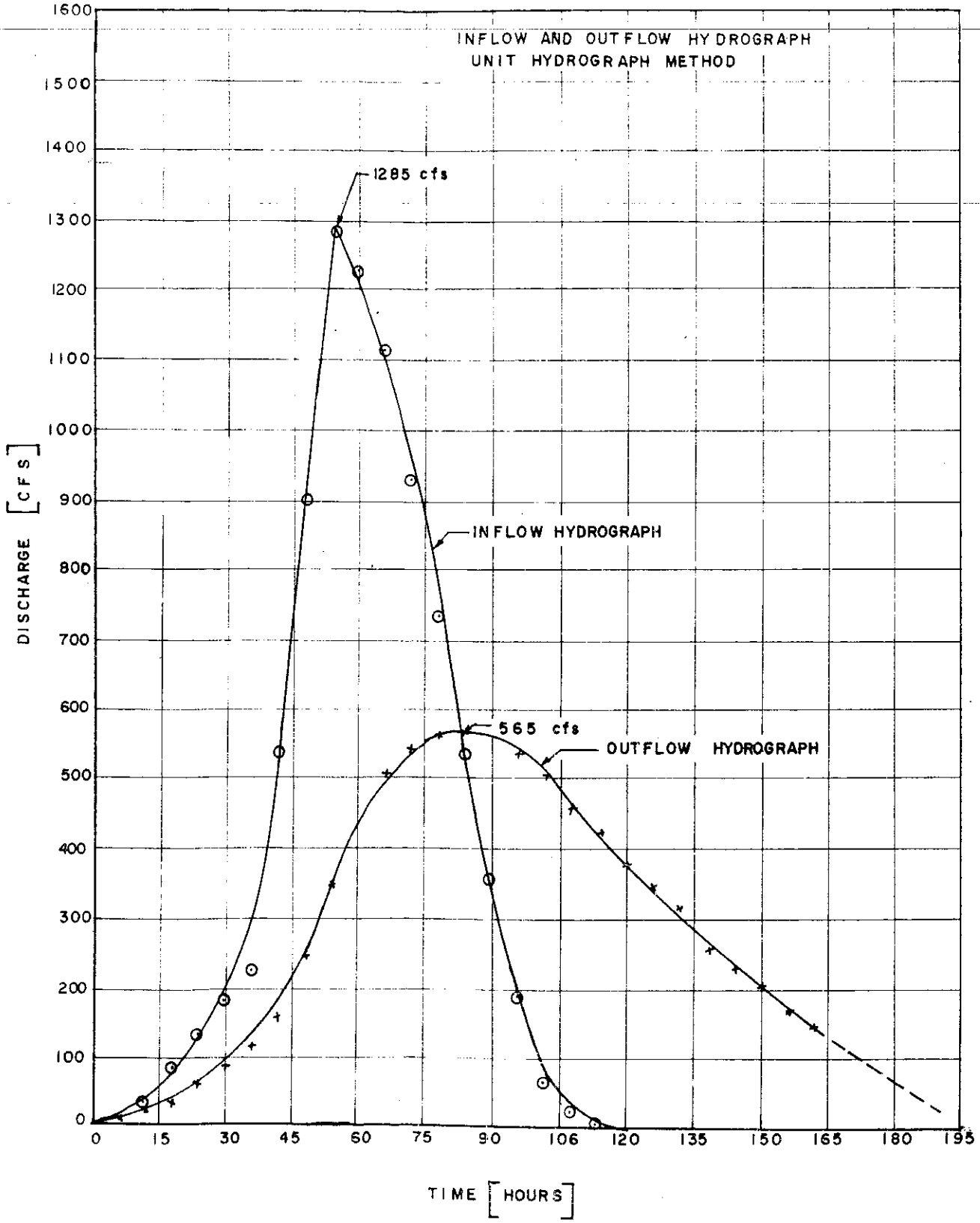

INFLOW AND OUTFLOW HYDROGRAPH
UNIT HYDROGRAPH METHOD
1285 cfs
INFLOW HYDROGRAPH
565 cfs
OUTFLOW HYDROGRAPH
DISCHARGE [CFS]
TIME [HOURS]
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
0
15
30
45
60
75
90
106
120
135
150
165
180
195

FIG. 19

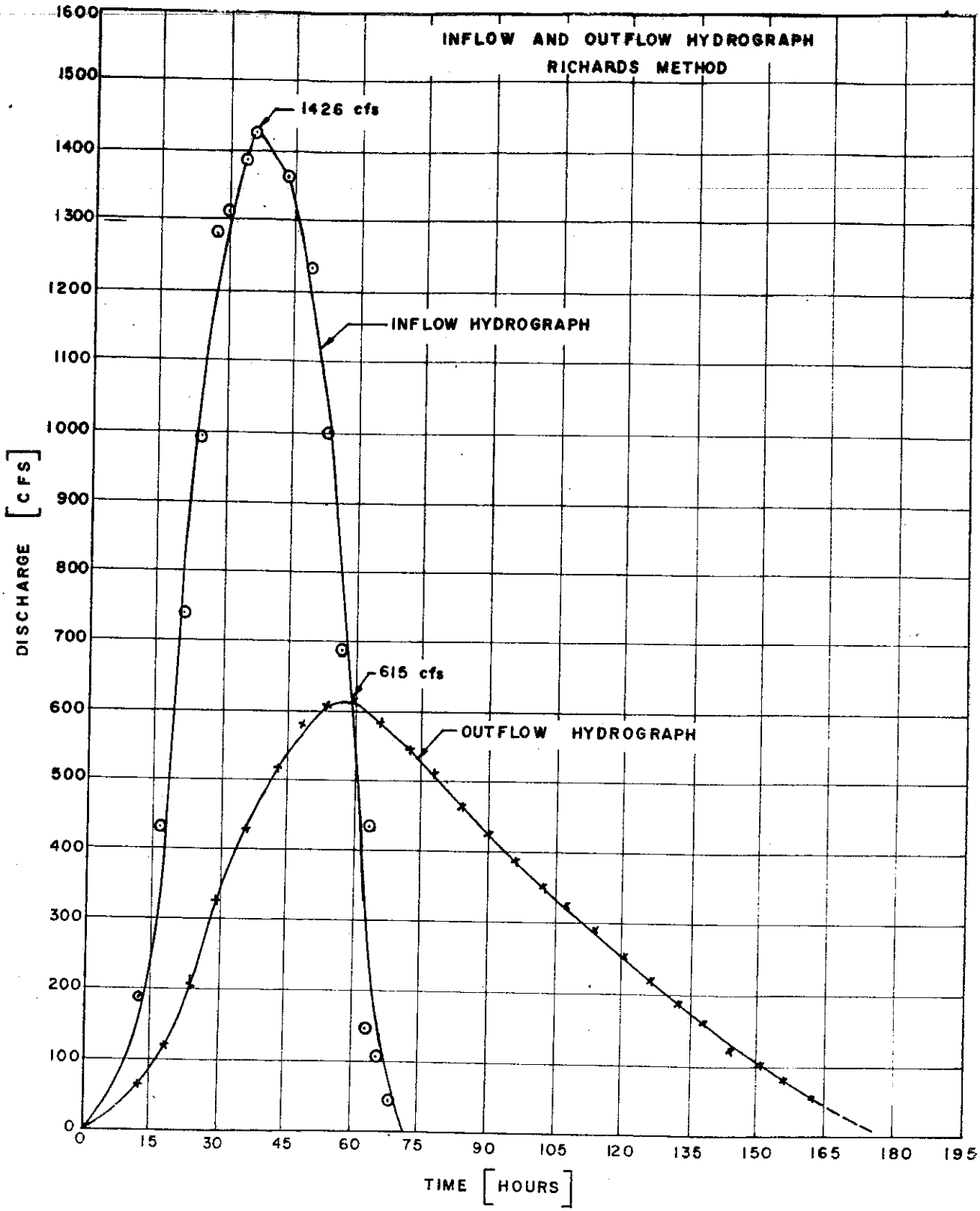

INFLOW AND OUTFLOW HYDROGRAPH
RICHARDS METHOD
1426 cfs
INFLOW HYDROGRAPH
615 cfs
OUTFLOW HYDROGRAPH
DISCHARGE [CFS]
TIME [HOURS]
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
0
15
30
45
60
75
90
105
120
135
150
165
180
195

FIG. 20

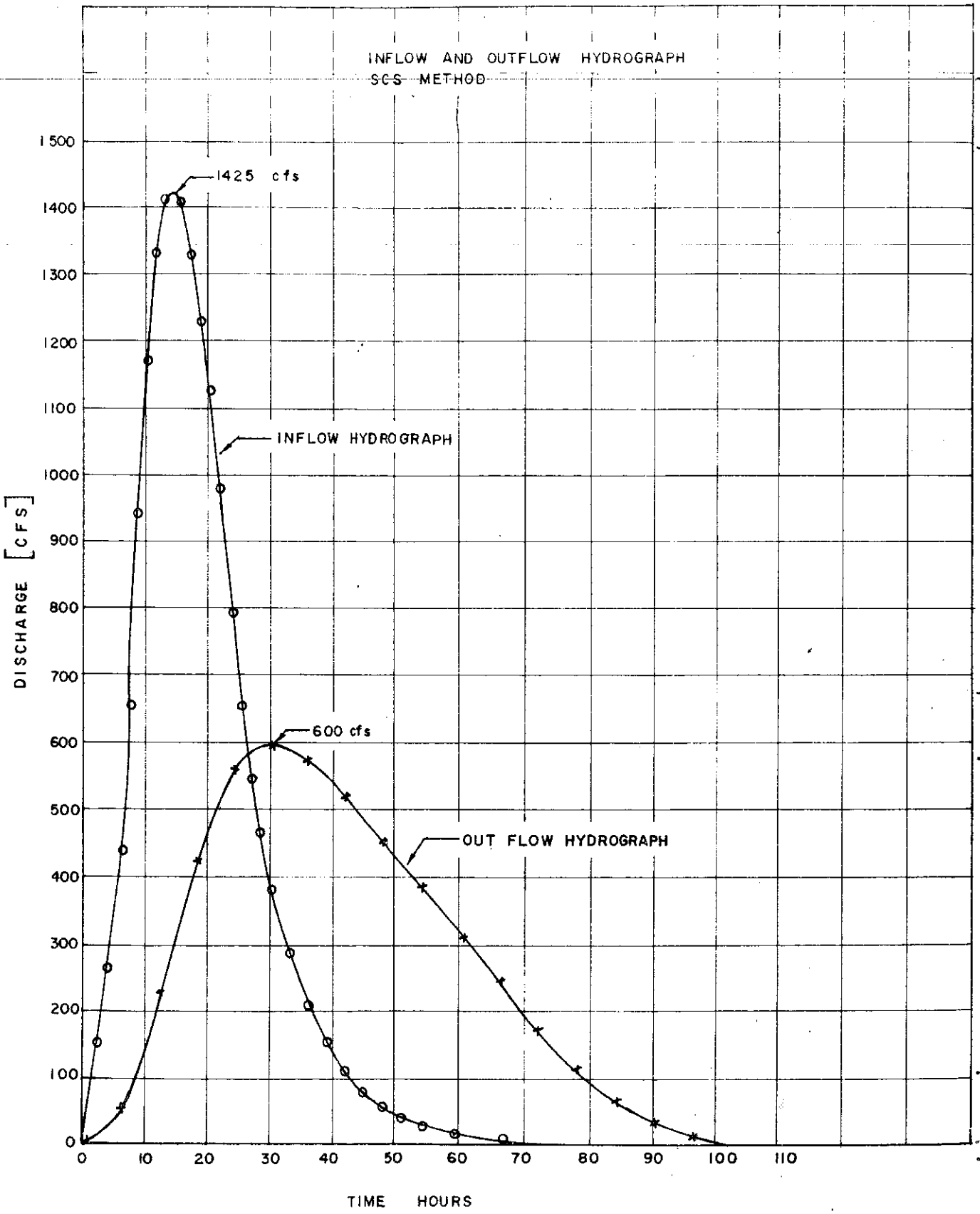

INFLOW AND OUTFLOW HYDROGRAPH
SCS METHOD
1425 cfs
INFLOW HYDROGRAPH
600 cfs
OUT FLOW HYDROGRAPH
DISCHARGE [CFS]
TIME HOURS
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
0
10
20
30
40
50
60
70
80
90
100
110

FIG. 21

TABLE-26

RAINFALL RUNOFF VOLUME (NON PADDY LAND)

Month	Monthly Rainfall (inch)	Monthly evapo-transpi-ration (inch)	Initial soil Moisture loss (inch)	Depression storage (inch)	Runoff (inch)	Runoff volume (Acre ft.)
June	13.18	4.6	0.5	1.0	7.18	6407
July	13.70	4.6	-	-	9.10	8120
August	11.72	4.4	-	-	7.32	6531
Sept.	8.56	3.9	-	-	4.66	4158
Oct.	6.02	3.7	-	-	2.32	2070

TABLE-27

RAINFALL RUNOFF VOLUME (PADDY LAND)

Month	Monthly Rainfall (inch)	Monthly evapo-transpi-ration (inch)	Initial soil Moisture loss (inch)	Depression storage (inch)	Runoff (inch)	Runoff volume (Acre ft.)
June	13.18	6.1	-	4.0	3.08	2748
July	13.70	6.2	-	-	7.50	6692
August	11.72	5.9	-	-	5.82	5193
Sept.	8.56	5.2	-	-	3.36	2998
Oct.	6.02	5.3	-	-	0.72	643

15.0 POST MONSOON ROUTING

As the river stage starts rising the gates of the regulator is closed before the flooding inside the project area exceeds the tolerable inundation level. For our case it is +20.00 which the outfall river attains on 15th June (Fig.22). During the period the gates are closed and flood water cannot enter into the project area provided the project area is completely empoldered, but the excess rainfall runoff will be accumulated inside the basin. Computations of rainfall runoff volume for non-paddy land, paddy land and weighted basin average for different months are presented in Table 26, Table 27 and Table 28 respectively. The basin water surface elevation after each month is computed in Table 29 and presented in Fig 22. It may be observed that the basin water level always remains below the level in the peripherial rivers. The difference is the benefit accrued out of the project by reclaiming land which were subjected to annual flooding during pre-project condition. It may also be observed from Fig. 22 that at falling limb of the outfall hydrograph on 9th October the river water level coincide with the basin water level. The gates are opened on that day and the post monsoon routing begins. The computations are presented in Table 30

TABLE-28

RAINFALL RUNOFF VOLUME (WEIGHTED BASIN AVERAGE)

Month	Non paddy land 50%		Paddy land (50%)		Basin Weighted Runoff(Acre ft.
	Net Runoff	Weighted Run off	Net Run off	Weightted Runoff	
June	6407	3204	2748	1374	4578
July	8120	4060	6692	3346	7406
August	6531	3266	5193	2597	5863
Sept.	4158	2079	2998	1499	3578
Oct.	2070	1035	643	322	1357

TABLE-29

COMPUTATION OF INSIDE STORAGE AND WATER LEVEL

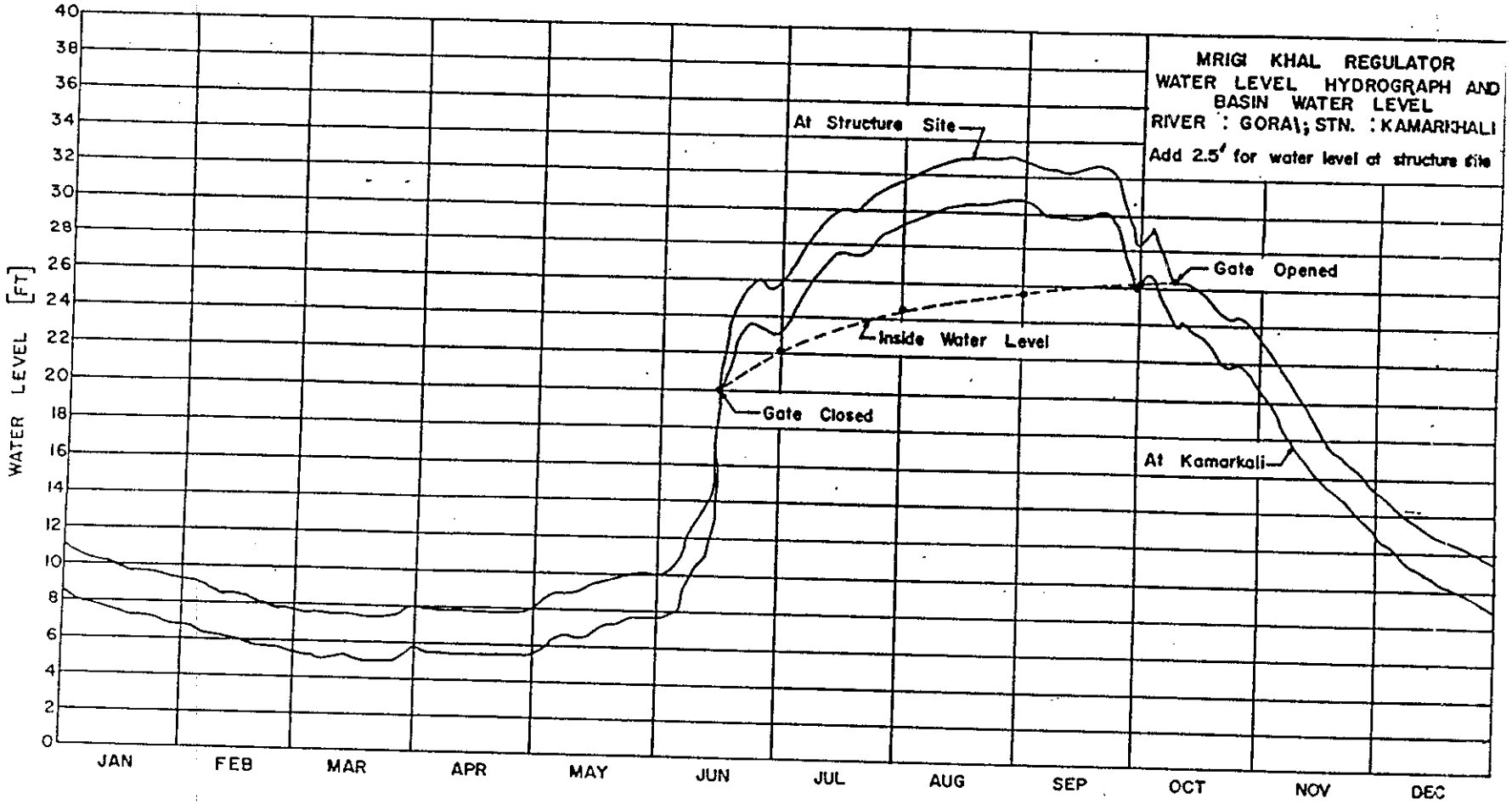
Date	River W/L (in ft.PWD)	Run off volume (Acre ft.)	Gate position	Inside	
				Storage Acre ft.	Water Level (in ft. PWD)
15th June	20.0	2275	Gate closed	2275	20.0
30th June	25.8	2289		4564	21.86
31st July	31.4	7406		11970	24.43
31st August	32.6	5863		17833	25.57
30th Sept.	28.4	3578		21411	26.14
9th Oct.	26.2	394	Gate opened	21805	26.20

TABLE-30

POST MONSOON FLOOD ROUTING COMPUTATIONS (3 Vents 5' x 6')

Date	C/s water level W 1	R/s water level W 2	Δh (W -W) 1 2	Total discharge Q(Acre ft	Residual storage (Acre ft	Correspondence C/s water level
Oct. 9	26.20	26.20	0	0	21805	26.20
10	26.20	25.98	0.22	551	21254	26.12
11	26.12	25.76	0.36	705	20549	26.02
12	26.02	25.54	0.48	814	19735	25.89
13	25.89	25.32	0.57	889	18846	25.73
14	25.73	25.10	0.63	933	17913	25.58
15	25.58	24.88	0.70	983	16930	25.41
16	25.41	24.66	0.75	1017	15913	25.23
17	25.23	24.44	0.79	1044	14869	25.06
18	25.06	24.22	0.84	1077	13792	24.83
19	24.83	24.0	0.83	1070	12722	24.59
20	24.59	23.78	0.81	1058	11664	24.36
21	24.36	23.86	0.80	1051	10613	24.12
22	24.12	23.34	0.78	1038	9575	23.84
23	23.84	23.12	0.72	997	8575	23.53
24	23.53	22.90	0.63	932	7646	23.24
25	23.24	22.68	0.56	879	6767	22.94
26	22.94	22.46	0.48			

Figure 21


MRIGI KHAL REGULATOR
WATER LEVEL HYDROGRAPH AND BASIN WATER LEVEL
RIVER : GORAI; STN. : KAMARKHALI
Add 2.5' for water level at structure site
At Structure Site
Gate Opened
Inside Water Level
Gate Closed
At Kamarkali
WATER LEVEL [FT]
40
38
36
34
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16.0 DISCUSSION AND CONCLUSION

The results of the premonsoon routing for 3 vents of size 5'x6' are tabulated below in Table 31.

TABLE-31

PRE-MONSOON FLOOD ROUTING RESULTS (3 vents 5'x6')

Item	Unit Hydrograph Method	Richard's Method	SCS Method
1) Runoff hydrograph peak discharge	1283 cfs	1426	1425 cfs.
2) Time to runoff hydrograph peak	54 hours	36 hours	15 hours
3) Time base of runoff hydrograph	120 hours	72 hours	75 hours
4) Out flow hydrograph peak	565 cfs	615 cfs	600 cfs.
5) Basin storage level corresponding to peak discharge	+19.80(PWD)	+20.15(PWD)	+ 20.00 (PWD)
6) Percentage of area submerged corresponding to peak discharge	8.97%	9.63%	9.44%
7) Basin submergence level for more more than 72 hours	+18.10(PWD)	+18.25(PWD)	+ 15.75 (PWD)
8) Percentage of area submerged for more than 72 hours	4.96%	5.30%	1 %

From Table 31 it may be observed that out of the three methods Richard's method is little more conservative & dectates the designs to go for next higher size of the structure.

Considering the simplicity of computation by Richard's method & SCS method when there is time constraint for detail design computation, the Richard's method or SCS method for generating runoff hydrograph may be used.

The computed size of the structure selected by pre-monsoon routing will also be checked by post-monsoon routing. In non tidal zone it is observed that the size of the structure is mostly governed by post-monsoon conditions. However when more sophistication in design computations is required, specially for a large structure the conventional unit hydrograph method for runoff hydrograph computation may be recommended. For Mrigi Khal regulator the results of post-monsoon routing shows that the maximum difference of water level between C/s and R/s is about 10" for a 3 vents regulator of size 5'x6'. It may therefore be concluded that a 3 vents drainage regulator of size 5'x6' placed at the outfall of Mrigi Khal is adequate for Mrigi Basin.

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CHAPTER III

HYDRAULIC DESIGN

G.M.A. HOSSAIN

CHAPTER III

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1.0 Introduction

Structure designed to control and or modify the flow of water in channels, natural or man made and to control the flow of water into or out of a particular area (e.g. a coastal sluice) and to control the forces imposed by flowing water, are called hydraulic stuctures. Bridges and offshore structures e.g. jetties, quaywalls etc. are also called hydraulic structures but those are not included in this text as we are concerned with only small scale water control structures. Size of structures may vary from small installations (e.g. farm turnouts, canals) handling just a few cubic feet per second or less to large structures capable of controlling and withstanding flows in excess of 10,000 cft per second. The purpose of this paper is to present in brief the basic guide lines and criteria to be followed in the design of small hydraulic stuctures like weirs, regulators, water retention structure and sluices.

2.0 Types of Flow

2.1 Uniform Flow

The three types of flow conditions encountered near or within control structures are uniform flow, gradually varied flow and rapidly varied flow. Uniform flow generally occurs in hydraulically rough channels having uniform slopes and constant cross-section over a considerable distance. However, uniform flow can be assumed for special reaches of streams which, over a long period of time, have established a course through their

own deposits. Such reaches show a remarkable regularity of slope and cross-section. The basic equations required for solving the problem of uniform flow are given as :

$$V = C\,(RS)^{1/2} \qquad \ldots\ldots \quad (1)$$

$$V = \frac{1.486}{n}\,R^{2/3}\,S^{1/2} \qquad \ldots\ldots \quad (2)$$

Where, V = average velocity in a cross-section (ft/sec)

C = Chezy's Coefficient . . . ($ft^{1/2}$/sec)

R = hydraulic radius $\frac{A}{P}$ . . . (ft)

A = cross-sectional area . . . (ft^2)

P = wetted perimeter of the cross-section (ft)

S = longitudinal slope

n = Manning's roughness . . . ($sec/ft^{1/3}$)

$$C = \frac{1.486}{n}\,R^{1/6} \qquad \ldots\ldots \quad (3)$$

$$Q = VA \qquad \ldots\ldots \quad (4)$$

Design aids for solving these equations are given in Charts 1 to 13 of Appendix A. Table 1 of Appendix A gives values for Manning's "n".

2.2 Gradually Varied Flow

In the vicinity of control structures the depth of flow in streams and man made canals does not remain constant as in uniform flow. The depth of flow increases or decreases as the control structure is approached and the flow either accelerates or decelerates. The type of flow considered in this section changes gradually with distance. The

mechanics of gradually varied flow are dependent on the Bernoulli Equation which is expressed as shown :

$$\frac{P_1}{\gamma} + Z_1 + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + Z_2 + \frac{V_2^2}{2g} + (\text{energy loss}) \quad . . (5)$$

γ is the unit weight of water = 62.4 lb^3/ft

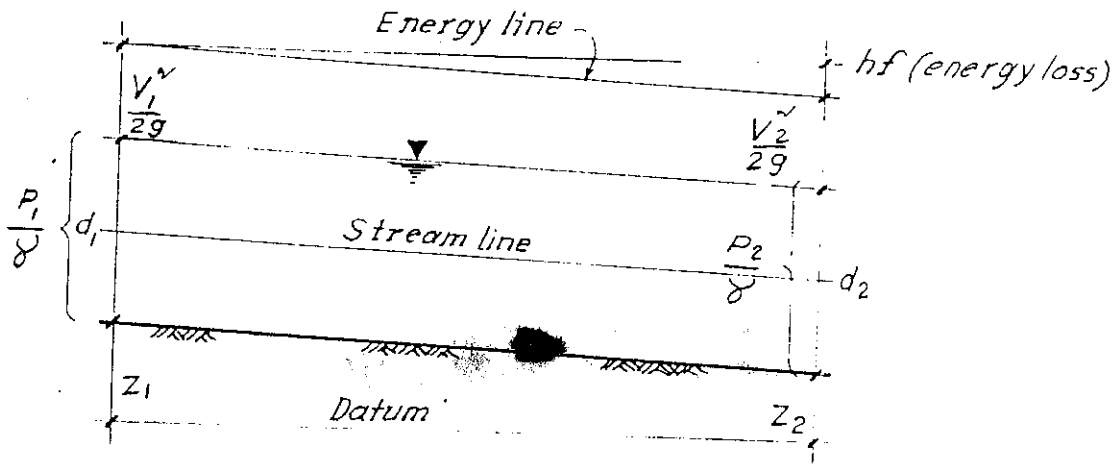


Fig. 1 : Defination sketch for Bernoulli's equation.

Strictly speaking the Bernoulli equation applies only between points on the same streamline. A velocity head coefficient "α" must therefore be applied to take care of the non-uniform velocity distribution in the cross-section but for all practical purposes this coefficient is unity and it shall be disregarded. Additionally, since the curvature of the streamlines is very minor in gradually varied flow the pressure distribution is essentially hydrostatic and the term (P_1/γ) becomes ($\gamma d_1/\gamma$), the

depth of flow. The equation becomes.

$$d_1 + Z_1 + \frac{V_1^2}{2g} = d_2 + Z_2 + \frac{V_2^2}{2g} + hf \qquad \ldots\ldots \quad (6)$$

where hf = energy loss

g = acceleration due to gravity

Z = height of bed above datum (ft)

d = depth of flow

v = average velocity in cross-section (ft/sec)

To solve problems of gradually varied flow, the slope of the energy grade line must be known. In uniform flow the energy grade line parallels the water surface but the two lines are not parallel in gradually varied flow. Nonetheless, the problems of gradually varied flow are solved by assuming that the Chezy and Manning's relationships also hold for gradually varied flow. Starting from these equations a differential equation is derived. Rewriting the Bernoulli Equation in the form

$$H = Z + d + \frac{V^2}{2g} \qquad \ldots\ldots \quad (7)$$

(various terms are shown in Figure 2), allows for solution to gradually varied flow problems.

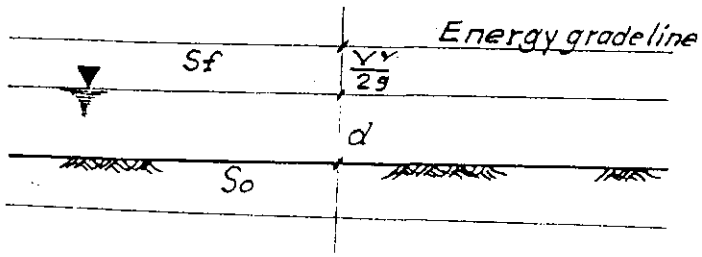


Fig. 2 : Definition sketch for gradually varied flow.

$$\frac{dH}{dx} = - S_f \qquad \ldots\ldots \quad (8)$$

Thus $\frac{d}{dx}\left(Z + d + \frac{V^2}{2g}\right) = - S_f$

Now $\frac{dZ}{dx}$ = slope of the bed = $- S_o$

$\frac{dd}{dx}$ = rate of change in depth with x.

and $\frac{d}{dx}(V^2/2g) = \frac{d}{dx}\left(\frac{Q^2}{2gA^2}\right) = \frac{-Q^2}{gA^3}\frac{dA}{dx}$

Now $\frac{dA}{dx} = \frac{d}{dx}(Bd) = B\frac{dd}{dx}$ resulting in

$$- S_O + \frac{dd}{dx}\frac{(1-Q^2B)}{gA^3} = - S_f$$

$$\frac{dd}{dx} = \frac{S_O - S_f}{\frac{1-Q^2B}{gA^3}} = \frac{S_O - S_f}{1 - \frac{V^2}{gd}} \qquad \ldots\ldots \quad (9)$$

$$\text{or} \quad \frac{dd}{dx} = \frac{S_o - S_f}{1 - Fr^2} \quad \text{where } Fr \text{ is equal to } \frac{V^2}{(gd)^{1/2}}$$

This equation is the final form of the equation for gradually varied flow.

The differential equation for gradually varied flow, equation (9) can only be integrated directly in special cases where simplifying assumptions have been adopted. One such case is the wide rectangular channel with constant Chezy's C. This integration was first performed by Bresse and later expanded by researchers Bakhmeteff and Ven Te Chow for application to trapezoidal channels. To evaluate more complicated situations, step methods must be used to perform the integration. The two basic computational methods are the direct step method which is applicable to uniform channels and the standard step method which does not require the channel to be prismatic and can therefore be used for natural water courses. More data is required for the standard step method than the direct step method and computational time is longer. These method are explained in standard textbooks on hydraulics and open channel flow and will not be discussed here.

In the following section the equations for the hydraulic jump, a special case of rapidly varied flow, will be presented.

2.3 Rapidly Varied Flow

Rapidly varied flow occurs at the crest of weirs and spillways, at the exit of under-flow gates, and at drops in channels. When this type of flow occurs water jets of high velocities are produced but these high velocities cannot be allowed to act indefinitely. Eventually those high velocity jets have to be broken up so that the flow becomes more managable.

The hydraulic jump is one of the most important examples of rapidly varied flow and for this reason the equations for the jump condition are presented here.

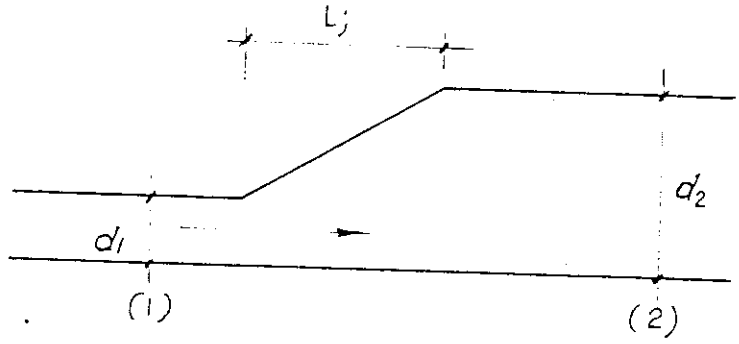


Fig. 3 : Definitation sketch for hydraulic jump.

To solve hydraulic jump problems, the engineer needs to solve for the depths, d_1 and d_2, depths before and after the jump, the length of the jump and the energy loss at the jump.

Equations required are provided below :

Starting with this equation :

$$V^2 = gd_1 \left(1 + \frac{d_2 - d_1}{d_1}\right) \left(1 + \frac{d_2 - d_1}{2d_1}\right) \quad \ldots\ldots (10)$$

and simplification leads to :

$$V_1^2/g = d_2 (d_1 + d_2)/(2d_1) \qquad \ldots\ldots (11)$$

Using the expression for the Froude number.

$$F_1 = \frac{V_1}{(gd_1)^{1/2}} \qquad \ldots\ldots (12)$$

Equation (11) can be re-written and then rearranged to :

$$d_2 = \frac{d_1}{2} [(1 + 8F_1^2)^{1/2} - 1] \qquad \ldots\ldots (13)$$

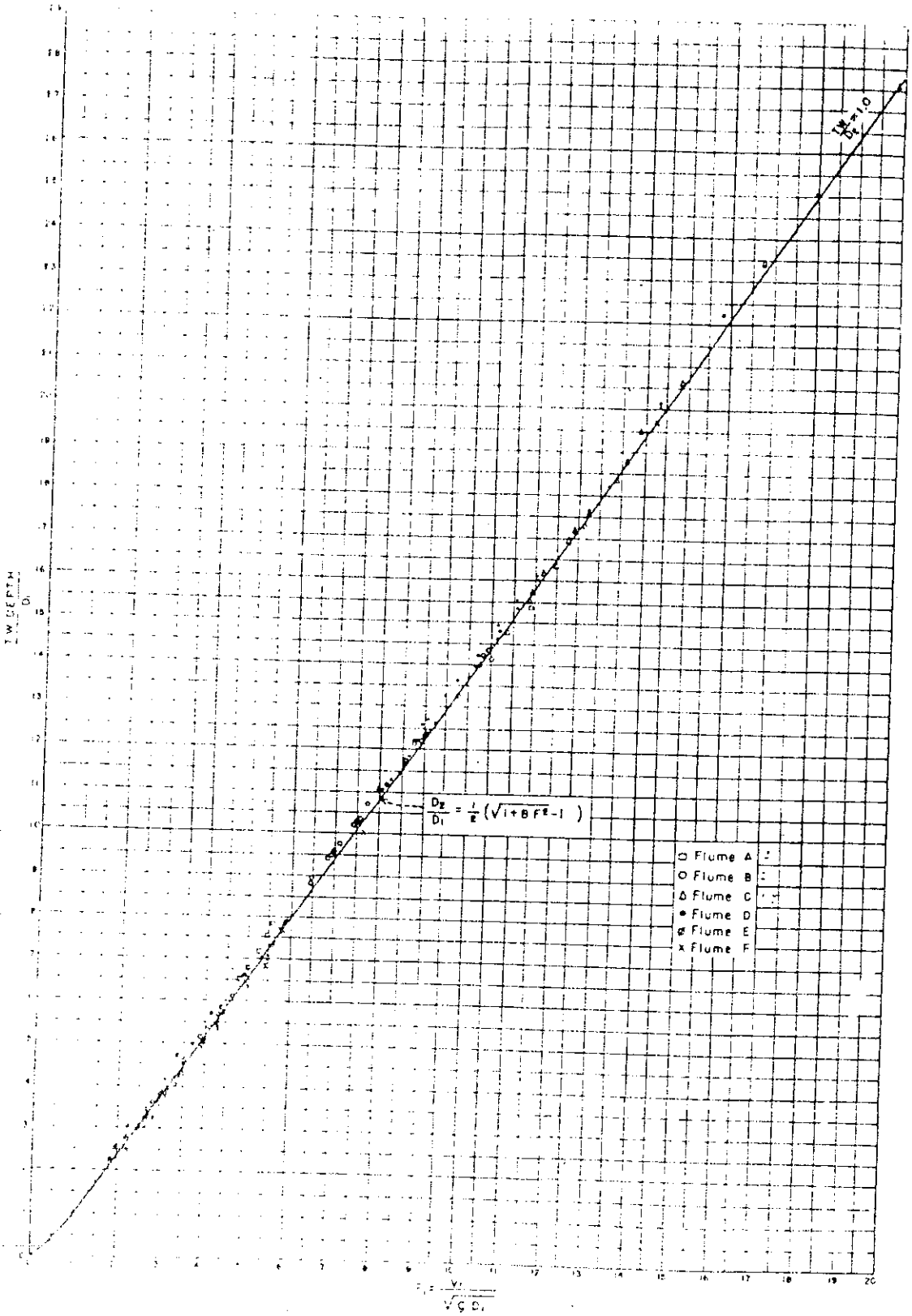
Figure 4 can be used to determine the ratio d_2/d_1 for various Froude numbers, <u>for stilling basins having vertical and parallel walls and a horizontal floor</u>. The length of the jump varies with the Froude Number of the incoming flow and the tailwater depth. In Figure 5, it can be seen that relationship between the length of the jump, L_j and the downstream depth, in most cases, can be taken to be

$$L_j = 6 d_2 \qquad \ldots\ldots (14)$$

The energy loss in the jet is obtained using the equation

$$E = \frac{(d_2 - d_1)^3}{4 d_2 d_1} \qquad \ldots\ldots (15)$$

and it is shown graphically in Figure 4 (a)



Source: Hydraulic Design of Stilling Basins and Energy Dessipators. U.S. Department of the Interior

Figure 4. Froude Number versus Tailwater Depth to D_1 Ratio for Horizontal Rectangular Channels

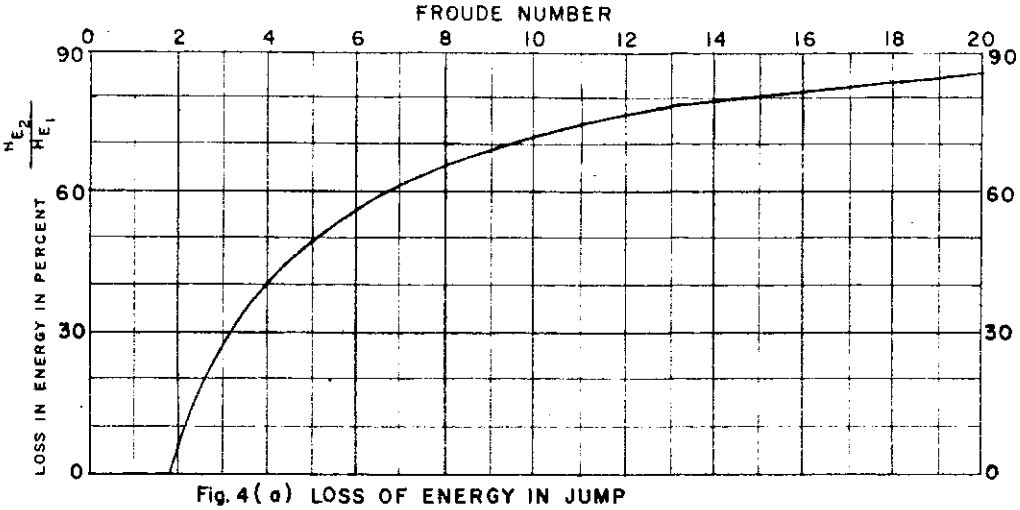


Fig. 4 (a) LOSS OF ENERGY IN JUMP

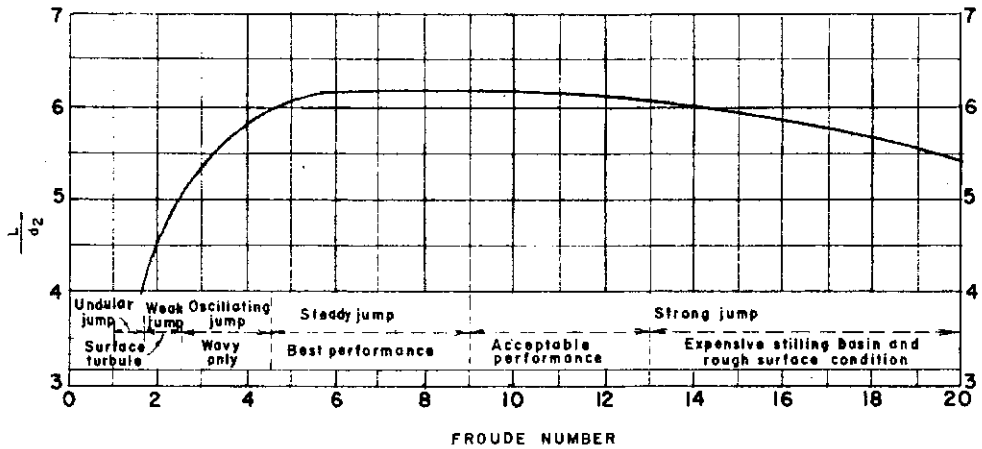
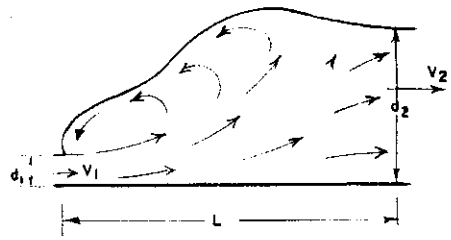


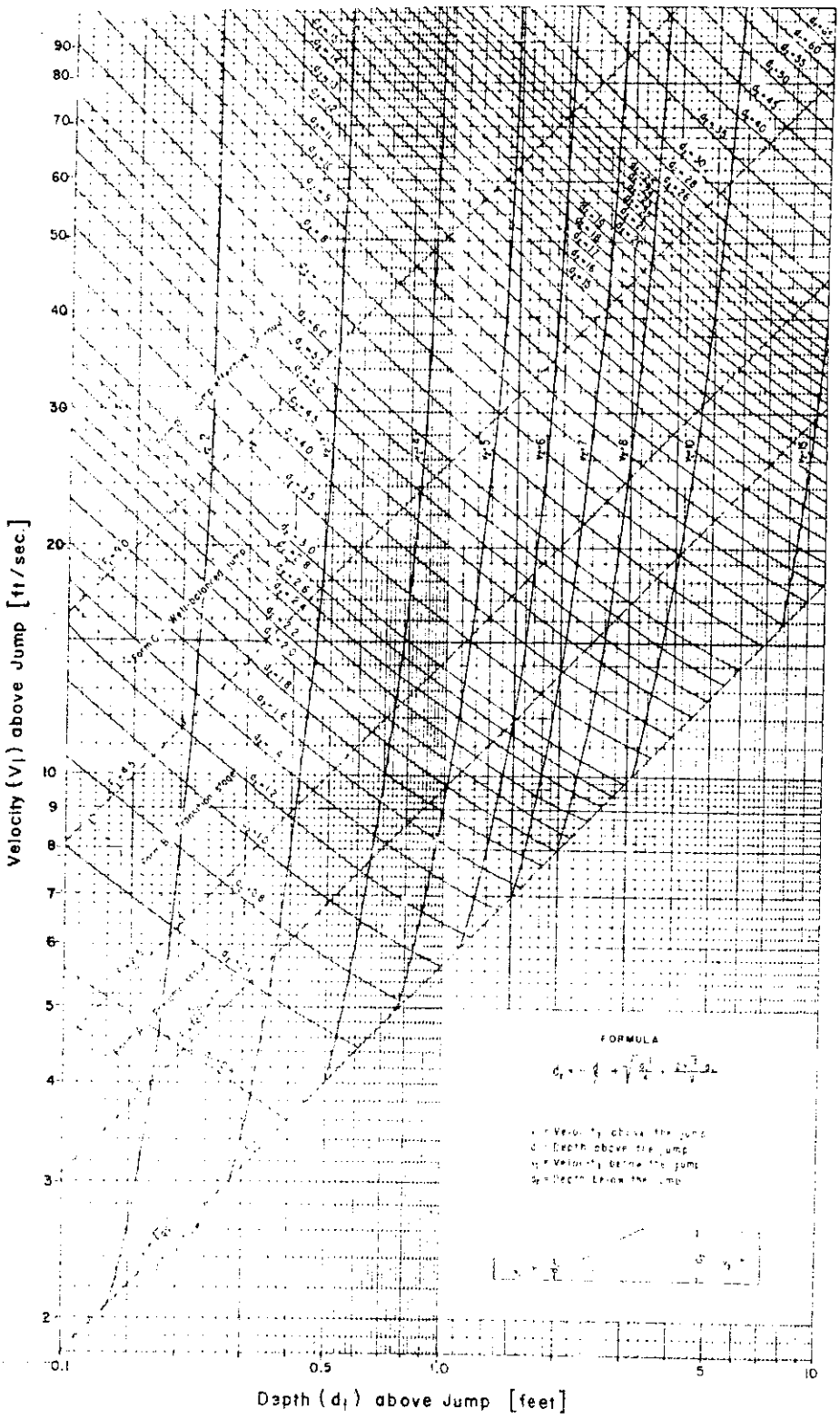
Fig. 4 (b) LENGTH OF JUMP



Sources: Fig. 4 (a) "Design of Small Dams" by U.S. Department of the Interior.

Fig. 4 (b) "Research Studies on Stilling Basins, Energy Dessipators and Associated Appurtenances," Hydraulic Lab. Report No. Hyd-399 by U.S. Department of the Interior.

Figure:-5. Properties of the Hydraulic Jump in Relation to the Froude Number.



Source: Design of Small Dams, U.S. Department of the Interior

Figure: 5.1 Relations Between Variables in Hydraulic Jump for Rectangular Channel

3.0 Small Hydraulic Structures

The most common types of small hydraulic structures, encountered in Bangladesh are weirs, regulators, water retention structure and sluices. A brief description for functions and utility of these structures are given in the succeding paragraph.

3.1 Weirs

Weirs are structures usually located in open channel for the purpose of raising the upstream water levels, controlling the flow or for measuring discharge. The weirs can be given a variety of shapes, each suitable for a specific purpose. The three basic types (based on the profile), are the broad crested, sharp crested and ogee-crested weirs. The flow characteristics and relationship between upstream and downstream of the broad crested weir are described below. For flow characteristics and water level relationship for other two types, the reader should refer to standard text books.

The Broad Crested Weir

A weir is called broad crested, if the crest has a breadth (B) long enough in the direction of flow to maintain essentially parallel stream lines. The significance of this is that it ensures hydrostatic pressure distribution in a cross section over the weir. This condition could be achieved when $0.05 < H/B < 0.50$ for rounded leading edge and when $0.08 < H/B < 0.33$ for sharp leading edge. The longitudinal profile of a broad crested weir, with a

rounded leading edge is shown in figure 6.

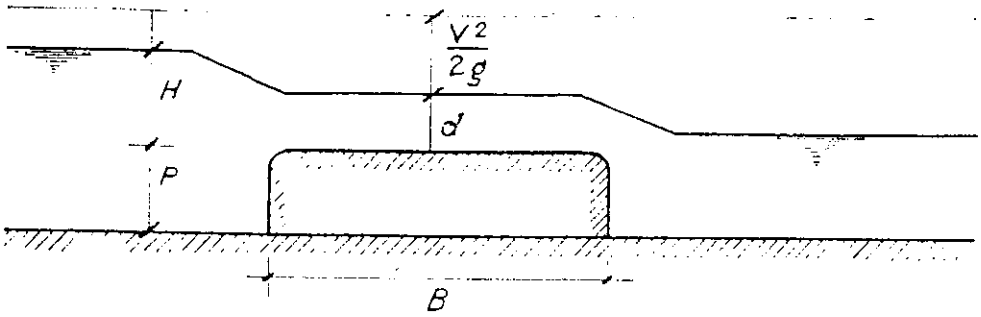


Fig. 6 : Defination sketch for a broad crested weir.

The discharge formula for the broad crested weir for idealized case of no friction can be derived from above figure.

$$\frac{V^2}{2g} = H - d \qquad \ldots\ldots (16)$$

$$q = \frac{Q}{L} = \frac{VdL}{L} d\,(2g)^{1/2}\,(H - d)^{1/2} \qquad \ldots\ldots (17)$$

Differentiating with respect to H it can be shown that the velocity on the crest becomes critical when the discharge reaches its maximum value. For free flow condition, the discharge is given by :

$$q = \frac{2}{3}\left(\frac{2}{3} g\right)^{1/2} H^{3/2} \qquad \ldots\ldots (18)$$

or $$q = 3.09\, H^{3/2} \qquad \ldots\ldots (19)$$

$$q = CH^{3/2} \qquad \ldots\ldots (20)$$

The above equation is theoretical because it assumes no friction along the crest. Thus to allow for friction,

Figures 7 and 7.1 can be used to obtain the discharge co-efficient "C" for various ratios of H/B. Equation (20) is valid for free flow condition. If the tailwater level rises to the point where the downstream head becomes 0.6 times the upstream head, the flow becomes submerged. The discharge co-efficient for the submerged flow condition can be determined using Figure 7.1.

3.2 Regulators

Regulators are gated structure which permit passage of water in and out of a particular area at a controlled rate. Flow characteristics through a regualtor is described below :

If an open channel with subcritical flow is constricted with either a reduction of width or a rise in the bed there will be reduction in the water surface elevation as illustrated in figure B. As the contraction becomes more and more severe for a given discharge, the water surface in

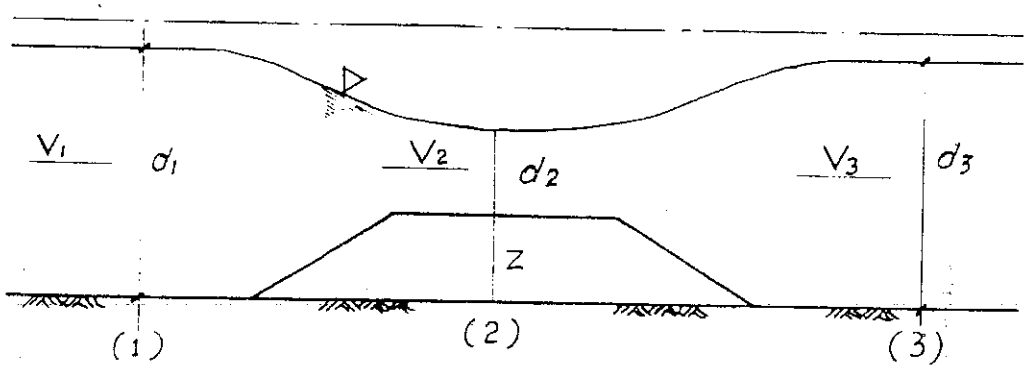


Fig. 8 : Definition sketch for flow in contraction.

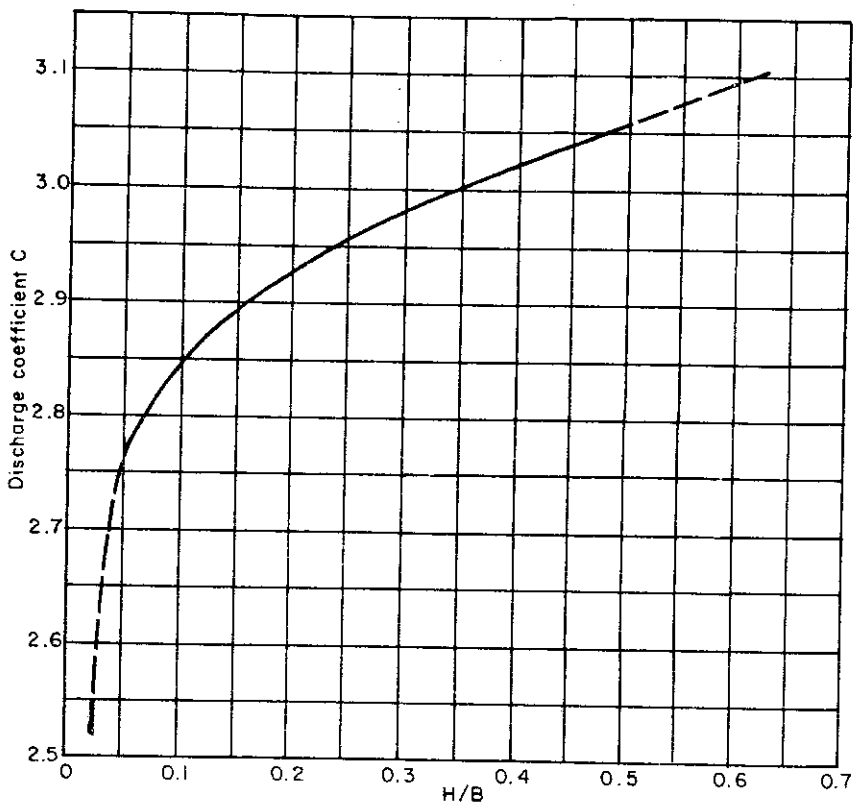
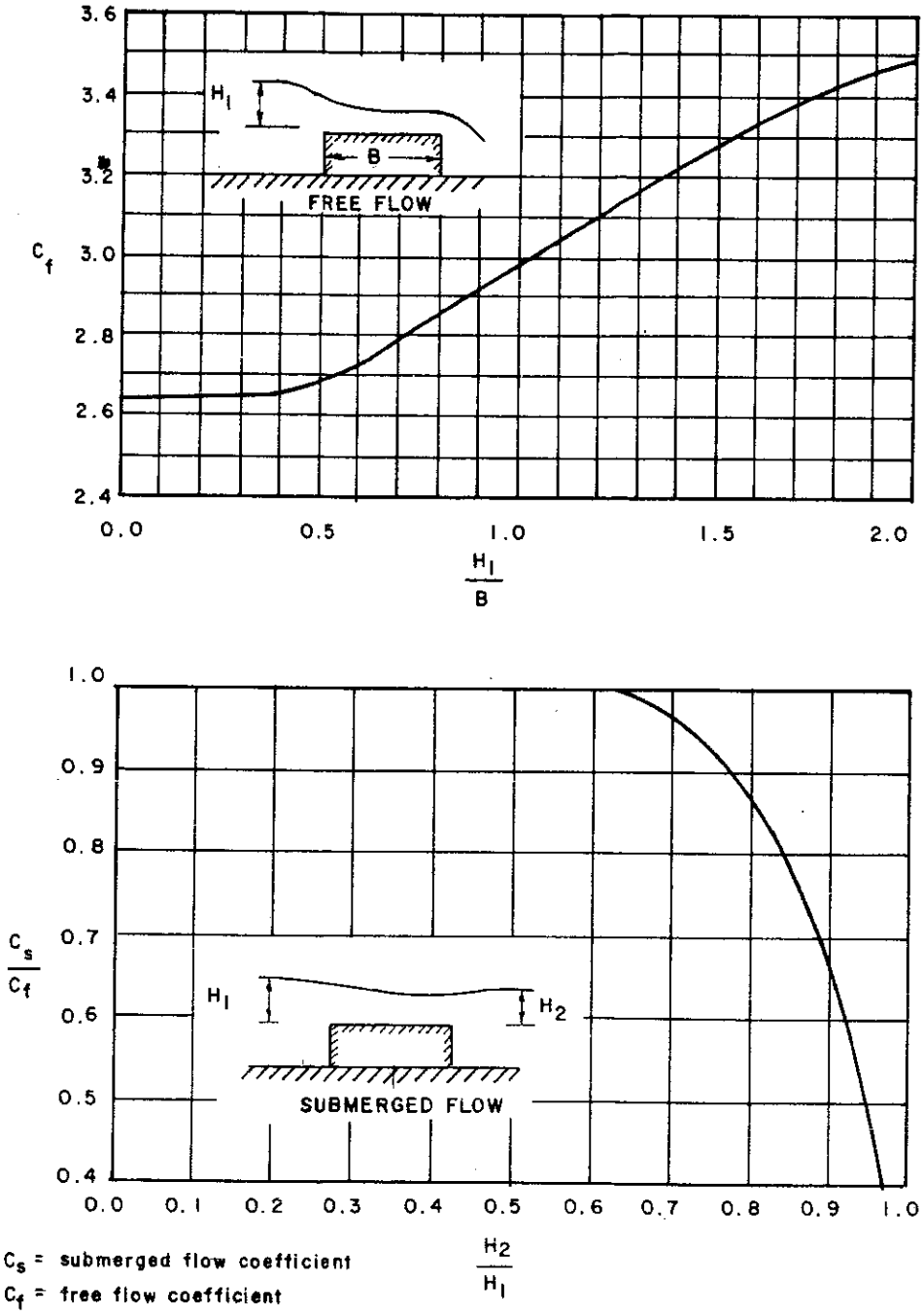


Figure 7. Coefficient of Discharge for Broad-Crested Weirs with Rounded Upstream Edge.



C_s = submerged flow coefficient

C_f = free flow coefficient

Source: Engineering Manual 1110-2-1602, U.S. Army Corps of Engineers, 1963, p.III-15

Figure 7.1 Coefficient of Discharge for Broad-Crested Weirs with Sharp Upstream Edge.

the constriction will drop until a stage is reached when further reduction will cause a rise in water level in both the constriction and the channel upstream.

The depth of water in the constriction is critical when the specific energy corresponding to that depth is minimum as illustrated in Figure 9. The equation relating to depth of flow and velocity is as follows :

$$\frac{V_1^2}{2g} + d_1 = \frac{V_2^2}{2g} + d_2 + Z \qquad \ldots\ldots (21)$$

At critical depth $\frac{V_2^2}{2g} = \frac{d_2}{2}$

Thus equation (21) becomes equation (22) for negligible losses :

$$\frac{V_1^2}{2g} + d_1 = \frac{3d_2}{2} + Z \qquad \ldots\ldots (22)$$

For conditions where the flow into, through and out of the constriction is sub-critical, the flow relationship is derived by starting from some known downstream water level and working in the upstream direction to compute water levels. The relationships used are :

$$Z_4 + d_4 + \frac{V_4^2}{2g} + H_{le} + H_{f_2} = Z_3 + d_3 + \frac{V_3^2}{2g} \qquad \ldots (23)$$

where the terms are given in Figure 10 and

$$H_{le} = C_e \left(\frac{V_3^2}{2g} - \frac{V_4^2}{2g} \right) \qquad \ldots\ldots (24)$$

and

H_{f_2} = energy loss due to friction

H_{le} = energy loss due to expansion

C_e = expansion co-efficient

$$Z_2 + d_2 + \frac{V_2^2}{2g} + H_{lc} + H_{f_1} = Z_1 + d_1 + \frac{V_1^2}{2g} \qquad \ldots\ldots (25)$$

and $H_{lc} = C_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right)$ energy loss due to contraction

and H_{f_1} = energy loss due to friction

C_c = contraction co-efficient

The terms are illustrated in Figure 10 for a constriction in width and Table 1 gives the coefficient for various transition types.

Table 1. : Transition Loss Co-efficients

Transition Type	Contraction-C_c	Expansion-C_e
Warped	0.10	0.20
Cylindrical Quadrant	0.15	0.25
Wedge	0.30	0.50
Straight Line	0.30	0.50
Square End	0.30	0.75

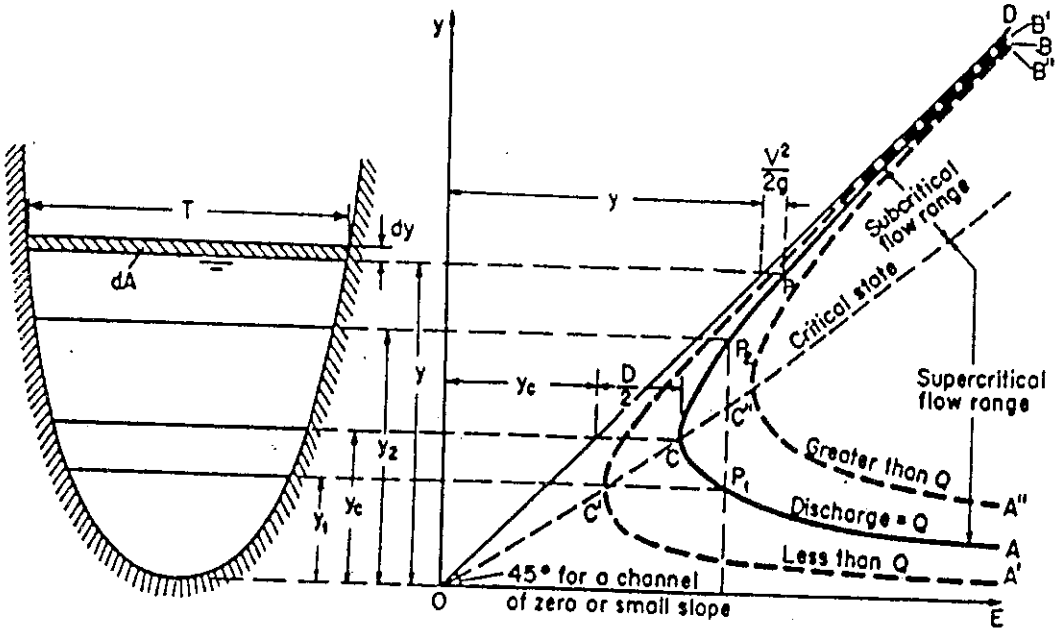
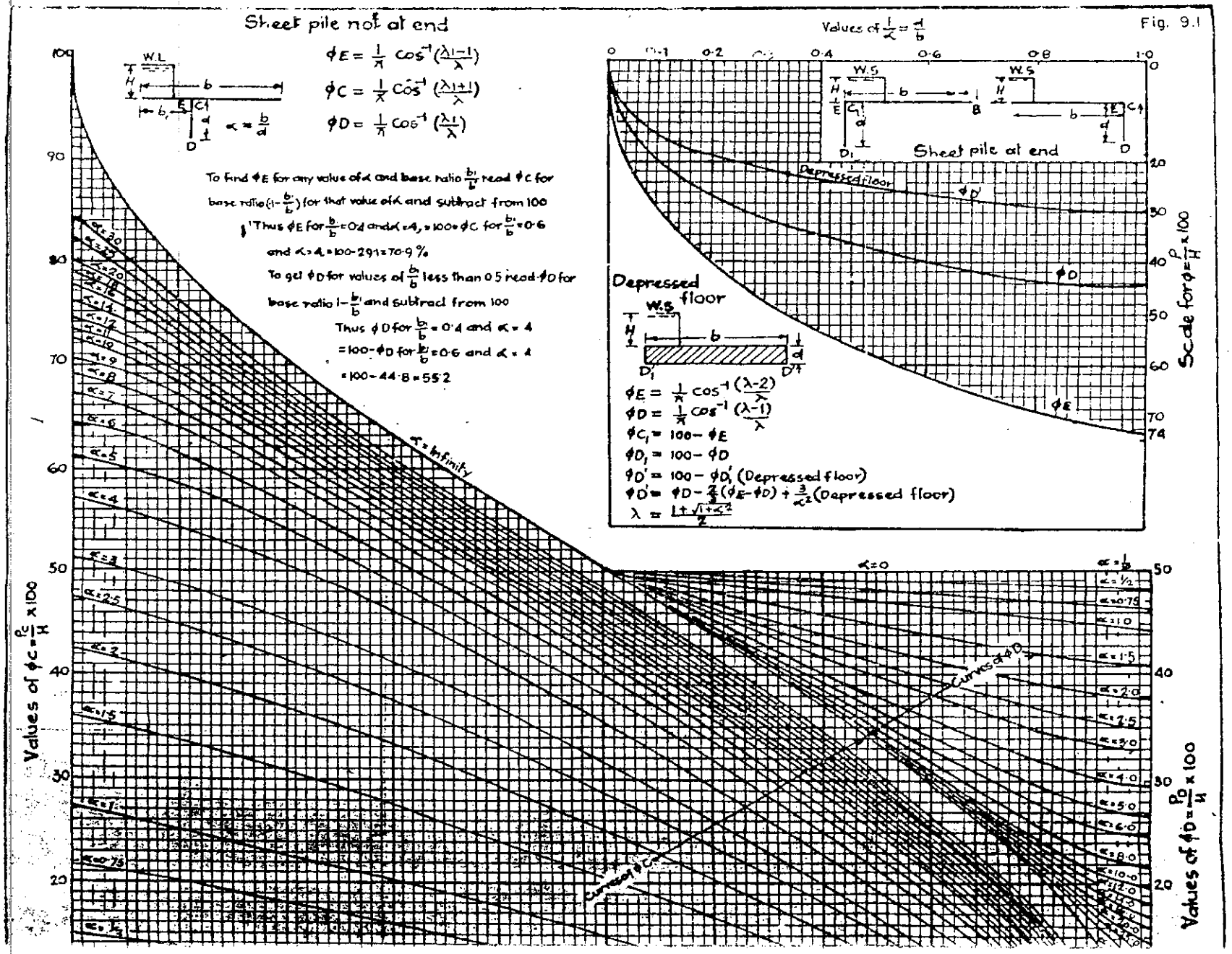


FIG. 9. Specific-energy curve.


Sheet pile not at end
Fig. 9.1
$\phi E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$
$\phi C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$
$\phi D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$
$\alpha = \frac{b}{d}$
To find ϕE for any value of α and base ratio $\frac{b_1}{b}$ read ϕC for base ratio $(1-\frac{b_1}{b})$ for that value of α and subtract from 100
Thus ϕE for $\frac{b_1}{b}=0.4$ and $\alpha=4$, $=100-\phi C$ for $\frac{b_1}{b}=0.6$ and $\alpha=4$, $=100-29.1=70.9\%$
To get ϕD for values of $\frac{b_1}{b}$ less than 0.5 read ϕD for base ratio $1-\frac{b_1}{b}$ and subtract from 100
Thus ϕD for $\frac{b_1}{b}=0.4$ and $\alpha=4$
$=100-\phi D$ for $\frac{b_1}{b}=0.6$ and $\alpha=4$
$=100-44.8=55.2$
Values of $\frac{1}{\alpha}=\frac{d}{b}$
Sheet pile at end
Depressed floor
$\phi D'$
ϕD
ϕE
Depressed floor
$\phi E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$
$\phi D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$
$\phi C_1 = 100 - \phi E$
$\phi D_1 = 100 - \phi D$
$\phi D' = 100 - \phi D_1'$ (Depressed floor)
$\phi D' = \phi D - \frac{2}{3}(\phi E - \phi D) + \frac{3}{\alpha^2}$ (Depressed floor)
$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$
Scale for $\phi = \frac{P}{H}\times 100$
Values of $\phi C = \frac{P_C}{H}\times 100$
Values of $\phi D = \frac{P_D}{H}\times 100$
α = infinity
$\alpha = 0$
Curves of ϕD
Curves of ϕC
$\alpha = \frac{b}{d}$

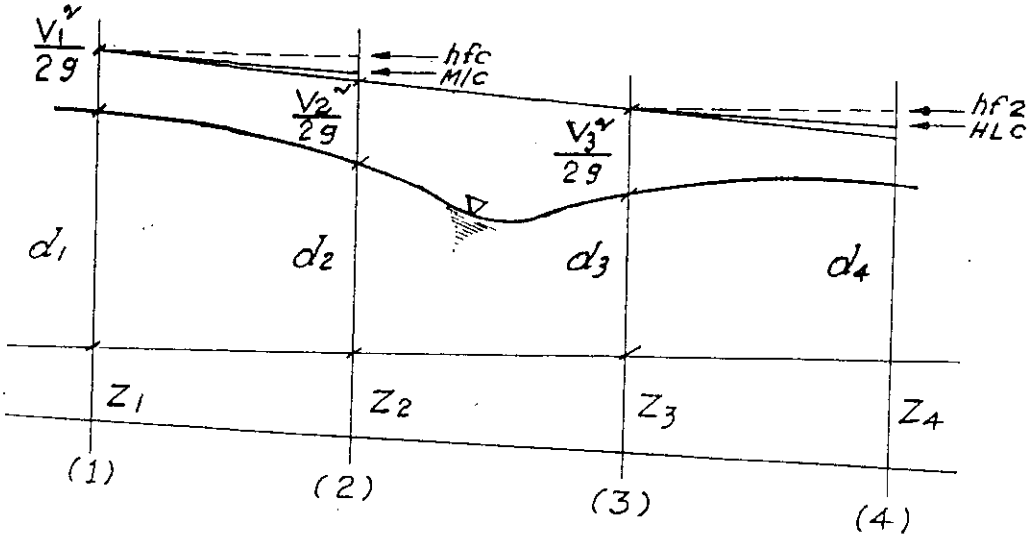


Fig. 10 : Flow depths at a constriction.

3.3 Sluices

A sluice is a structure provided with a gated opening to permit passage of water in one direction only when the head difference permit. These type of structures are provided with flap gate and are normally used in coastal region to prevent salinity intrusion and to permit drainage at low tide. Tide cycle is carefully considered in the design of these sluices.

3.4 Water Retention Structure

Water retention structure are designed to retain water for dry period irrigation and to check over drainage of beel areas. They are usually submergible low height structures (masonry or R.C.C.) and are designed as either sloping or vertical side walls. Vertical walls are preferred as they are more stable (Fig.11) against any possible erosion due to over flanking by flash floods in hill stream.

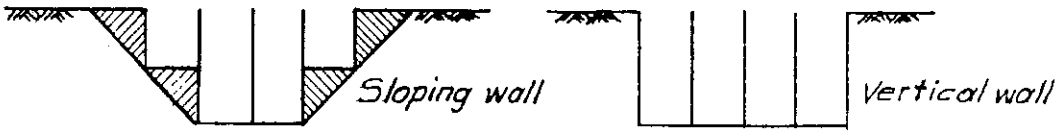


Fig. 11 : Water Retention Structure.

4.0 Design Procedures

The following design steps are usually considered in the hydraulic design of regulators, sluices, weirs and water retention structures.

- Energy dissipation and stilling bsin design
- Depth of scour and cut off wall
- Estimation of design head
- Exit gradient and floor length
- Uplift pressure and floor thickness
- Upstream and downstream

  Loose appron - Inverted filter.

4.1 Engergy Dissipation and Stilling Basin Design

The hydraulic jump was discussed in section 2.3 and its equations were derived. iI was shown that :

$$\frac{d_2}{d_1} = \frac{1}{2}\left[(1 + 8F_1^2)^{1/2} - 1\right] \qquad \ldots\ldots (26)$$

$$F_1 = V_1/(gd_1)^{1/2}$$

$$L_j = 6\, d_2 \qquad \ldots\ldots (27)$$

$$E = (d_2 - d_1)^3 / 4d_2 d_1 \quad \ldots\ldots (28$$

where d_2 = depth of flow after jump

d_1 = " " " before jump

L_j = Length of jump

and E = Loss of energy.

The hydraulic jump basins, most suitable for small hydraulic structues are the USBR (United States Bureau of Reclamation) basin type III and IV and the SAF (Saint Anthony Falls) stilling basins (Figures 12, 13 and 14).

It should be noted that the basin walls, in the USBR designs should be vertical and parallel. In the SAF design, the basin walls should be vertical but may be diverging.

The energy dissipation in the jump depends upon the froude number of the incoming flow. Different types of jumps have been described for different values of Froude numbers. If the Froude number of the incoming flow is higher, the greater will be the energy dissipation. The approximate percentage loss of energy for various values of F_1 are given in table 2.

Table 2 : Loss of energy for various values of F_1

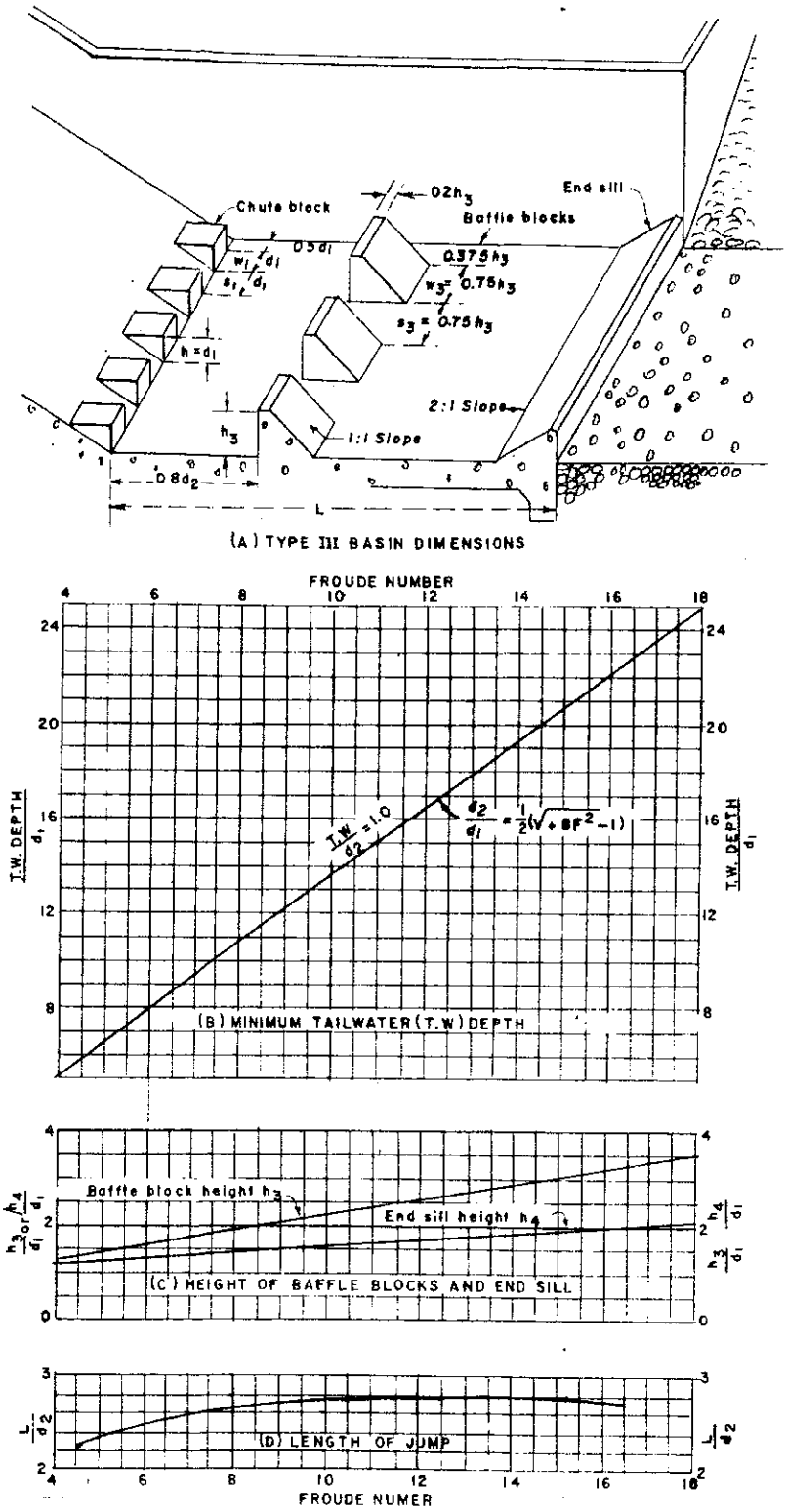
F_1	% loss of energy
2.5	17
4.5	45
9.0	70
14.0	80
20.0	85

The hydraulic jump basins are to be designed in such a way that the energy of exit velocity is to be dissipated to a tranquil state before the discharge is returned to downstream channel bed. USBR Stilling basin designs suitable to provide stilling action for various forms of jump are described here :

i) For F = 1, the flow is critical and hence no jump can occur.

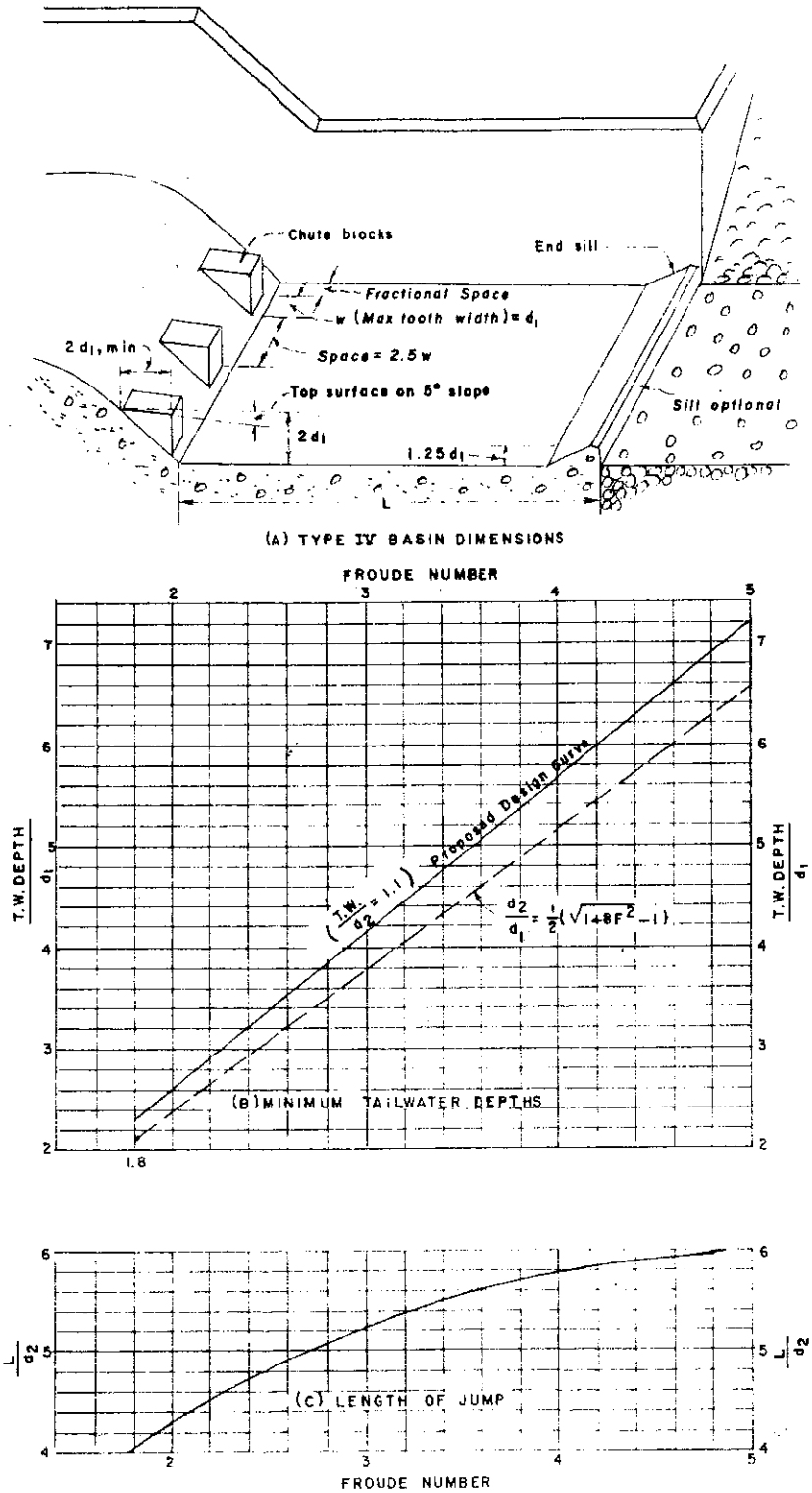
ii) For F = 1.7, the conjugate depth d_2 is about twice the incoming depth or about 40 percent greater than the critical depth. No special stilling basin is required to still flows except that the channel lengths beyond the point where the depth starts to change shall not be less than about $4d_2$. No baffles or other dissipating devices are needed. These basins are designated as type I.

iii) For F = 1.7 to 2.5. This is a prejump stage where flows are not attended by active turbulence and therefore



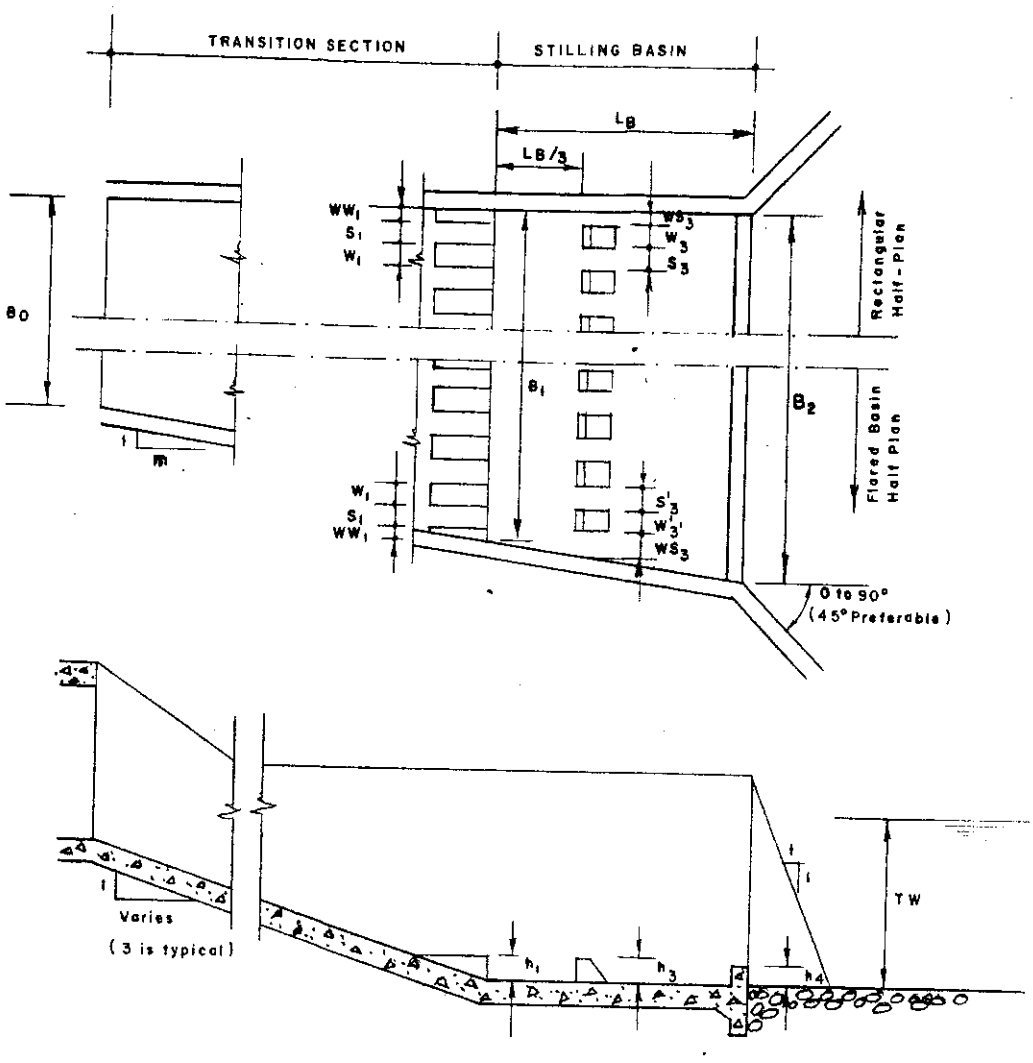
Source: "Design of Small Dams" by U.S. Department of the Interior

Figure 12. USBR Hydraulic Jump Stilling Basin Type III.



Source: "Design of Small Dams" by U.S. Department of the Interior

Figure 13. USBR Hydraulic Jump Stilling Basin Type IV.



Basin Floor: Upstream width, $B_1 = K\sqrt{Q}$ ($K = 0.6$ is commonly used)

Wall flare rate, $m = 3F_o$

Basin length, $L_B = 4.5\, d_2 / F_1^{0.76}$

Tailwater depth, $TW = (1.1 - F_1^2/120)\, d_2$ for $1.7 < F_1 < 5.5$

$TW = 0.85\, d_2$ for $5.5 < F_1 < 11$

$TW = (1 - F_1^2/800)\, d_2$ for $11 < F_1 < 17$

Chute Blocks: $h_1 = d_1$ $w_1 = s_1 = 0.75\, d_1$ $ww_1 > 0.375\, d_1$

Baffle Blocks: $h_3 = d_1$ $w_3 = s_3 = 0.75\, d_1$ $ws_3 > 0.375\, d_1$

$w_3' = s_3' = w_3 \left(1 + \frac{2m}{3B_1}\right)$

End sill: Height, $h_4 = 0.07\, d_2$

Figure 14. SAF Hydraulic Jump Stilling Basin.

baffles or sills are not required. The basin should be sufficiently long to contain the flow prism. Conjugate depths and basin lengths shown in figure 5 will provide acceptable basins.

iv) F = 2.5 to 4.5. This is a transition flow stage since a true hydraulic jump does not fully develop. The jump is trouble some and oscillating giving rise to heavy waves on the surface which persist beyond the end of the basin. Stilling device must be provided to dissipate flows for this range of froude number. The basin shown in figure 13 designated as type IV, has proved to be effective for dissipating the bulk of energy. The transition flow stage is normally avoided in the design.

v) F = 4.5 to 9.0. A true hydraulic jump will form in this range with the best jump performance. The installation of accessory devices such as blocks, baffles and sills along the floor of the basin produce a stabilizing effect on the jump, which allows for shortening the basin and provides a safety factor against sweepout due to inadequate tail water depth.

The basin illustrated in figure 12 can be adopted where incoming velocity does not exceed 50 ft/sec. This basin utilizes chute blocks, impact baffle blocks and an end sill to shorten the jump length and to dissipate the high velocity flow within the shortened basin length.

SAF (Saint Anthony Falls) stilling basins.

The SAF stilling basin was developed for canal and diversion structures on agricultural distribution systems and small streams having low heads and small unit discharges. SAF basin is so short that a significant amount of energy dissipation must occur downstream from the end sill. The downstream channel can be allowed to erode until a stable scour hole occurs or suitable protection can be provided downstream from the stilling basin to minimize scour. These basins should only be used where a certain amount of erosion is tolerable at the downstream channel. The range of froude number for SAF basin varies between $2 < F < 10$ where $F = \frac{V_1}{\sqrt{gD}}$. The details of SAF basin are illustrated in figure 14.

Low froude number stilling basin.

The development of a low froude basin design started with the basin type III design and modified according to the results of model tests. Data obtained du;ring these tests included basin length, upstream depth, tailwater depth and discharge. These variables were used to create the dimensionless plots for the design criteria (figures 15, 15.1 and 15.3. The recommended design is a relatively short stilling basin (L equals approx. $3d_2$) with chute blocks, baffle piers and dentated end sill (figure 15.2).

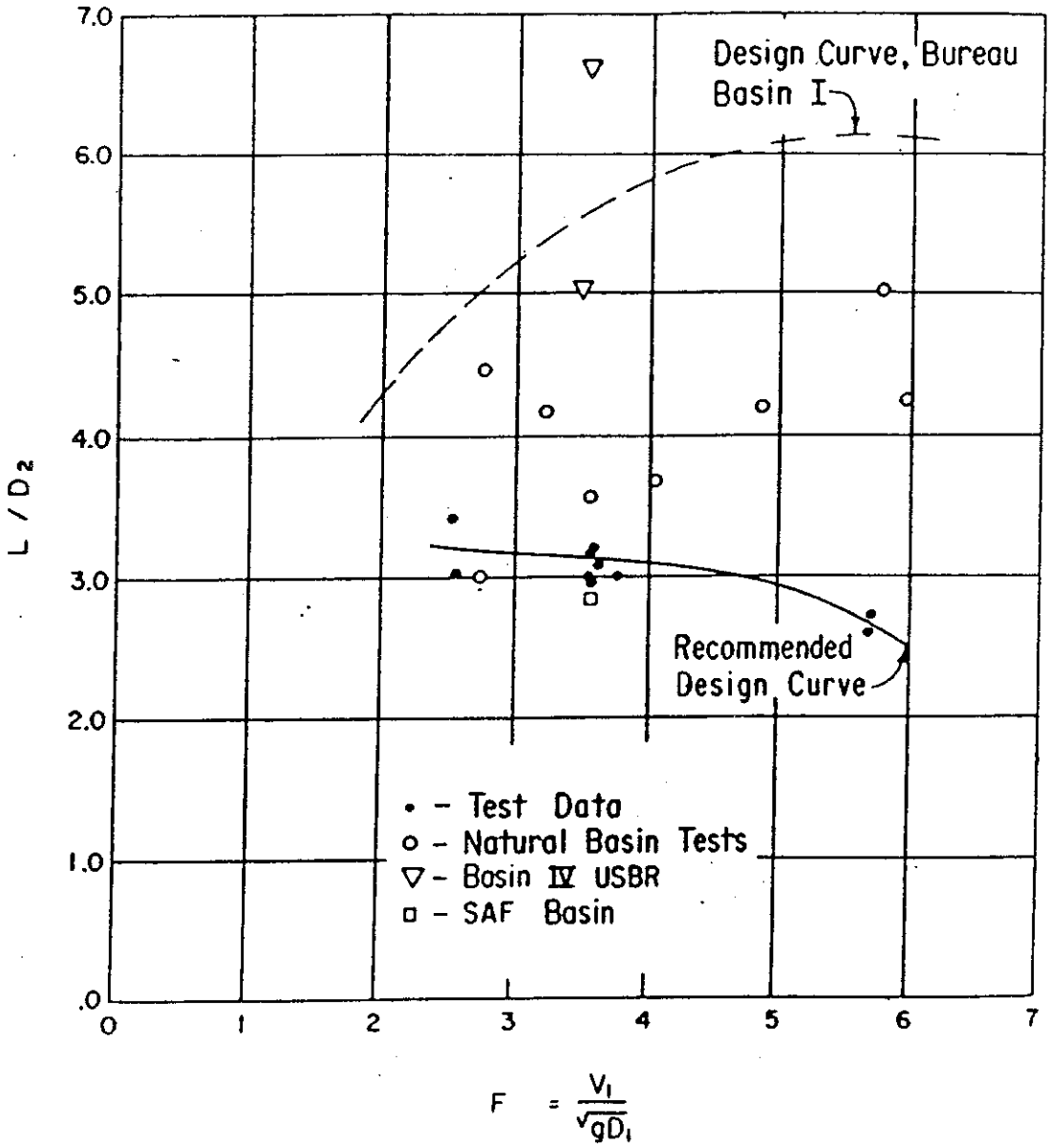


Figure 15. Dimensionless length of stilling basin.

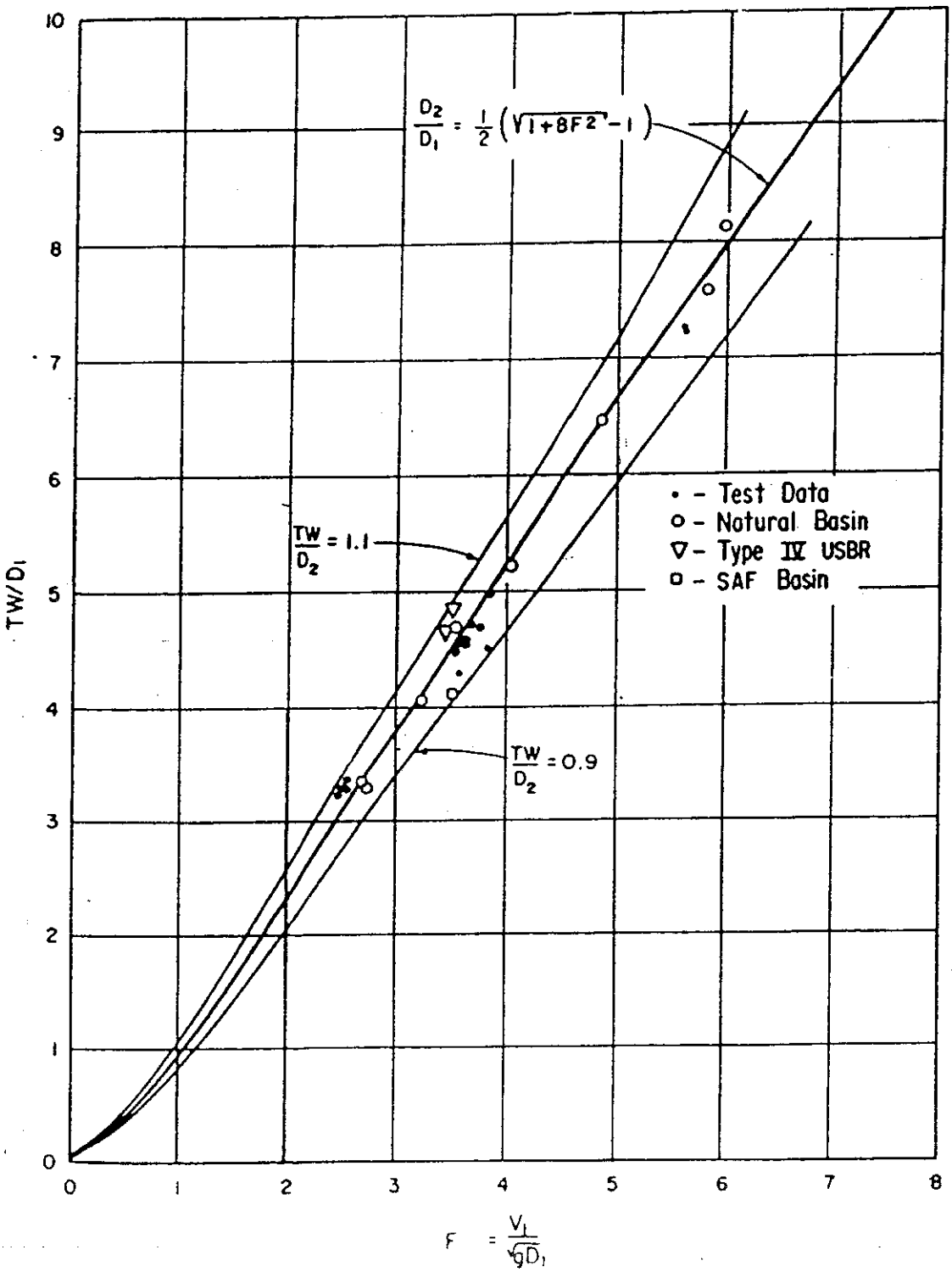
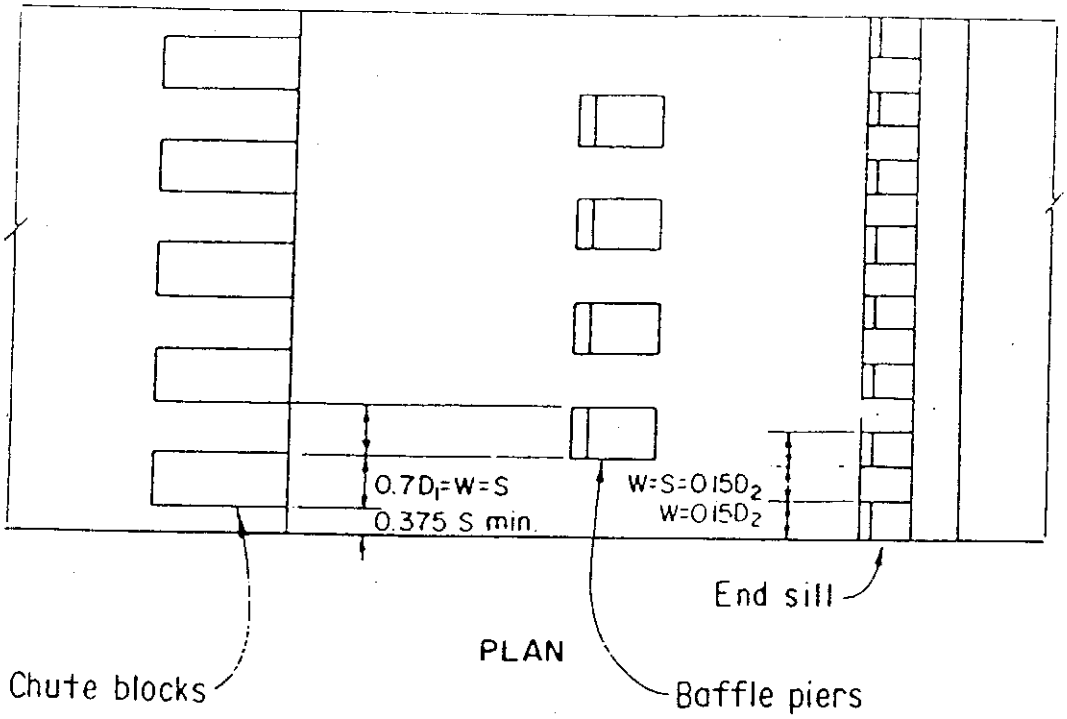
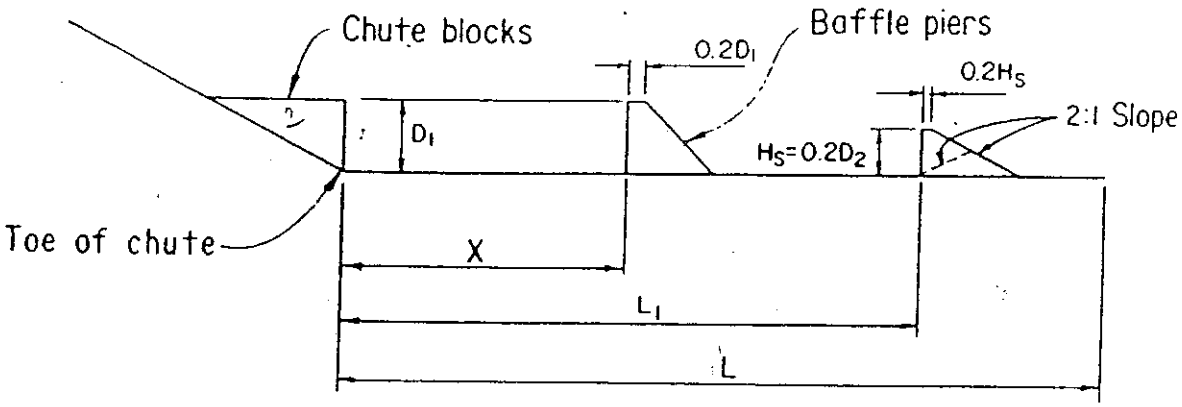


Figure 15.1. Ratio of tailwater to inflow depth.





ELEVATION

Figure-15.2 Recommended stilling basin ($2.5 \leq F \leq 5.0$).

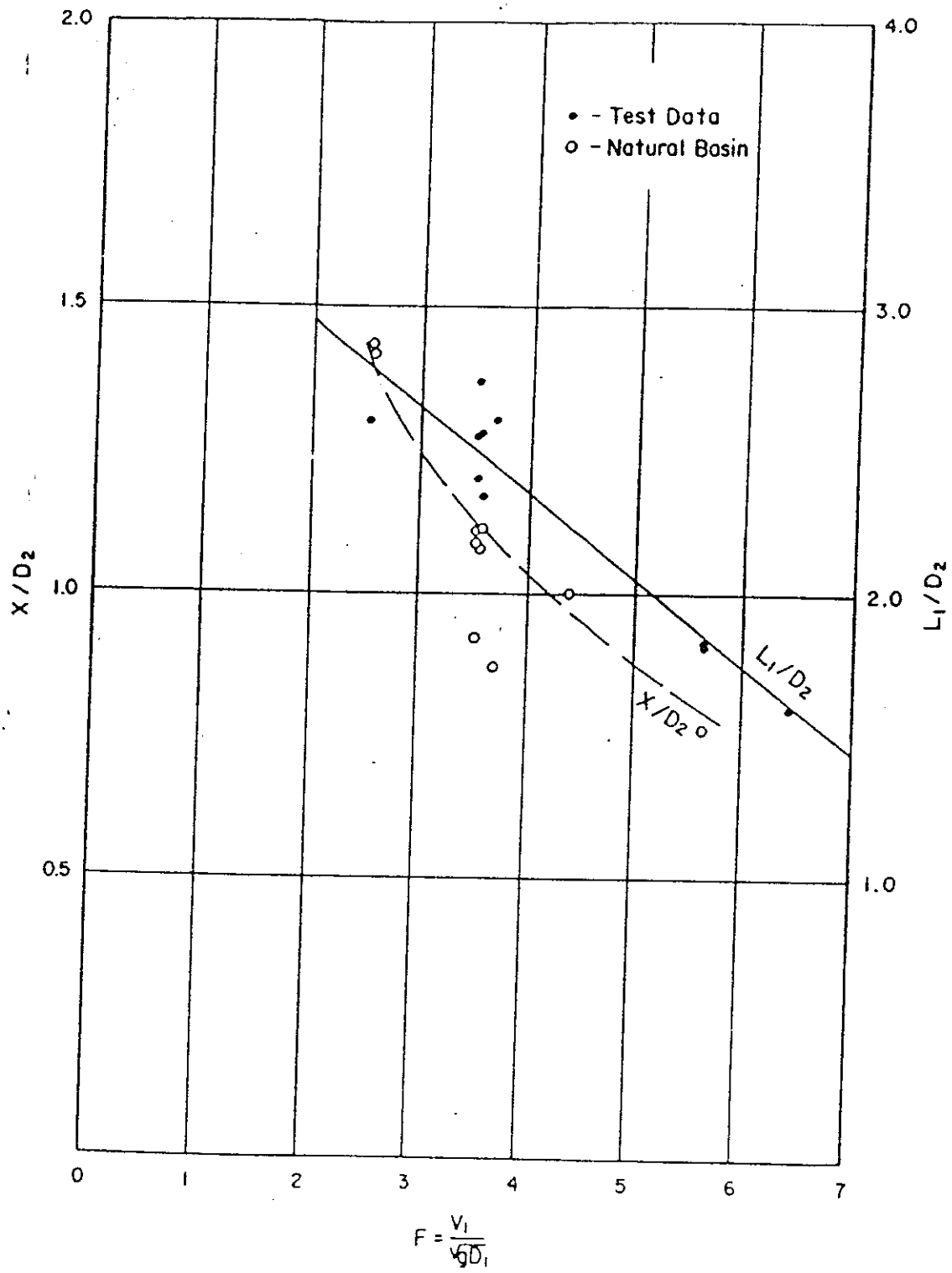


Figure 13-Distance from toe of chute to baffle piers (X) and end sill (L)

Chute Blocks and Baffle Piers.

The recommended height and width of both the baffle piers and the chute blocks are equal to D_1 and $0.70\ D_1$, respectively. The recommended spacing between these piers or blocks is equal to the width, that is $S = W = 0.70\ D_1$.

The following relationship can be used to obtain N (the total number of blocks and spaces) :

$$N = \frac{\text{Width} - 2kW}{W}$$

where k = fractional width of block equal to side clearance , $0.375 < k , 0.50$

Width = total width of stilling basin

$W = 0.70\ D_1$

The N obtained should be rounded to the nearest odd number and then adjusted values of either or both W and k can be computed.

The baffle piers should be placed in line with the openings between the chute blocks to increase their effectiveness and to decrease the waves. The clear space between the sidewall and the chute blocks should not be less than 0.375W nor greater than 0.50W. Usually, no baffle piers will be placed within 1.375W of the sidewall. However, if the blockage (summation of the widths of baffle piers divided by the width of the channel) is less than 0.40, then partial sections of the baffle piers could be placed along the sidewall to obtain approx. 0.50 blockage. Any configuration with less than four baffle

piers will beed partial baffle piers placed along the sidewalls to obtain the necessary blockage. Blockage should be kept between 0.45 to 0.55. When more than four bafffle piers were tested, no difference in performance was noted when partial side piers were in place, thus the partial side piers would usually not be needed.

The location of the chute blocks and baffle piers is shown in figure 15.2. The distance from the chute blocks to the baffle piers (X in figure 15.2) varies from 1.3 to about 0.7 times D_2 as the froude number varies from 2.5 to 5.6.

End Sill and Basin Length.

The dimension L_1 (the distance from the toe of the chute to the upstream side of the end sill) may be obtained from figure 15.3. L plus the length of the end sill is somewhat shorter than L for Froude numbers greater than 3 and almost equal to L for Froude numbers less than 3. The additional length beyond the end sill was required for acceleration and deceleration of the flow. However, the distance L plus the length of the end sill might be slightly longer than L for froude numbers less than 2.7. The stilling basin must be extended to include the end sill for the latter.

The end sill is approximately 0.2 D_2 high. The width of the dentates and spacing between the dentates are both equal to approximately 0.15 D_2. This width may be adjusted to obtain an integer number of dentates across

the end sill. Generally, dentates should be placed against the sidewalls of the stilling basin.

The recommended L shown as the ratio L/D_2, (figure 15) is very close to the value 3, for all froude numbers shown and could be used instead of the value obtained from the curve. Suitable scour protection will generally be needed downstream from the end of the stilling basin in highly erodible channels.

Tailwater Depth.

A tailwater depth of D_2 maintained the jump at the intersection of the horizontal apron and the chute. However, sweepout did not occur for the recommended design when the tailwater was 0.8 of D_2. The TW (tailwater depth) should be maintained at or slightly higher then D (five percent or less). The additional depth increases the factor of safety against sweepout and decreases the flow velocity.

4.2 Depth of scour and cut off wall

The depth of scour is usually determined by Lacey's regime scour depth

$$R = 0.91 \left(\frac{q^2}{f}\right)^{1/3} \qquad \ldots\ldots (29)$$

where q = discharge per unit width in ft^3/sec

f = Lacey's silt factor

$$= 1.76 \, (dmm)^{1/2} \qquad \ldots\ldots (30)$$

dmm = Average particle size in mm of the material forming the channel bed.

The total depth of scour below HFL is taken as XR where R is Lacey's normal scour depth and values of X for different classes of scour are shown in Table 3 below.

Table 3 : Lacey's safety factor against scour depth.

Class of scour	Reach	Mean value of X	D = XR - Water depth above bed
A	Straight	1.25	1.25R-Y
B	Moderate bend	1.50	1.50R-Y
C	Severe bend	1.75	1.75R-Y
D	Bend	2.00	2.00R-Y

A value of 1.25R on upstream and 1.50R on downstream is widely accepted as a via media. The depth of cut off wall is calculated from scour depth and later checked from exit gradient consideration.

4.3 Estimation of Design Head

Estimation of design head is very important in calculating the uplift pressure and floor thickness downstream. The following criteria should be considered in estimating design head :

Regulators

In determining design head for regulators two conditions should be considered :

- Pre monsoon
- Post monsoon

Pre monsoon :

. Maximum c/s water level (from pre monsoon routing)

. Minimum r/s water level (usually considered dry bed)

Post monsoon :

. C/S storage level (basin storage level from post monsoon routing)

. R/S water level (from river hydrograph)

Water Retention Structure.

. Upstream retention level & downstream minimum water level

Sluices (coastal)

. River side high tide level and country side minimum water level

. Country side retention level and riverside low tide level

4.4 Exit Gradient and Floor Length

According to Khosla's theory, for a standard form consisting of floor length b with a vertical cut off depth d (Fig. 16), the exit gradient at its downstream end is given by

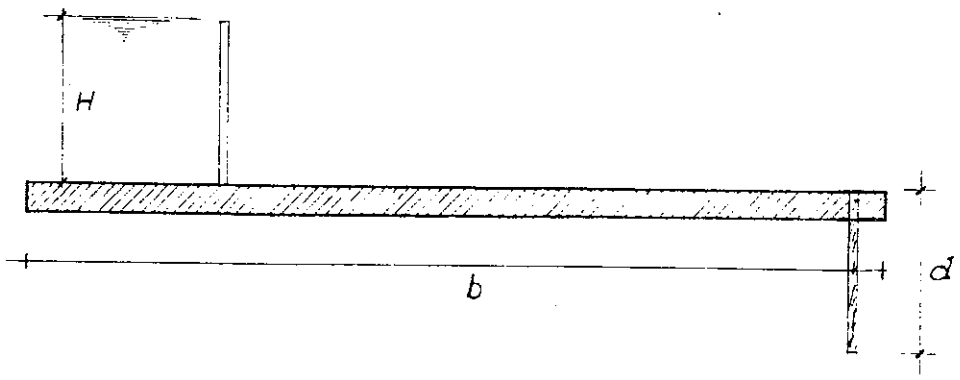


Fig. 16 : Khosla's standard form.

$$GE = \frac{H}{d} \frac{1}{\pi \sqrt{L}} \qquad \ldots\ldots (31)$$

$$\text{where} \quad = \frac{1 + \sqrt{1 + a^2}}{2} \qquad \ldots\ldots (32)$$

$$\text{and} \quad a = \frac{b}{d} \qquad \ldots\ldots (33)$$

H is maximum seepage head.

The exit gradient is said to be critical when the upward disturbing force developed by difference in pressure head on the material grain is just equal to the submerged weight of the grain at the exit. For the structure to be safe against piping, an exit gradient equal to 1/6 to 1/7 of the

critical exit graadient should be ensured. Value of Khosla's safe exit gradient for different type of material is shown in Table 4 below :

Table 4 : Khosla's safe exit gradient.

Type of material	Khosla's safe exit gradient
Shingle	0.25 to 0.20 (1/4 to 1/5)
Coarse sand	0.20 to 0.17 (1/5 to 1/6)
Fine sand	0.17 to 0.14 (1/6 to 1/7)

Bligh's creep theory and Lane's weighted creep theory are also used to check safe exit gradient but Khosla's theory is the conservative one.

i) <u>Bligh's creep theory</u>

According to Bligh's theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the length of the creep. Further it is assumed in this theory that the loss of head is proportional to the length of the creep. If H_L is the total head loss between the upstream adn downstream, and L is the length of the creep, loss of head per unit of creep length (i.e. H_L/L) is called the hydraulic gradient.

Bligh's theory did not consider any distinction between horizontal and vertical creep.

Consider a section shown in Fig. 17 below where H_L

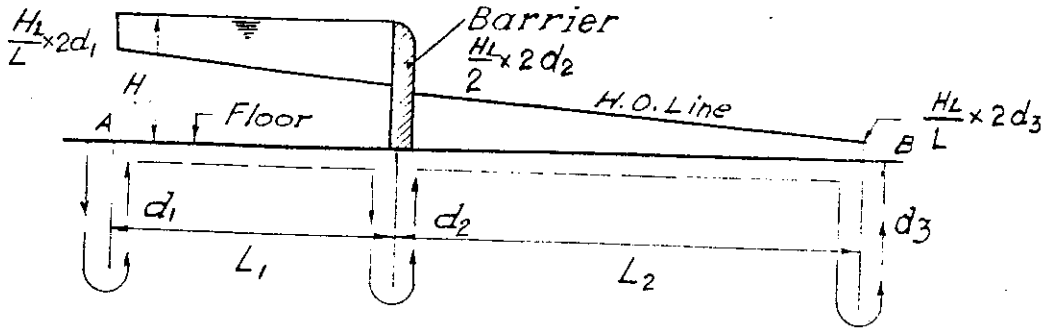


Figure 17 : Bligh's Creep theory.

is the difference of water levels between upstream and downstream ends (downstream dry bed). Water will seep along the bottom contour as shown and the total length of Creep is

$$L = d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3$$

$$= 2d_1 + L_1 + 2d_2 + L_2 + 2d_3$$

$$= (L_1 + L_2) + 2(d_1 + d_2 + d_3) \qquad \ldots\ldots (34)$$

Hydraulic gradient

$$= \frac{H_L}{b + 2(d_1 + d_2 + d_3)} = \frac{H_L}{L}$$

Head Losses equal to

$$\frac{H_L}{L} \times 2d_1 , \quad \frac{H_L}{L} \times 2d_2 \quad \text{and} \quad \frac{H_L}{L} \times 2d_3 \quad \ldots . (35)$$

will occur respectively in the planes of three vertical cut offs. The spacing between two adjacent cutoff shall be not less than twice the depth of cut off.

Safety against piping

Safety against piping can be ensured by providing sufficient creep length $L = CH_L$ where C is Bligh's creep co-efficient for soil. Different values of C for different types of soil are given in table 5 below

Table 5 : Bligh's creep co-efficient.

Sl.No.	Soil type	Value of C	Safe Hydraulic gradient should be less than
1	Fine sand	15	1/15
2	Coarse sand	12	1/12
3	Sand mixed with boulder and gravel	5 to 9	1/5 to 1/9
4	Light sand and mud	8	1/8

ii) **Lane's weighted Creep theory**

Bligh made no distinction between horizontal and vertical creep as stated above but Lane based his analysis on the study carried out on about 200 dams all over the world. He stipulated that the horizontal creep is less effective in

reducing uplift (or in causing loss of head) than that of the vertical creep. He, therefore, suggested a weightage factor of 1/3 for the horizontal creep as against 1.0 for the vertical creep. Refering to figure 17 the total Lane's creep length.

$$Le = (d_1 + d_2) + 1/3\ (L_1 + L_2) + (d_2 + d_2) + (d_3 + d_3)$$

$$= 1/3\ (L_1 + L_2) + 2\ (d_1 + d_2 + d_3)$$

$$= 1/3\ b + 2\ (d_1 + d_2 + d_3) \qquad \ldots\ldots (36)$$

To ensure safety against piping, the creep length Le must not be less than CH where H is the head causing flow and C is Lane's Creep co-efficient given in table 6.

Table 6 : Lane's creep co-efficient.

Sl. No.	Soil type	Values of Lane's co-efficient 'C'	Safe Lane's Hyd. gradient should be less than
1	Very fine sand or silt	8.50	1/8.5
2	Fine sand	7.0	1/7
3	Coarse sand	5.0	1/5
4	Gravel and sand	3.50 to 3.0	1/3.50 to 1/3
5	Boulders,gravel	2.50 to 3.0	1/2.5 to 1/3
6	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

4.5 Uplift pressure and floor thickness

In order to know as to how the seepage below the foundation of a hydraulic structure is taking place, flow net should be plotted. This can be solved mathematically or by graphical sketching by adjusting the stream lines and

equipotential lines with respect to boundary condition. These are complicated approach and are time consuming. Khosla has evolved a simple, quick and accurate approach, called Method of Independent Variables to design hydraulic structures on pervious foundation. In this method, complex profile like a regulator or a weir is broken into a number of simple profiles. Mathematical solutions of flow nets for these profiles are presented in the form of curves (see fig. 16) which can be used for determining percentage pressure at various key points under the structure. The simple profiles are shown in Fig. 18 below :

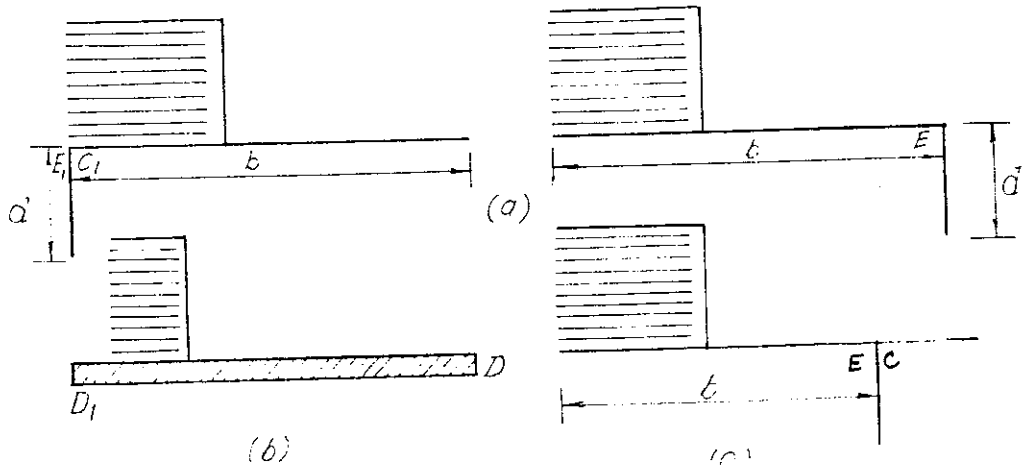


Fig. 18 : Khosla's simple profiles.

a) A straight horizontal floor of negligible thickness with a sheet pile on the upstream end or downstream end.

b) A straight horizontal floor depressed below the bed but without any vertical cutoffs.

c) A straight horizontal floor of negligible thickness with a sheetpile line at some intermediate points.

The key points are the junctions of the floor and pile lines on either side and the bottom points of the pile line and the bottom corners in the case of depressed floor. The percentage pressures at various key points is valid for the complex profiles if corrected for

(i) Mutual interference of pile

(ii) Thickness of floor

(iii Slope of the floor

(i) <u>Correction for mutual interference of piles</u>

$$C = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d+D}{b}\right]^{1/2} \quad \ldots\ldots (37)$$

where c = correction (percentage of head)

b' = distance between two piles

D = depth of pile line, the influence of which has to be determined on the neighbouring pile of depth d. D is to be measured below the level at which interference is desired.

d = depth of pile on which effect is considered.

b = total length of floor

This correction is +ve for points in the rear or backwater and -ve for the points foward in the direction of flow. This equation does not apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is at a distance less than twice the length of the outer pile.

(ii) <u>Correction for floor thickness</u>

Khosla's standard profile assumed to have negligible thickness and percentage pressure calculated shall pertain

to top levels of the floor while the junction points E and C are at the bottom of the floor (Fig. 19). Since the corrected pressure at E_1 should be less than the

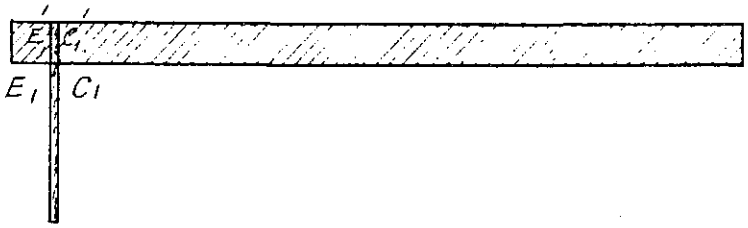


Fig. 19 : Definitation sketch for correction to floor thickness.

calculated pressure at $E_1^{/}$, the correction to be applied for the point E_1 shall be -ve. Similarly pressure calculated at $C_1^{/}$ is less than the corrected pressure at C_1 and hence, the correction to be applied at C_1 is +ve. The pressures are found out by assuming linear pressure distribution.

(iii) <u>Correction for slope of floor:</u>

A correction is applied for a sloping floor and is taken as +ve for the down and -ve for the upslopes following the direction of flow. Values of correction for standard slopes are given in table 7.

Table 7 : Slope correction factor.

Slope Horizontal : Vertical	Correction factor
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8

The correction factor given above is to be multiplied by the horizontal length of the slope and divided by distance between two pile lines between which the slopping floor is located. This correction is only applicable to the key points of the pile line fixed at the beginning or the ends of the slope.

4.6 <u>Loose protective work - inverted filter</u>

Protective works are required on the upstream and downstream of the structure in order to obviate the possibility of scour hole travelling close to the pucca floor of the regulator/sluice and to relieve any residual uplift pressure through the filter. The arrangement consists of :

i) Inverted filter and (ii) Launching apron

(a) <u>Downstream Loose Protection</u>

(i) <u>Inverted filter</u>

After the end of concrete floor, an "inverted filter", 1.5 to 2D long is generally provided, where D is the depth of

scour below original river bed. An inverted filter invariably reduces the possibility of piping as it allows free flow of seepage water through itself without allowing foundation soils to be lifted upward. The filter, therefore, consists of layers of materials with increasing permeability from bottom to top. The gradation should be such that while it allows free flow of seepage water the foundation material does not penetrate or clog the filter. To prevent filter from dislocation under surface flow, concrete or masonry blocks are laid over the filtered material.

ii) **Launching apron**

After the inverted filter, the loose apron called launching apron is provided for a length, generally equal to 1.5D, where D has the same meaning as given above in inverted filter calculations. (Fig. 20)

The apron generally launches to a slope of 2:1, and if t is the thickness of the apron in the launched position, length being $(5)^{1/2}$ D; the volume of stone per meter length

will then be $V = (5)^{1/2} D.t$

$= 2.24\ D.t$ or say $2.25\ D.t$

Hence, Volume of stone per metre is given by 2.25 D.t.cu.m.

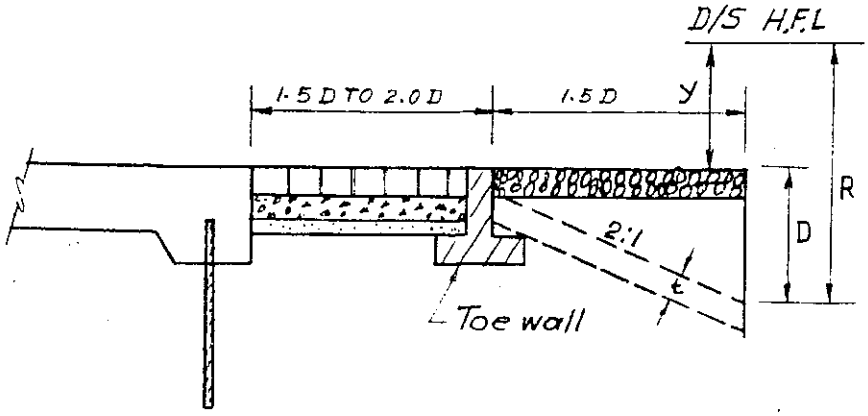


Fig. 20 : Details of loose protection.

(b) <u>Upstream Loose Protections</u>

Just upstream of concrete floor of the regulator/sluice, block protection is provided. It generally consists of brick blocks laid over packed stone, for a length equal to D (D=xR-y, where x=1.0 to 1.5, generally taken as 1.25), R is Lacy's normal scour depth and y is the depth of water above bed.

Upstream of the blocks, launching apron is provided in the same way as described for the downstream portion, except that the proper value of x should be chosen.

Toe walls are always generally constructed in between the 'filter' and the 'apron'.

5.0 Design Example.

5.1 Hydraulic Design of Mrigi Khal Drainage Regulator (3-5'x6' RCB)

The detailed hydrological analysis for this regulator has been given in preceeding Chapter II. Number of vents has been decided to be 3-5'x6' with a design discharge of 565 cfs. The step by step computations required for hydraulic design are given below :

- Stilling basin Design

Q = 565 cfs

No. of vents = 3

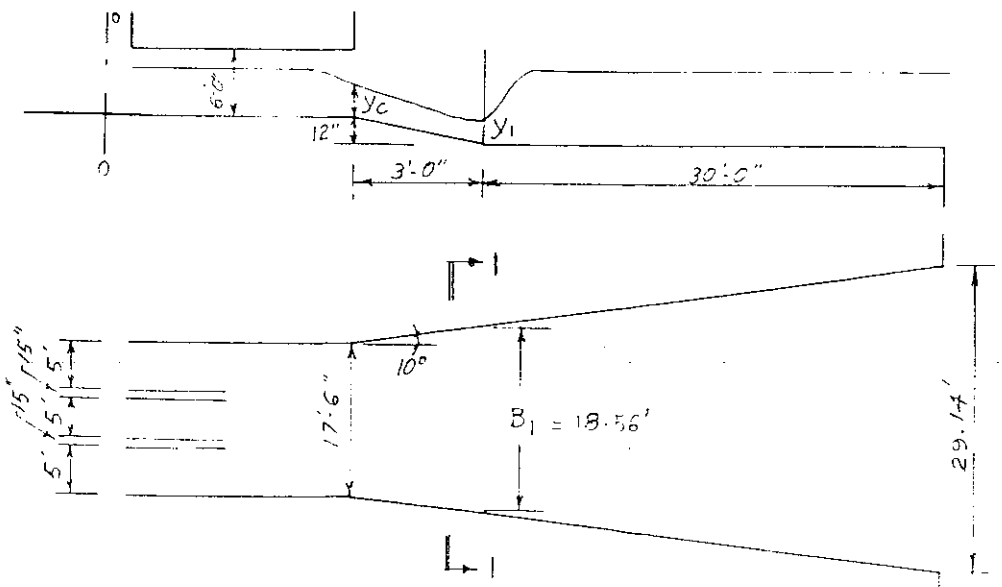
Size of vents : width = 5'-0" ; height = 6'-0"

Using pier width 1'-3"

Flow width = 3 x 5 + 2 x 1.75 = 17.5'

$$\text{Flow per ft. width, } q = \frac{565}{17.5} = 32.29 \text{ cfs/ft.}$$

$$\text{Critical depth, } Yc = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{32.29^2}{32.2}\right)^{1/3} = 3.19'$$



let the apron floor be depressed by 12"

Slope of glacis = 3:1

Flaring angle $= 10^{o}$

Flow width at the toe of glacis $B_1 = 17.5 + 2(3 \tan 10^{o}) = 18.56'$

$q_1 = Q/B = 565/18.56 = 30.44$ cfs.

$V_1 = q_1 / Y_1 = 30.44/Y_1$

Applying Bernoulli's Equation between critical section and sec.1-1.

$$Y_c + \frac{V_c^2}{2g} + z = Y_1 + \frac{V_1^2}{2g}$$

$$Y_c + 0.5\, Y_c + z = Y_1 + \frac{V_1^2}{2g}$$

$$\text{or} \quad 1.5\, Y_c + z = Y_1 + \frac{V_1^2}{2g}$$

$$\text{or} \quad 1.5 \times 3.19 + 1 = \frac{30.44^2}{Y_1^2 \times 64.4} + Y_1$$

$$\text{or} \quad 5.785 = \frac{14.4}{Y_1^2} + Y_1$$

By trial & error

$Y_1 = 1.93$

$V_1 = 30.44/1.93 = 15.78$ fps.

$$F_1 = V_1/(gy)^{0.5} = 15.78/(32.2 \times 1.93)^{0.5} = 2.0 < 2.5 \text{ O.K.}$$

$$Y_2 = \frac{Y_1}{2}\left[(1 + 8F_1^2)^{0.5} - 1\right] = \frac{1.93}{2}\left[(1 + 8 \times 2^2)^{0.5} - \right] = 4.58$$

From curve $\frac{L}{Y_2} = 4.35$ (see fig. 5)

$L = 4.58 \times 4.35 = 19.92$

With factor of safety 1.5

Length of apron $= 19.92 \times 1.5 + 3 = 33$ ft.

Checking design of basin from energy relation

Equation between section (0 - 0) adn (1 - 1), $E_o = E_1$

$$Y_o + \frac{V_o^2}{2g} + Z = Y_1 + \frac{V_1^2}{2g}$$

Neglecting entrance loss and loss through conduit

$Y_o = 19.8 - 14.0 = 5.8'$

$Z = 1'\text{-}0''$ (assumed)

$$\text{therefore} \quad 5.80 + 1 = Y_1 + \frac{(30.44)^2}{64.4\, Y_1^2}$$

$$\text{or} \quad 6.8 = Y_1 + 14.39/Y_1^2$$

By trial $Y_1 = 1.66$ ft.

$V_1 = 30.44/1.66$

$= 18.34$ ft/sec.

$F_1 = 18.34/\sqrt{32.2 \times 1.66}$

$= 2.50$

$$Y_2 = \frac{Y_1}{2}\left[\sqrt{(1 + 8F_1^2} - 1)\right]$$

$$= \frac{1.66}{2}\left(\sqrt{1 + 8 \times 2.5^2} - 1\right)$$

$= 5.09$ ft.

Level of $Y_2 = (14 - 1 + 5.09) = 18.09'$ which is greater then tail water level and the jump will be swept out.

2nd trial witht 2'-0" drop on River side

$B_1 = 17.5 + 2\,(6 \tan 10^{o}) = 19.62$ ft.

$q_1 = 565/19.62$

$= 28.80$ cfs.

$$V_1 = \frac{q_1}{Y_1} = 28.8/Y_1$$

therefore, $$5.8 + 2 = Y_1 + \frac{(28.8)^2}{64.4\,Y_1^2}$$

$$7.8 = Y_1 + 12.88/Y_1^2$$

By trial $Y_1 = 1.42$ ft $V_1 = 20.28$ ft/sec

$F_1 = 3$

$Y_2 = 5.36$ ft.

Level of $Y_2 = (14 - 2) + 5.36 = 17.36 \approx 17.00$

The jump matches the tail water level and therefore there would be no sweep out.

From curve (Fig. 5) $\frac{L}{d_2} = 5.30$

$L = 5.30 \times 5.36$

$= 28.40$ ft.

Therefore length of apron $= 28.40 + 6$

35'-0"

- **Computation of Cutoff Depth.**

Width at the end of D/S floor

$= 17.5 + 2\,(33 \tan 10^{o}) = 29.14$ ft.

q/ft at the end of D/S floor

$= \frac{565}{29.14} = 19.39$ cfs.

Depth of scour

$$R = 0.91 \left(\frac{q^2}{f} \right)^{1/3}$$

$$= 0.91 \left(\frac{19.32^2}{0.21} \right)^{1/3} = 11.05 \text{ ft.}$$

D/S scour depth with factor of safety 1.5 from D/S water level = 16.57 ft.

Elevation of the bottom of D/S cutoff wall

= (13 + 4.58) - 16.57 = + 1.01 say E.L. + 1.00

Provide 10 ft. cutoff wall at the U/S with factor of Safety 1.25 = EL + 3.0

- **Estimation of Design Head.**

Pre-monsoon

Maxm C/S water level = + 19.8 (From pre-monsoon routing).

Minm R/S water level = + 14 (Considering D/S dry)

Design Head for designing D/S floor = 5.8' say = 6'-0".

Post-monsson

C/S water level = 25.57 (Basin storage level from post monsoon routing)

R/S water level = + 32.6 (From the hydrograph)

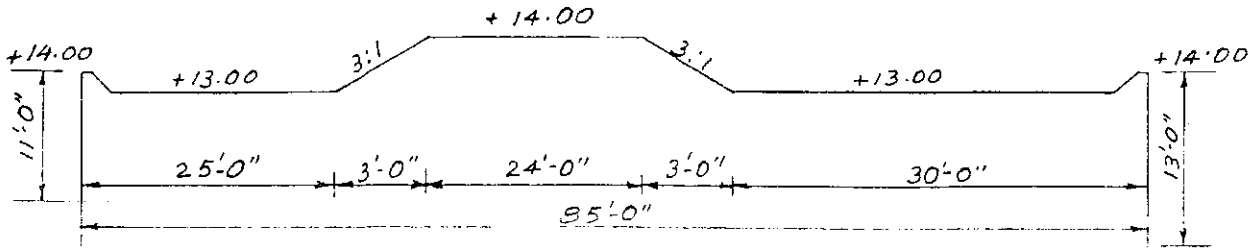
Design Head for designing U/S floor = 7.03' say = 7'-0".

- **Computation for exit Gradient.**

Maxm seepage head, H = 7'-0"

Depth of U/S cutoff d = 10'-0"

Total floor length b = 85'-0"



$$a = \frac{b}{d} = \frac{85}{11} = 7.73$$

$$L = \frac{1 + \sqrt{1 + a^2}}{2} \qquad \frac{1 + \sqrt{1 + 7.73^2}}{2} = 4.40$$

Exit Gradient.

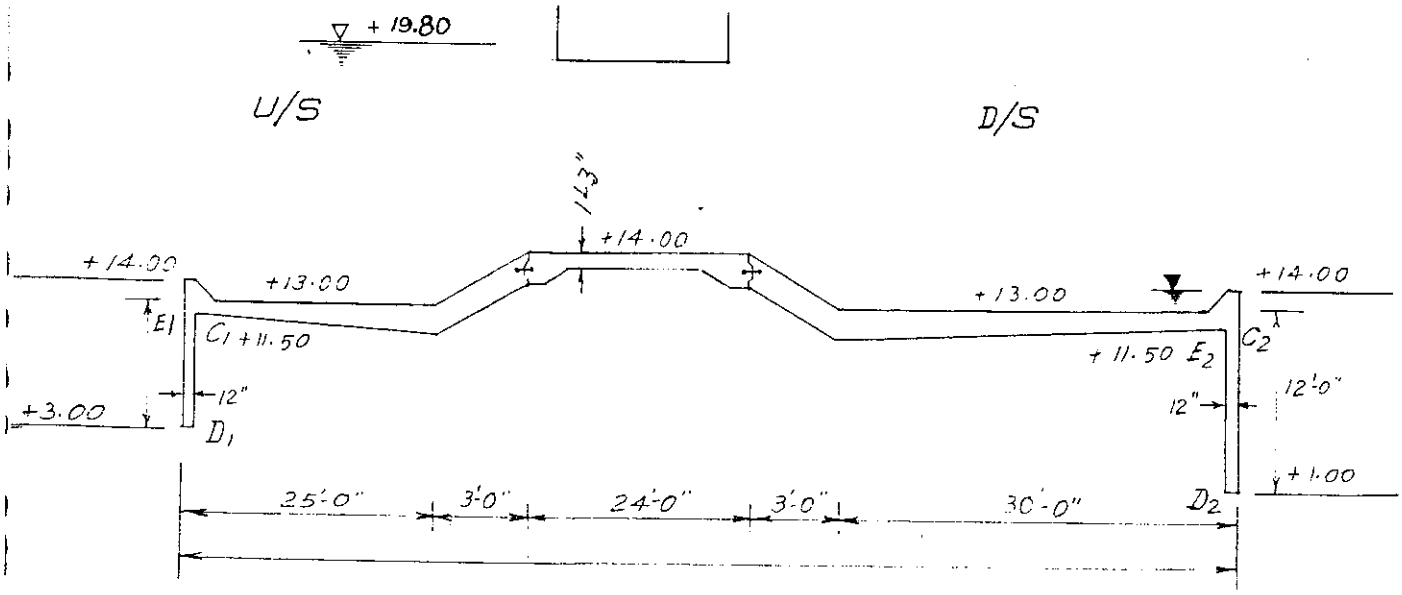
$$(GE) = \frac{H}{d} \times \frac{1}{\pi\sqrt{L}}$$

$$= \frac{7}{11} \times \frac{1}{\pi\sqrt{4.40}} = 0.097$$

Hence the exit gradient shall be equal to 0.097 i.e, $\frac{1}{10.35} < \frac{1}{7}$ O.K.

- **Computation for uplift pressure and floor thickness.**

a) **River side floor.**



Upstream cutoff No.1

Total length of the floor b = 85'

Depth of U/S cutoff d = 14.00 - 3.00 = 11.00'

$$a = \frac{b}{d} = \frac{85}{11} = 7.73$$

1/a = 0.129

From curve (Figure 9.1)

PE_1 = 100%

PC_1 = 100% - P_E = 100% - 32% = 68%

PD_1 = 100% - P_D = 100% - 22% = 78%

The value of PC_1 must be corrected for three corrections as below :

i) Correction due to the effect of cutoff No.2 on cutoff No.1

$$\text{Correction \%} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d + D}{b}\right]^{1/2}$$

where,

D = depth of cutoff (2) below the point C_1 i.e, the point at which interference is desired.

= 11.50 - 1 = 10.50'

d = depth of cutoff No. 1 = 11.5 - 3 = 8.50'

b' = distance between two cutoffs = 83'

b = Total length of floor = 85'

$$\text{Correction} = 19 \left[\frac{10.5}{83}\right]^{1/2} \left[\frac{8.5 + 10.5}{85}\right]^{1/2}$$

= 1.51 % (+ve)

ii) Correction due to thickness of floor.

Pressure calculated from curve for PC_1, is at the top of floor but we want to know the pressure at the bottom of slab.

$$\text{Correction} = \frac{PC_1 - PD_1}{\text{Distance between } C_1 \text{ \& } D_1} \times \text{thickness of floor}$$

$$= \frac{78\ \% - 68\ \%}{13.00 - 3.00} \times 1.5 = 1.50\ \% \ (+ve)$$

iii) Correction due to slope = nil

Corrected PC_1 = 68 % + 1.51 % + 1.5 % = 71.01 %

Downstream cutoff wall No. 2.

1/a = d/b = 13/85 = 0.153

PC_2 = 0 %

PE_2 = PE = 34 %

PD_2 = PD = 24 %

Correction for PE_2

i) Correction due to the effect of cutoff No.1 on cutoff No.2

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d + D}{b}\right]^{1/2}$$

D = 11.5 - 3 = 8.5'

d = 14.0 - 1.0 = 13.0

b' = 83'

b = 85'

$$\text{Correction} = 19 \left[\frac{8.5}{83}\right]^{1/2} \left[\frac{10.5 + 8.5}{85}\right]^{1/2}$$

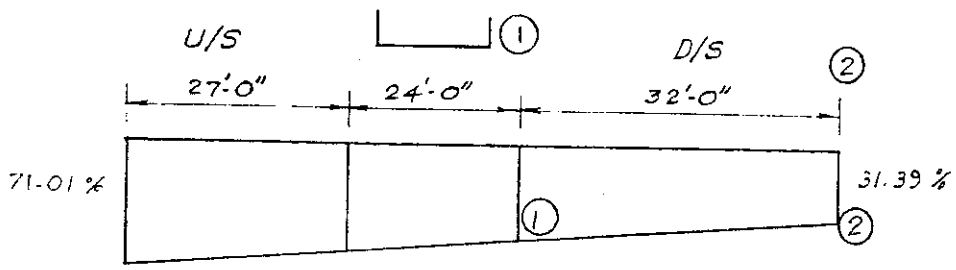
$$= 1.36\ \%\ (-\text{ve})$$

ii) Correction due to thickness of floor

$$= \frac{34\ \% - 24\ \%}{13.00 - 1.00} \times 1.5 = 1.25\ \%\ (-\text{ve})$$

iii) Correction due to slope = nil

$$\text{Corrected } PE_2 = 34\ \% - 1.36\ \% - 1.25\ \% = 31.39\ \%$$



Design Head, H = 6'

Maxm uplift pressure at sec.1 = 46.67 % of 6'-0" = 2.80' of water

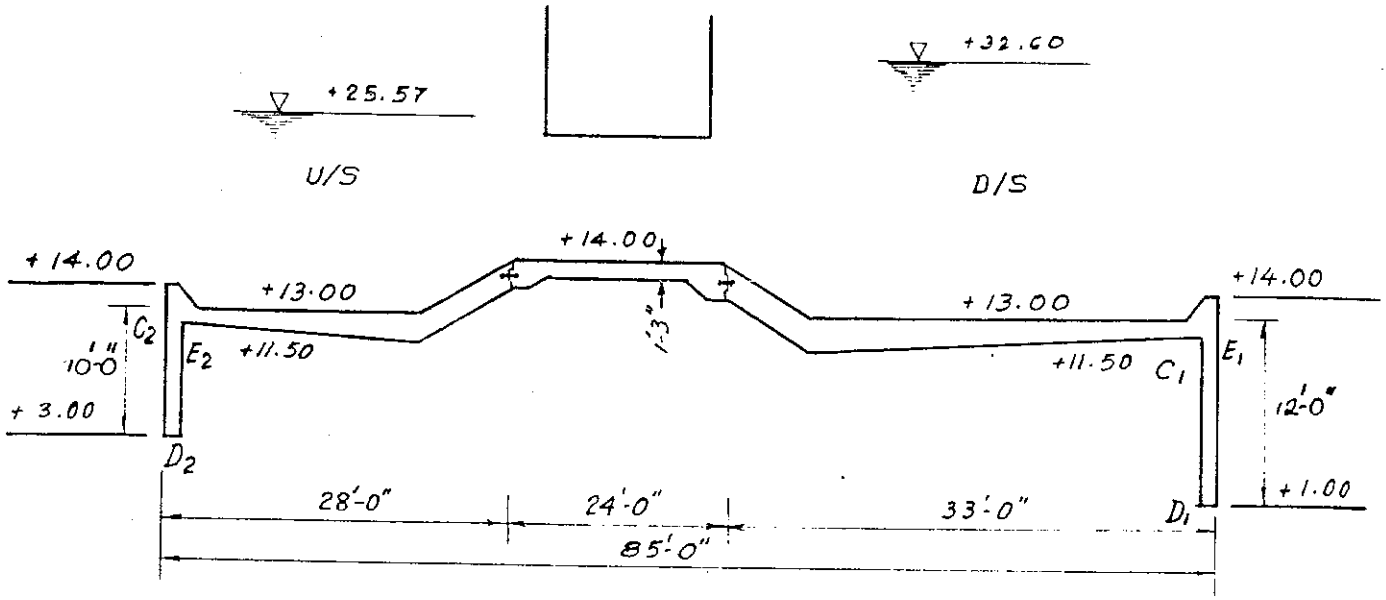
Maxm uplift pressure at sec.2 = 31.39 % of 6'-0" = 1.88' of water

$$\frac{\text{The submerged unit weight of concrete}}{\text{Unit weight of water}} = \frac{150 - 62.4}{62.4} = 1.4$$

$$\text{Required thickness of floor at sec.1} = \frac{2.80}{1.4} = 2.00 \text{ ft.}$$

$$\text{Required thckness of floor at sec.2} = \frac{1.88}{1.4} = 1.34 \text{ ft.}$$

b) C/S floor.



Downstream cutoff No.1

Total length of the floor b = 85'

Depth of D/S cutoff d = 14.00 - 1.00 = 13.00'

$$a = \frac{b}{d} = \frac{85}{13} = 6.538$$

1/a = 0.153

From curve (Figure 9.1 Page)

PE_1 = 100 %

PC_1 = 100 % - P_E = 100 % - 34 % = 66 %

PD_1 = 100 % - P_D = 100 % - 24 % = 76 %

Correction for PC_1

i) Correction due to the effect of cutoff No. 2.

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d + D}{b}\right]^{1/2}$$

$D = 11.50 - 3 = 8.5'$

$d = 11.5 - 1.0 = 10.5'$

$b' = 83', \quad b = 85'$

$$\text{Correction} = 19 \left[\frac{8.5}{83}\right]^{1/2} \left[\frac{10.5 + 8.5}{85}\right]^{1/2}$$

$$= 1.36\ \%\ (+\ve)$$

ii) Correction due to thickness of floor.

$$= \frac{76\ \% - 66\ \%}{13.00 - 1.00} \times 1.5 = 1.50\ \%\ (+\ve)$$

iii) Correction due to slope = nil

$$\text{Corrected } PC_1 = 66\ \% + 1.36\ \% + 1.5\ \% = 68.86\ \%$$

<u>Downstream cutoff wall No. 2.</u>

$1/a = d/b = 11/85 = 0.129$

$PC_2 = 0\ \%$

$PE_2 = PE = 32\ \%$

$PD_2 = PD = 22\ \%$

Correction for PE_2

i) Correction due to the effect of cutoff No.1 on cutoff No.2

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d + D}{b}\right]^{1/2}$$

$D = 11.5 - 1.0 = 10.5'$

$d = 11.5 - 3.0 = 8.5$

$b' = 83',$

$b = 85'$

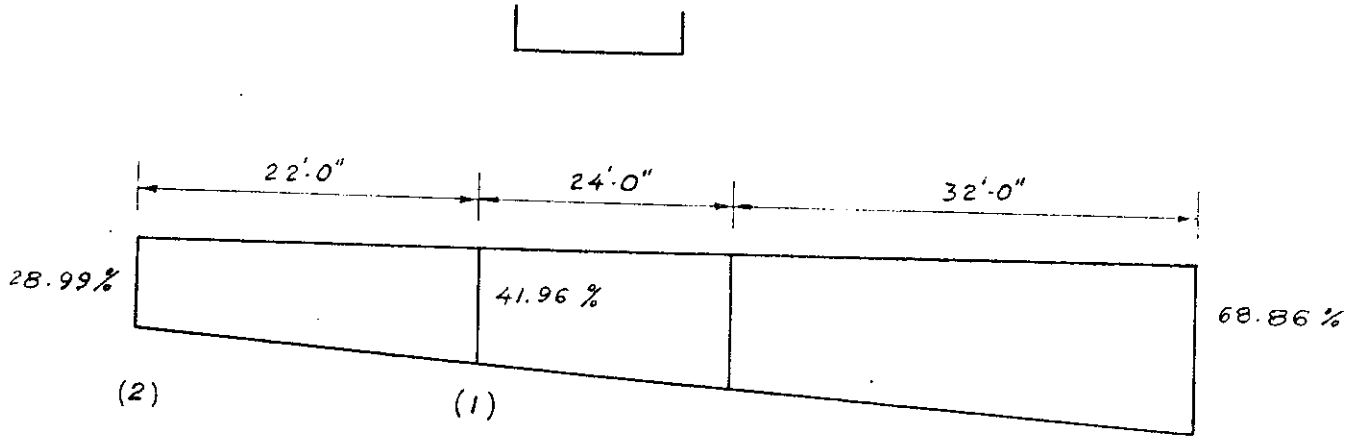
$$\text{Correction} = 19 \left[\frac{10.5}{83}\right]^{1/2} \left[\frac{8.5 + 10.5}{85}\right]^{1/2}$$

$$= 1.51\ \%\ (-\text{ve})$$

ii) Correction due to thickness of floor

$$= \frac{32\ \% - 22\ \%}{13.00 - 3.00} \times 1.5 = 1.50\ \%\ (-\text{ve})$$

$$\text{Corrected } PE_2 = 32\ \% - 1.51\ \% - 1.5\ \% = 28.99\ \%$$



Design Head, H = 7'

Maxm uplift pressure at sec.1 = 41.96 % of 7'- 0" = 2.94' of water

Maxm uplift pressure at sec.2 = 28.99 % of 7'- 0' = 2.02' of water

$$\text{Required thickness of floor at sec.1} = \frac{2.94}{1.4} = 2.10 \text{ ft.}$$

$$\text{Required thickness of floor at sec.2} = \frac{2.02}{1.4} = 1.44 \text{ ft.}$$

5.2 Design example for a sluice on Drainage Channel in Coastal Region.

Design data:

1. Drainage basin area = 5 square miles
2. Maximum R/S W/L in monsoon season (for average design conditions) = + 9.00 ft PWD
3. Minimum R/S W/L in monsoon season (for average design condition) = - 2.00 ft PWD
4. Bed level of natural channel at proposed site = - 1.00 PWD
5. Bed width of natural channel at proposed site = 26 ft.
6. Average ground level of the basin = + 4.00 ft PWD
7. Daily rainfall excess (10 year recurrence interval) = 1.8 inches
8. Crest level of embankment = 16.0 ft PWD

- **Sluice Size Determination.**

The analysis is as follows

Design C/S W/L = + 5.00 ft (Average GL + 1.0 ft)

Tidal range is 11 ft in 6 hour

Assuming a linear water level fluctuation, the water level rises or falls at a rate of 1.83 ft per hour.

Invert elevation of the sluice : - 2.0 ft.

Ratio of head water depth to barrel height : HW/D = 7/6 = 1.16

Discharg per unit barrel width : Q/B = 51 cfs (see design chart 14 on Appendix A)

Total discharge for a box, 5 ft wide = 51 x 5 = 255 cfs

Critical depth computed by $Y_c = 0.315\ (Q/B)^{2/3}$

(For rectangular sections)

$$Y_c = 0.315\ (255/5)^{2/3} = 4.33 \text{ ft.}$$

The sluice would be discharging in inlet-control as long as the tailwater is at or below the following elevation.

$$\frac{Y_c + D}{2} = \frac{4.33 + 6}{2} = 5.16 = \text{T.W.EL.} + 3.16 \text{ PWD}$$

Discharge in inlet control would remain constant at 255 cfs since the headwater is assumed constant. From the elevation of 3.16 ft PWD to el. 5.0 PWD control would be shifted to outlet and the discharge would linearly be reduced to zero.

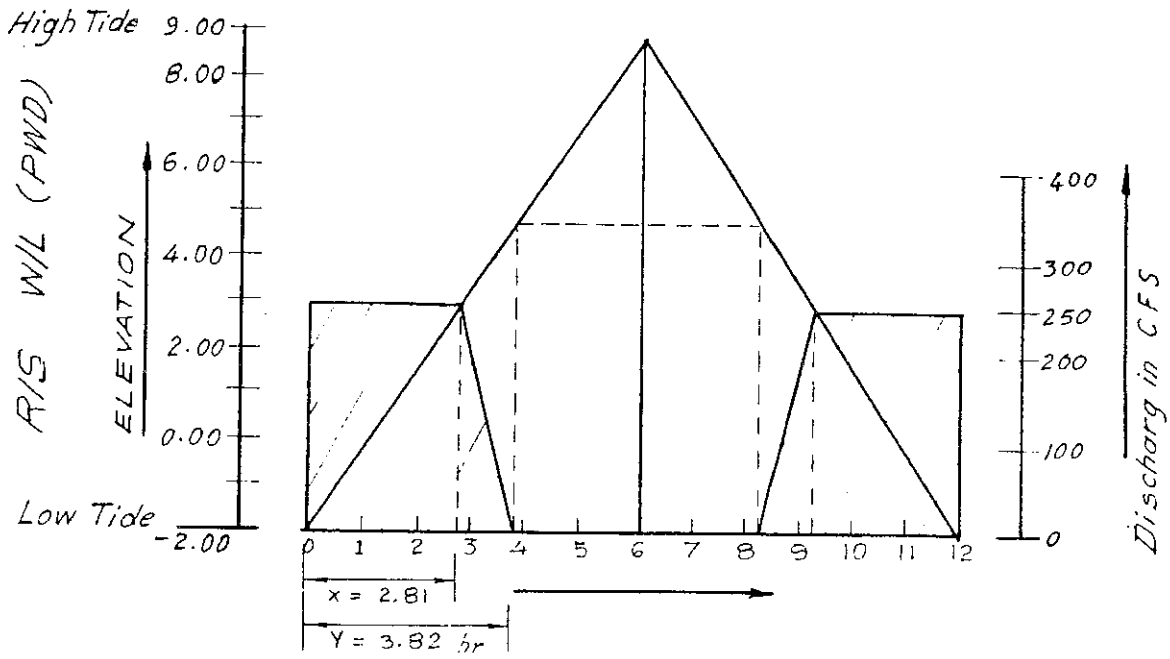


Fig. 21 : Tide and Discharge Curve.

Drainage is constant, by similar triangles, for

$x = 6/11 \times 5.16 = 2.81$ hour

Drainage reaches a discharge rate of zero from change of tide to tide blockage.

Y = 6/11 x 7 = 3.82 hour

Volume of water discharged under one drainage curve

$$V = [2\ (2.81 \times 255) + \frac{2\ (3.82 - 2.81) \times 255}{2}] \times 3600$$

= 6086340 cft.

= 139.72 acre ft for 12 hours

= 280 acre ft for 24 hours

= 140 cfs for 24 hours

(since 1 cfs = 2 acre ft per day)

Average discharge for 24 hours period by one vent 5'x 6' sluice is 140 cfs.

Since 1" of rainfall excess over an area of 1 square mile would generate 27 cfs, total drainage discharge for the basin would be

= 27 x rainfall excess x areas in square miles

= 27 x 1.8 x 5.0 = 243 cfs.

No. of vents required = 243/140 = 1.74 vents.

Provide 2 vents 5'x 6'

- <u>Design of outlet basin</u>

Discharge for 2 vent 5'x 6' sluice = 510 cfs (from nomograph, design chart 14).

Energy level U/S of the sluice = + 5.0 ft.

Velocity head at the U/S and losses at entrance are neglected since these would compensate for each other.

Energy level at the exit = 5.0 ft.

Try a 5.0 ft drop in elevation of R/S apron from the invert elevation - 2.00 PWD.

Specific energy at the toe of glacis = 5.0 - (- 5.0) = 10 ft.

Provide a flare angle of = $\tan^{-1}$ (1/3F) in the wingwall

For critical flow condition at the exit of barrel

Flare angle = $\tan^{-1}$ 1/3 = 18.43^{o}

Width of the basin at the toe of glacis

$$= 14 + 2 \times 15 \times \tan 18.43^{o} = 24 \text{ ft.}$$

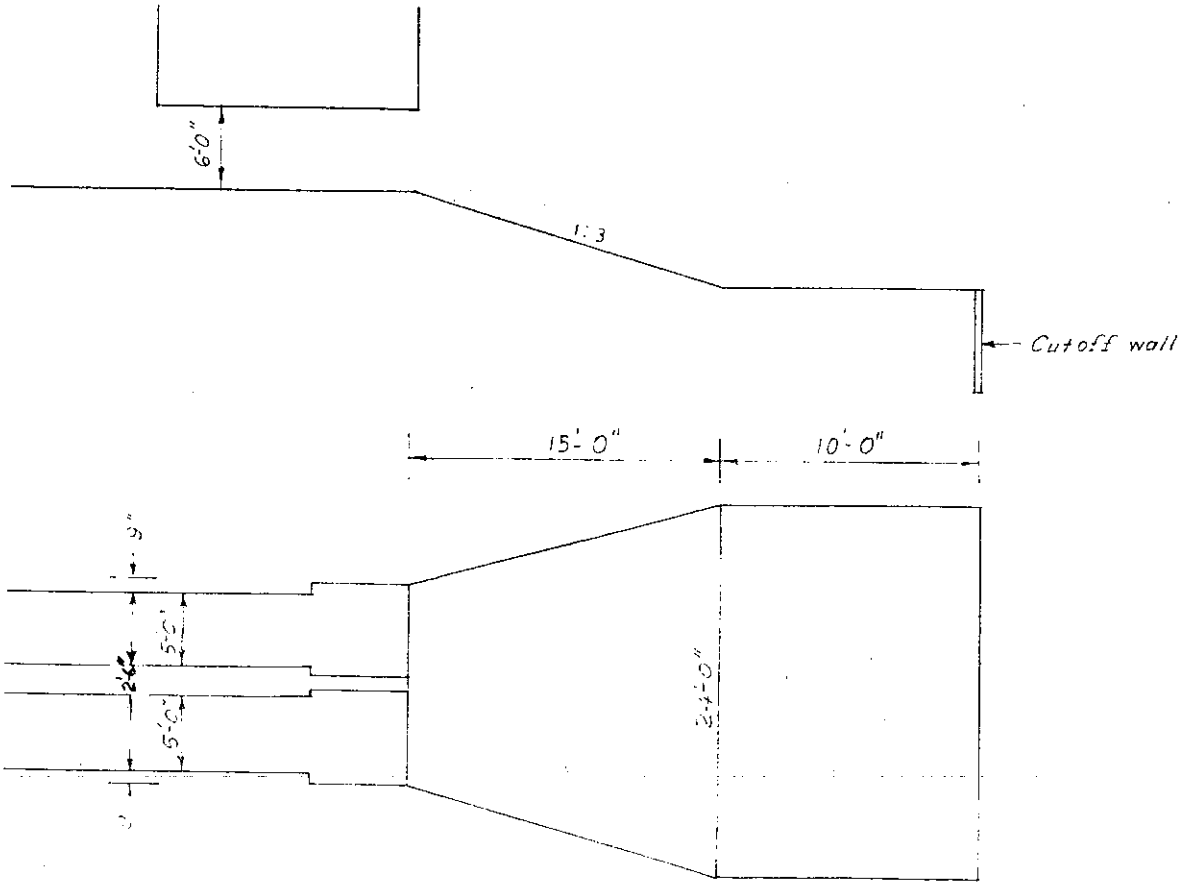


Fig. 22 : Plan and Section of Sluice Outlet Basin.

The glacis is usually sloped 1:3 (V:H)

For a 5 ft drop the horizontal length turns out to be 15 ft.

Velocity of flow at the toe $V_1 = \frac{Q}{b\,d_1} = \frac{510}{24 \times d_1} = \frac{21.25}{d_1}$

Thus $d_1 + V_1^2/(2g) = 10.00$

$$d_1 + (21.25/d_1)^2/(2g) = 10.00$$

Trail-and-error gives $d_1 = 0.875'$

Thus the depth of flow at the toe = 0.875'

Velocity of flow : $V_1 = 21.25/0.875 = 24.3$ fps.

Froude No. of flow : $F_1 = V_1/(gd_1)^{1/2} = 24.3/1(32.2 \times 0.875)^{1/2}$

$= 4.6$

For rectangular hydraulic jump basin, the sequent depth is obtained by :

$$d_2 = \frac{d_1}{2}\left[(1 + 8F_1^2)^{1/2} - 1\right] = 5.23 \text{ ft.}$$

However, the tailwater is 5 ft. Hence tailwater is inadequate for the jump to take place in a simple hydraulic jump basin. The situation would improve, and the length of the basin would be reduced if a SAF type basin was provided. For a SAF basin, (see Figure 14) the tailwater elevation required (D_2) for formation of jump is given by

$$D_2 = d_2\,(1.1 - F^2/120)$$

$D_2 = 4.84$ ft actual tailwater = 5.0 ft O.K.

Length of basin for a SAF type stilling basin is given by

$$L_B = 4.5\, d_2 / F_1^{0.76}$$

$$L_B = 7.4 \text{ ft} \quad \text{Provide 10 ft.}$$

Establish chute, apron (or floor) blocks and end sill dimensions using the guidelines for SAF stilling basin. For detailed hydraulic design, design example in para 5.01 may be used as guide.

5.3 Design Example for a Water Retention Structure.

Site Conditions and General Description

It is proposed to construct a water retention structure on a channel having a bed width of 60 feet, side slopes of 2:1 and depth from average bed level to top of banks of about 9 feet. The structure will store water following the monsoon season for use in irrigating cultivable land bordering the channel. The structure will be designed to permit little or no backwater effect upstream of the structure during flood flows. The site specific parameters necessary for establishing the opening dimension of the structure are given below :

Data

Natural Bed Width	=	60 feet
Natural Bank Height	=	9 feet
Side Slopes of Channel	=	2:1
Discharge (at Bankful stage)	=	2500 cfs.

Hydraulic Design Analysis

For a channel flowing at a discharge of 25oo cfs the normal depth of flow is obtained using recognized hydraulic charts or equations. For out problem the depth of flow is 9 feet. At this flow the average velocity in the cross-section is 3.6 feet per second.

The analysis proceeds as follows :

1. Compute the specific energy of the flow using the equation for trapezoidal sections :

$$E = Y + \frac{Q^2}{2gA^2}$$

Where Y = depth of flow from mean bed level

Q = discharge is cfs

A = cross-sectional area in ft^2

g = acceleration due to gravity i.e, 32.2 ft/s^2

E = Specific energy above bed level

Thus E = 9' + 0.2' = 9.2 feet

2. Compute the minimum width of a rectangular and a trapezoidal section which will convey this flow without "chocking" the flow. For the rectangular section the procedure is :

Assuming critical depth will occur at the constriction the depth of flow d_c is then 2/3 (two-thirds) of the specific energy of the upstream flow. Thus d_c = 2/3 x 9.2 = 6.1 ft

Now for rectangular channels

$$d_c = (q^2/g)^{1/3} \quad \text{where } q = Q/b$$

$$d_c^3 = q^2/g = \frac{2500^2}{gb^2}$$

$$b^2 = \frac{2500^2}{gd_c^3} = 855$$

$$b = 29 \text{ feet}, \quad \text{Say 30 feet.}$$

Thus an opening with a clear span of 30 feet will just permit flow to take place without noticeable backwater effect. However, if piers are installed in this opening to facilitate the placement of 5 ft long timber fall boards the opening must be increased to accomodate the piers plus some allowance for contraction of the flow at the piers. Thus for a design utilizing a rectangular opening and no step-up in the bed, minimum recommended opening distance from side to side is 41 feet.

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5. Handerson, F.M. Open Channel Flow Macmillan Publishing Co. Inc. Newyork, N.Y. U.S.A, 1966

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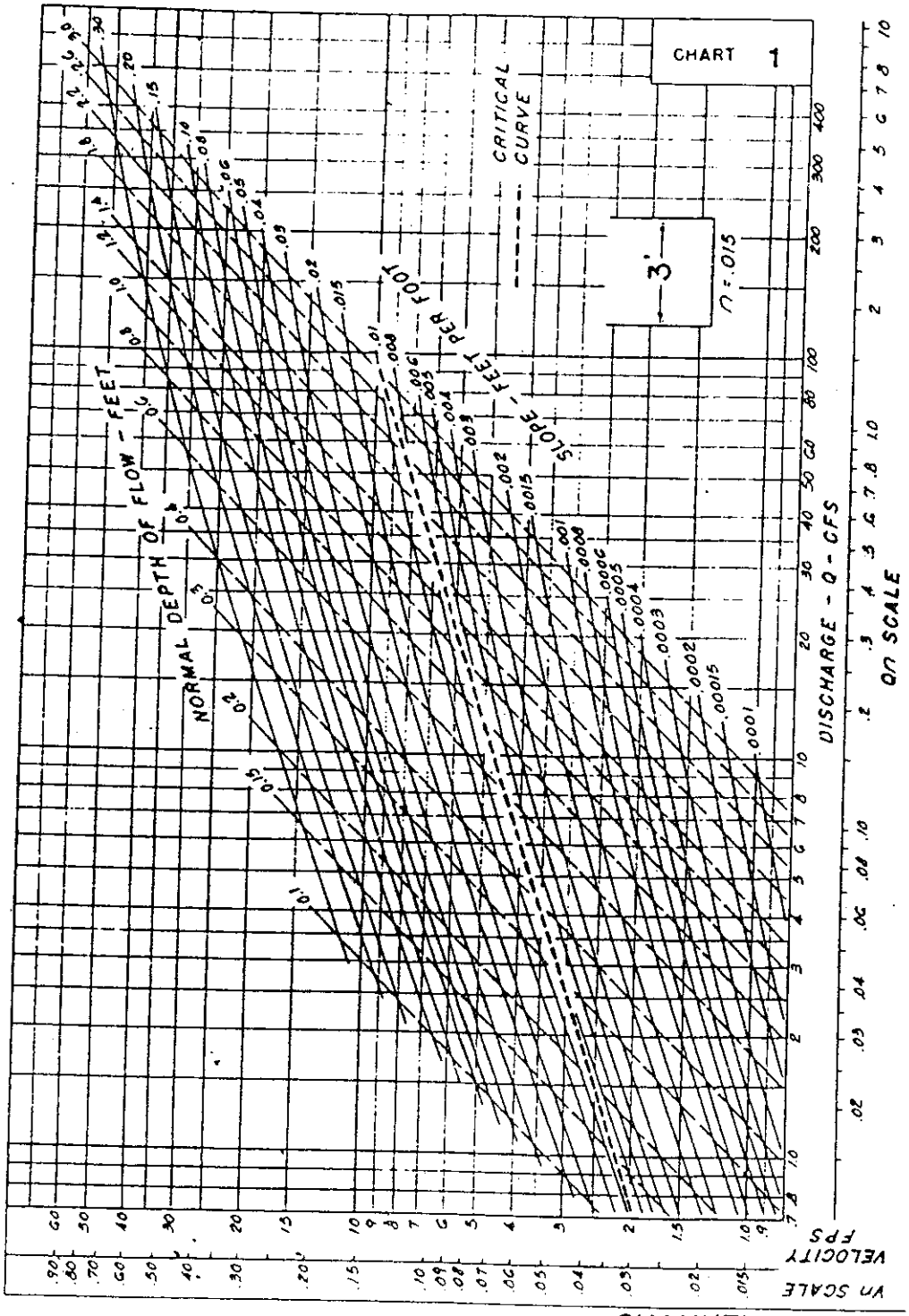
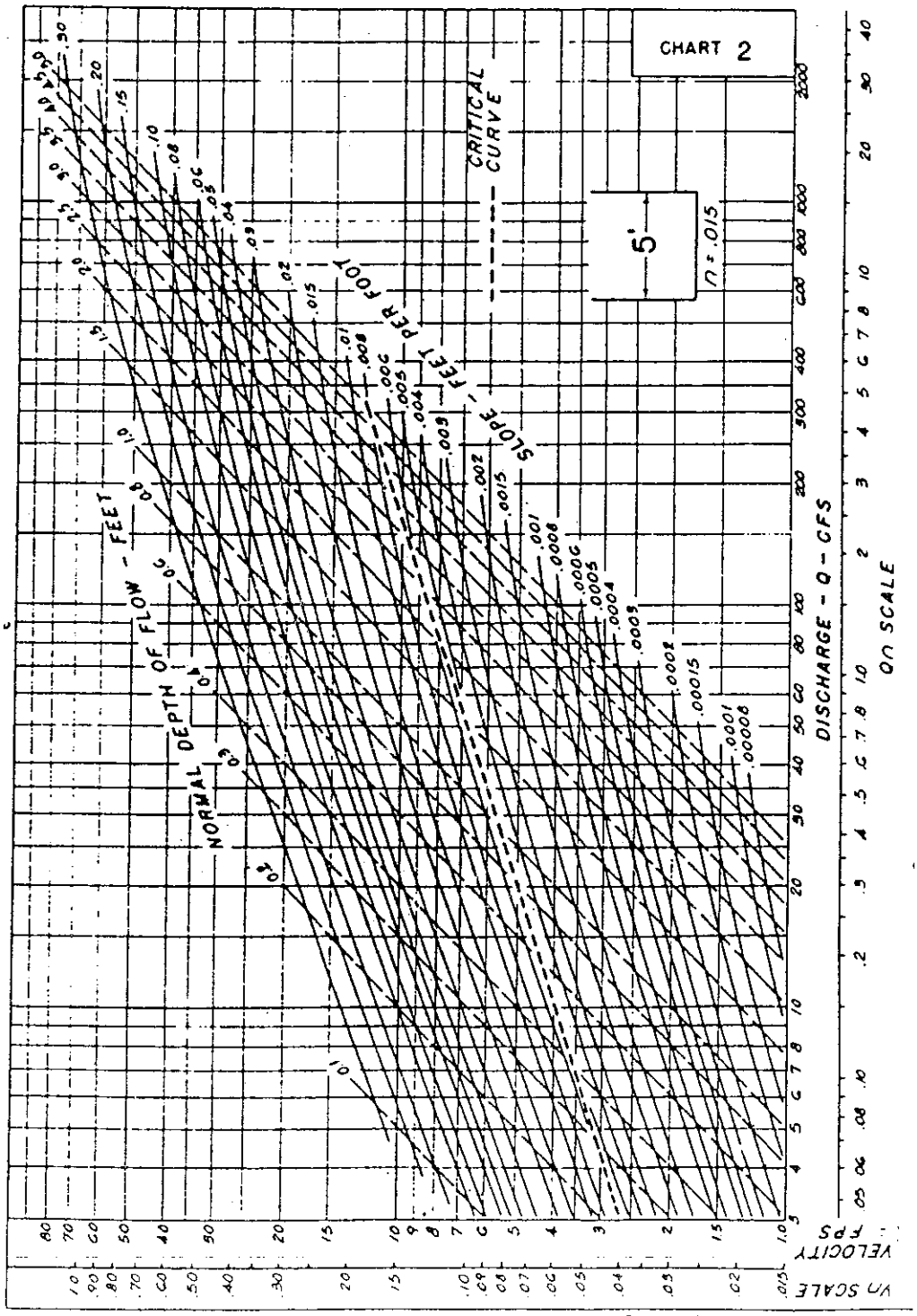
APPENDIX A

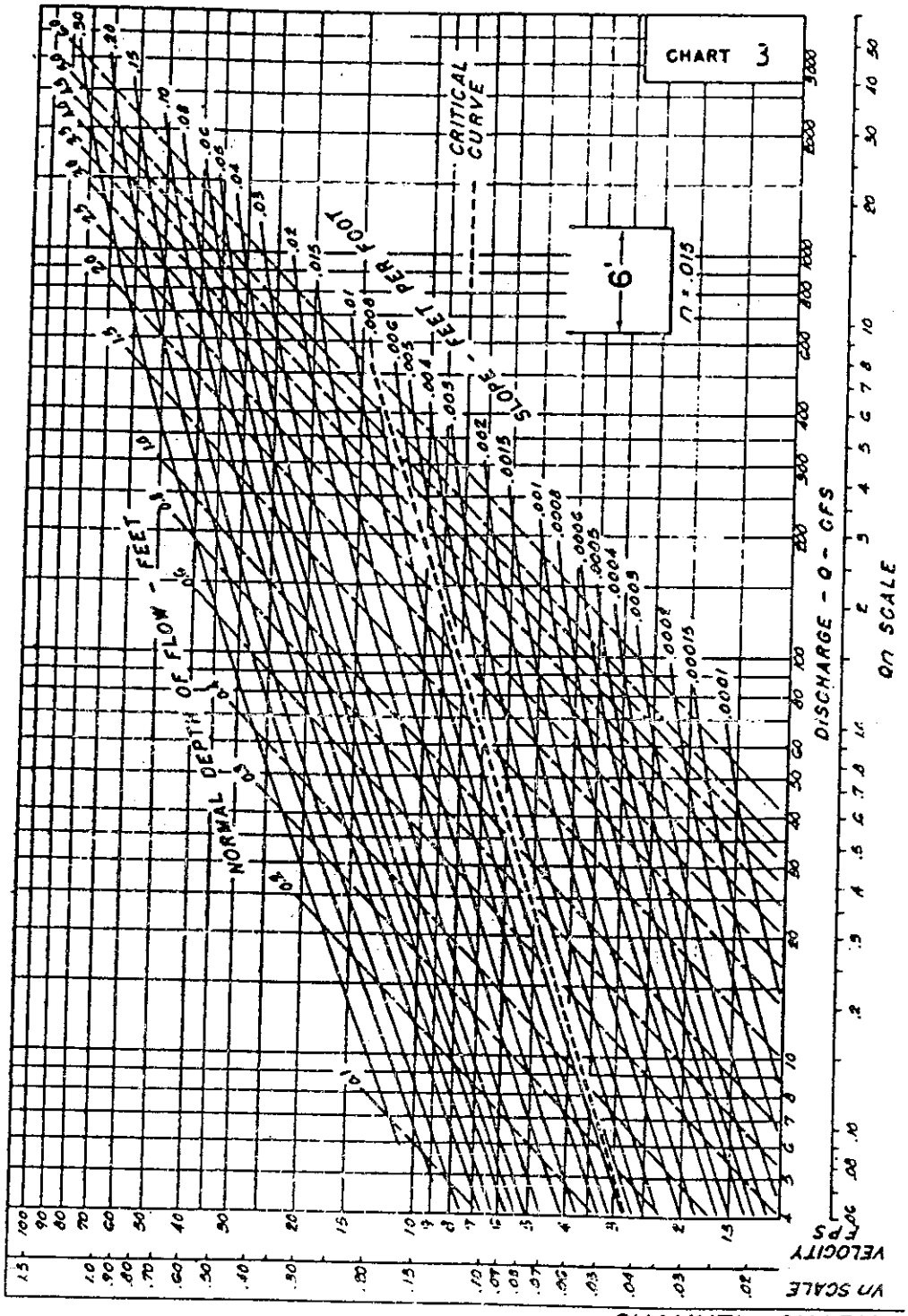
CHARTS AND TABLES



CHANNEL CHART
VERTICAL b = 3 FT.



CHANNEL CHART
VERTICAL b = 5 FT.



CHANNEL CHART
VERTICAL b = 6 FT.

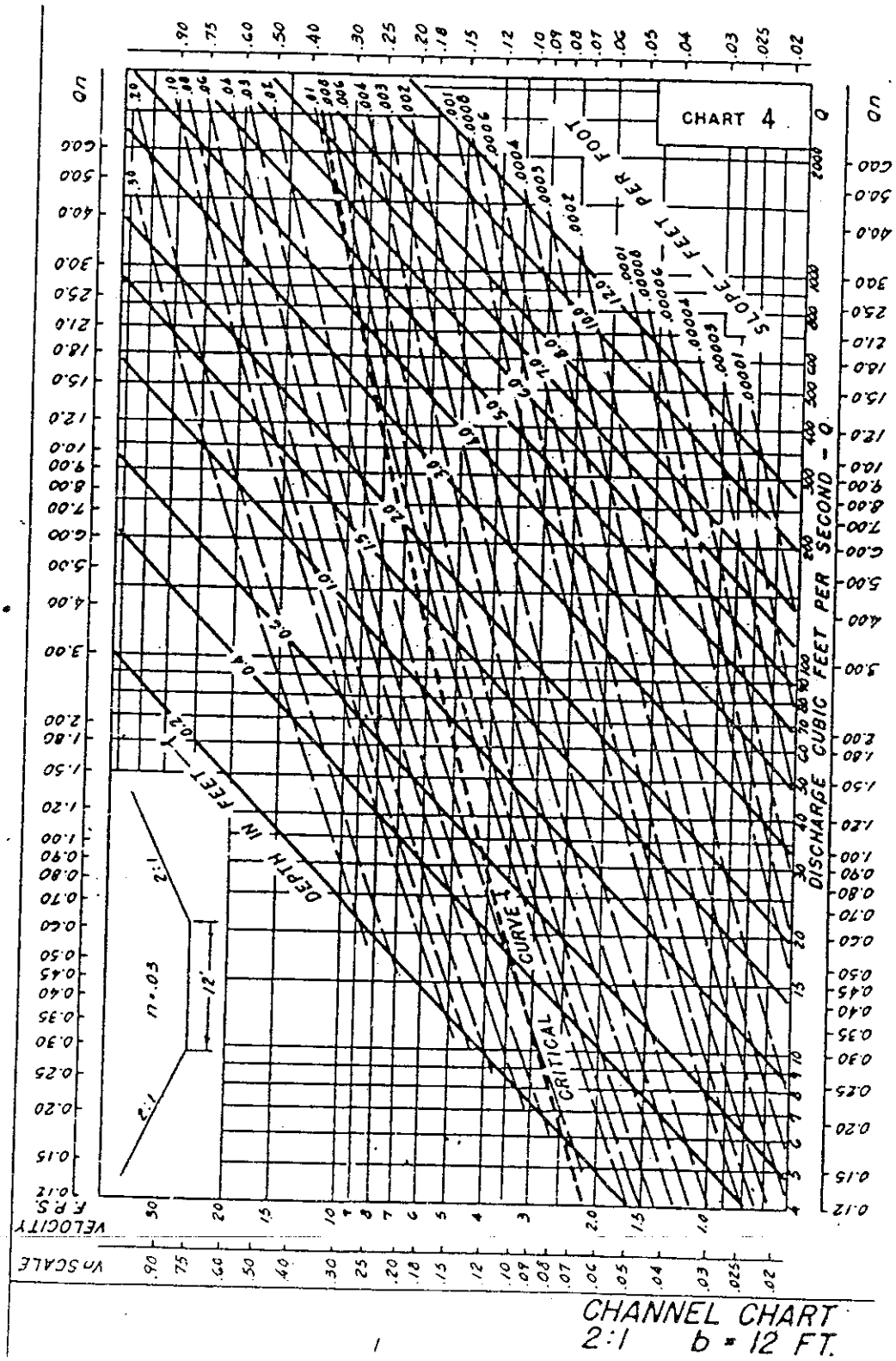
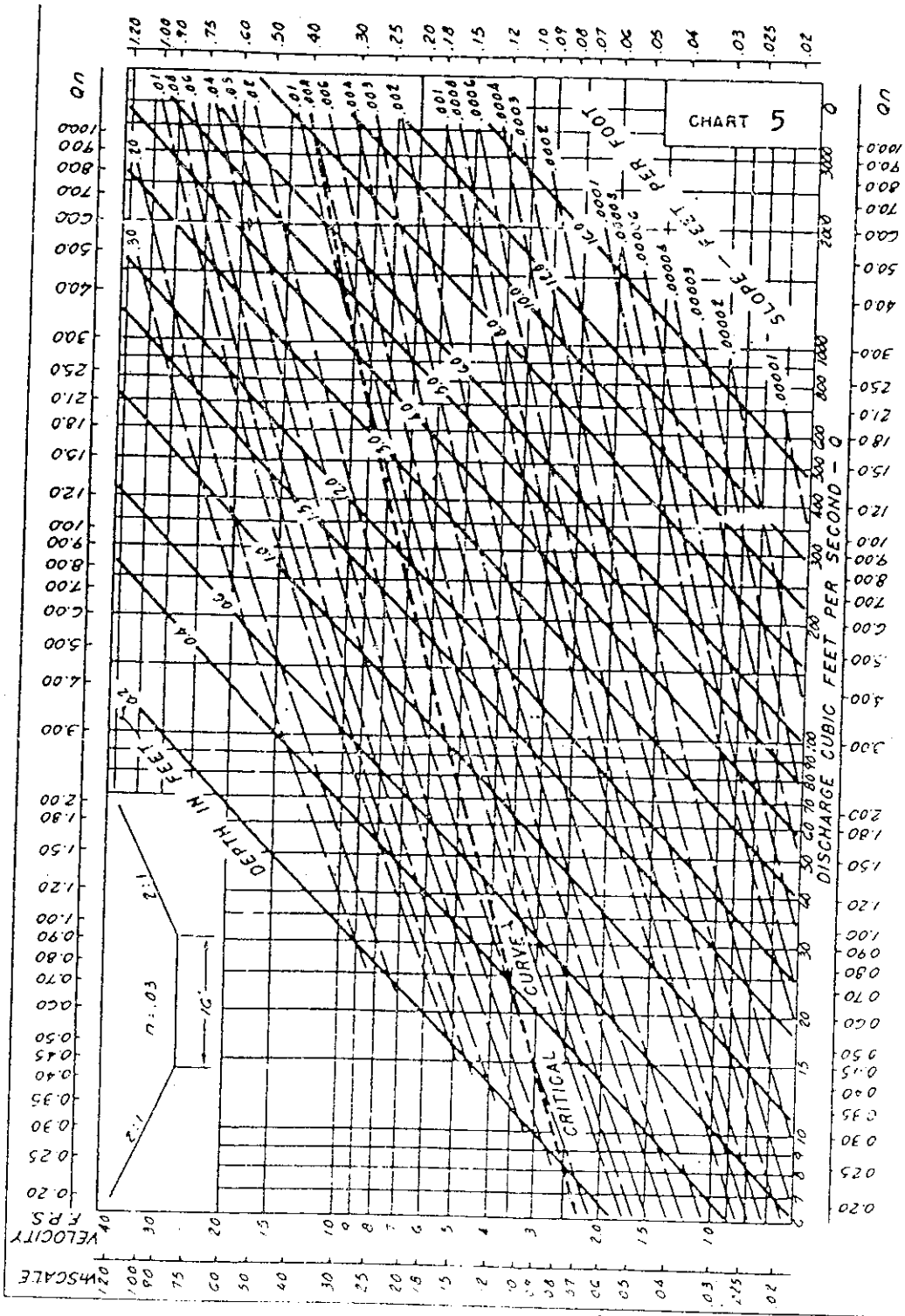
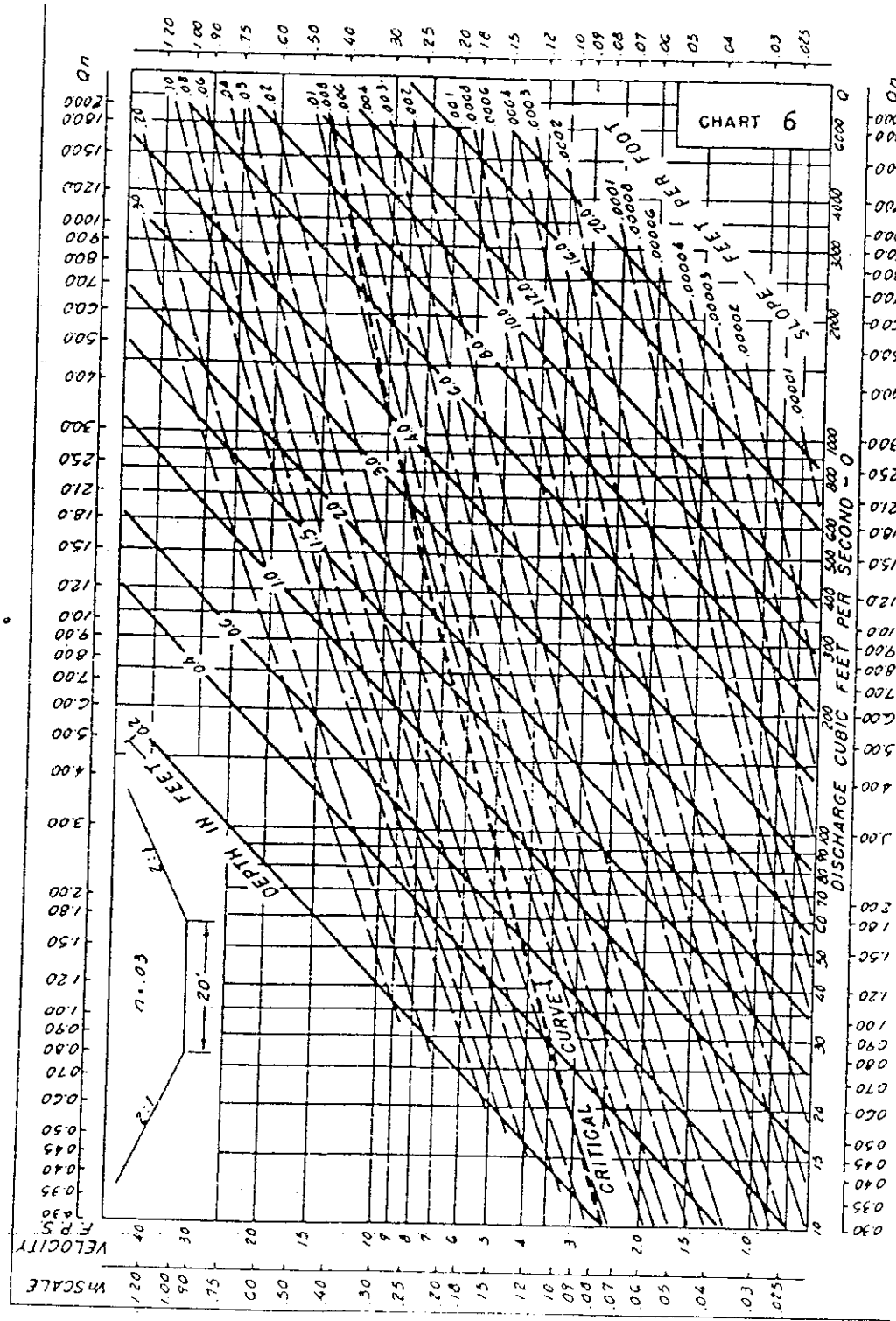


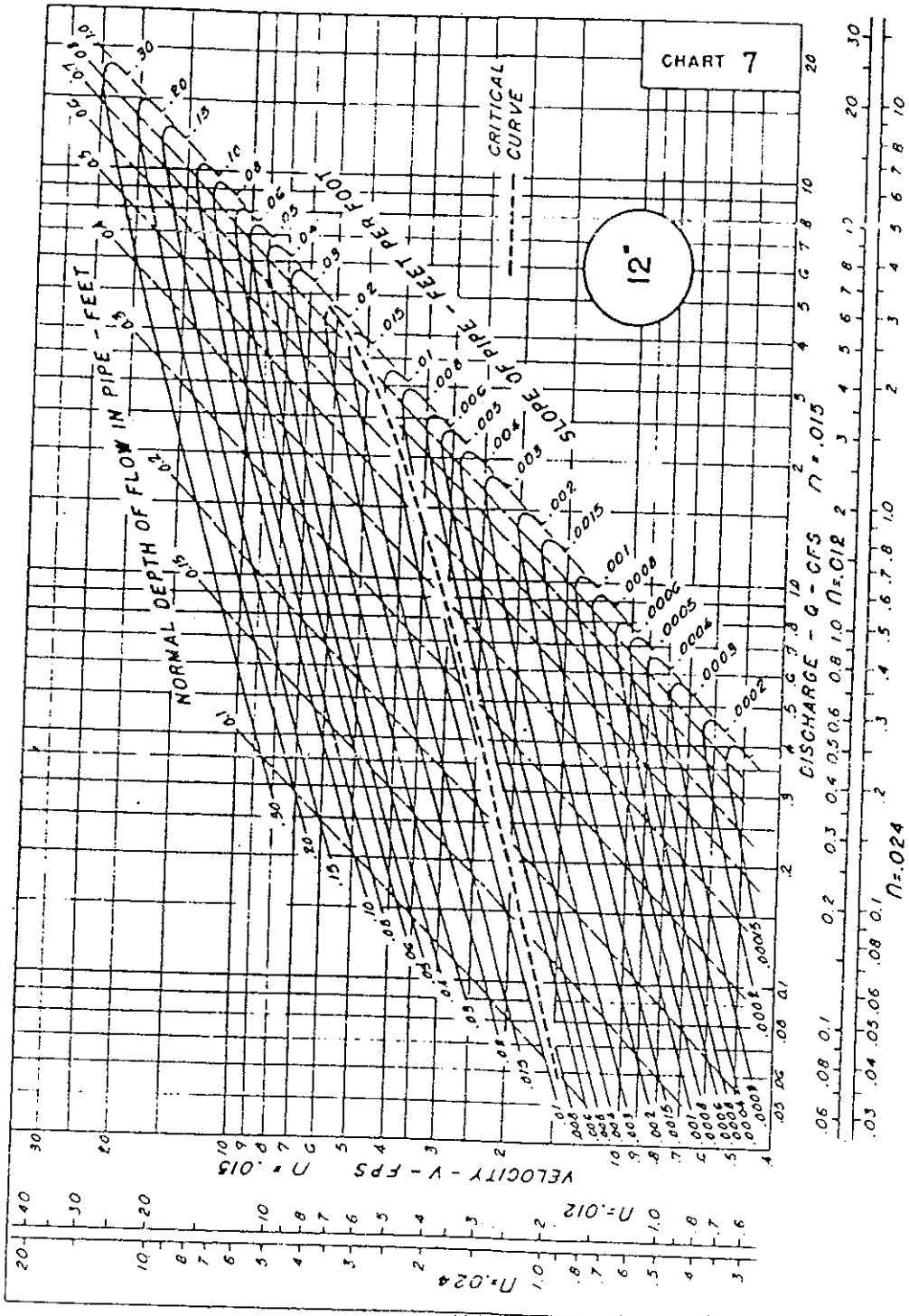
CHART 4
SLOPE - FEET PER FOOT
DISCHARGE CUBIC FEET PER SECOND - Q
DEPTH IN FEET - Y
CURVE 1
CRITICAL
VELOCITY F.P.S.
Vn SCALE
n = .03
12'
2:1
2:1
CHANNEL CHART
2:1 b = 12 FT.



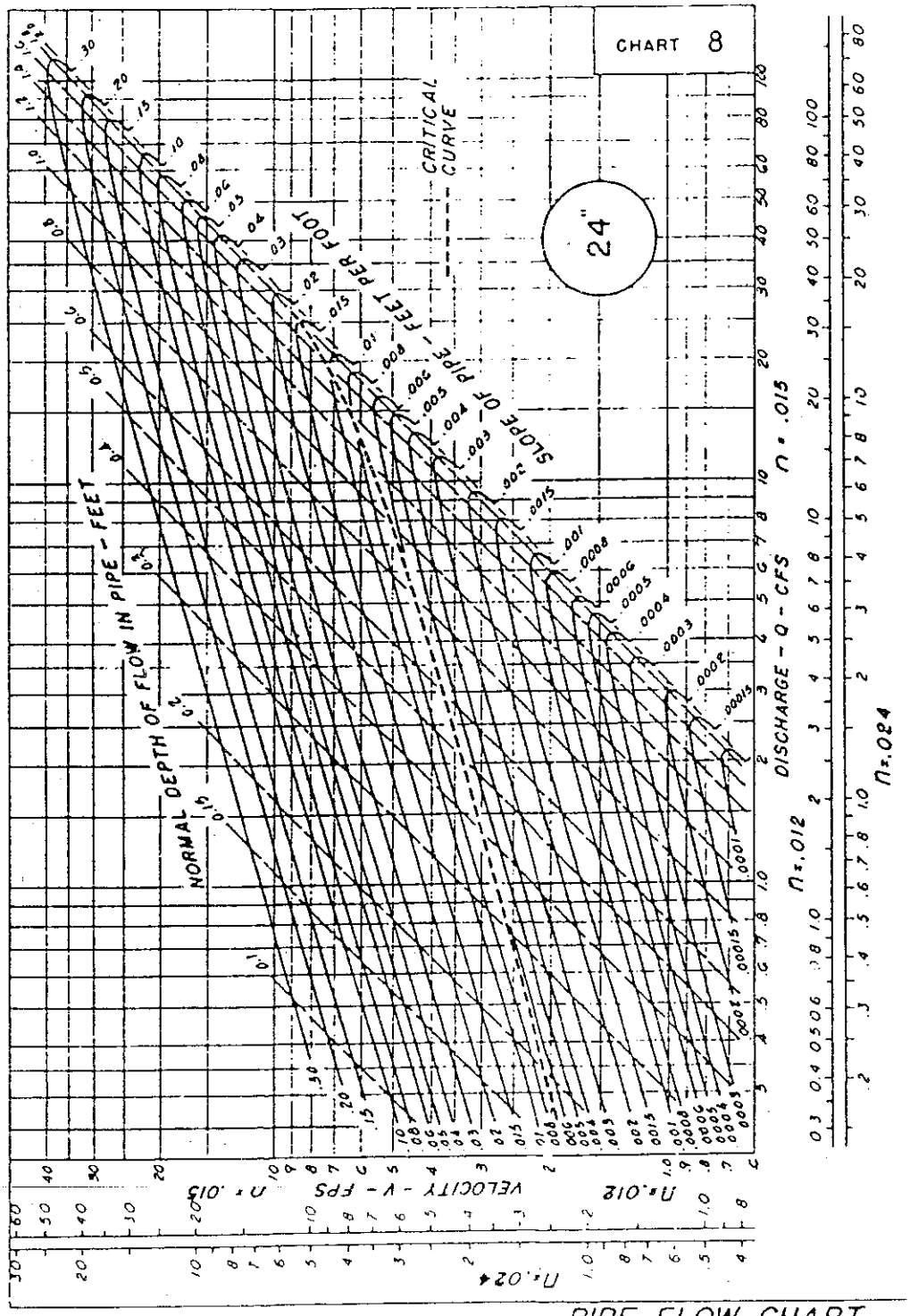
CHANNEL CHART
2:1 b = 16 FT.



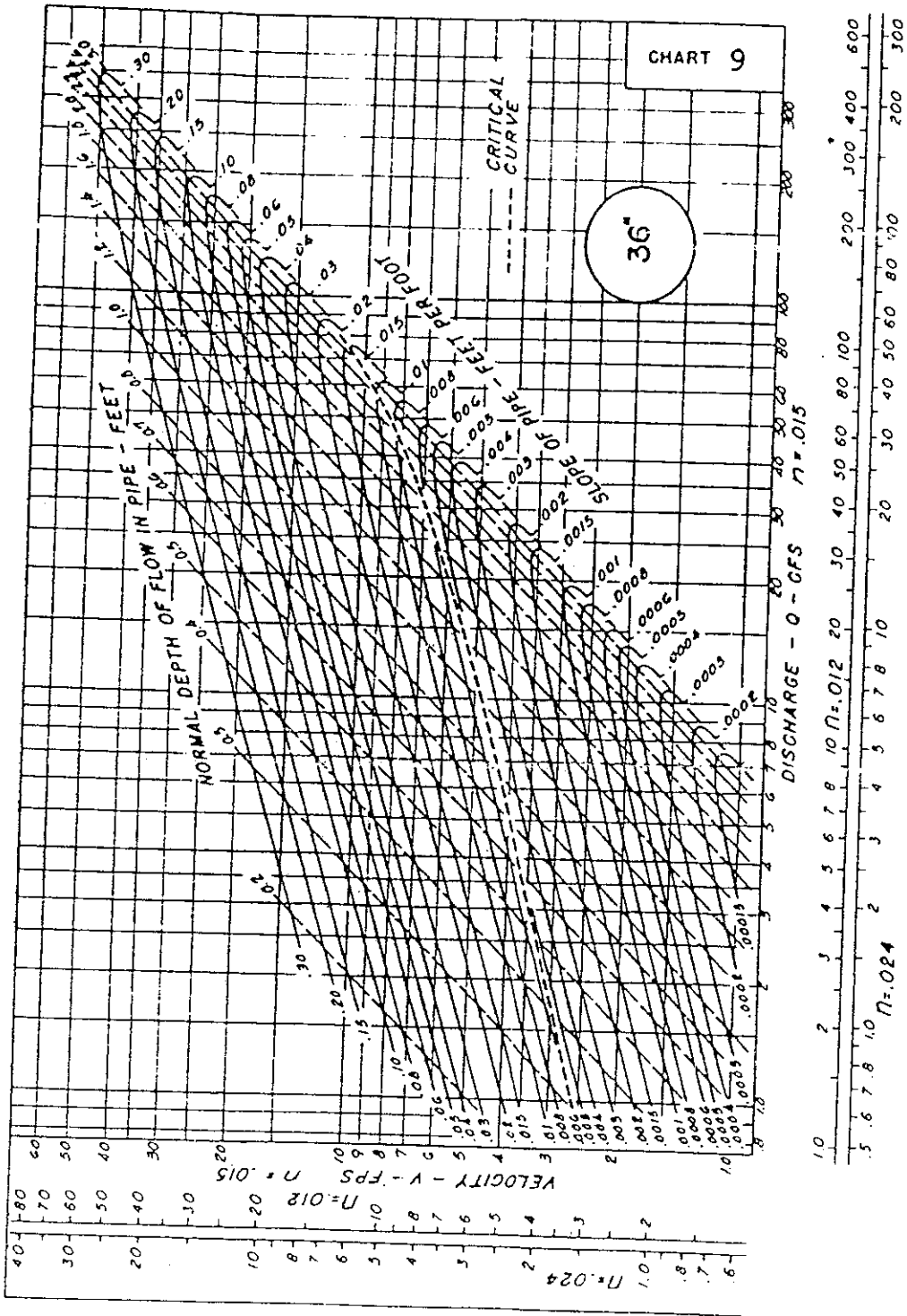
CHANNEL CHART
2:1 b = 20 FT.



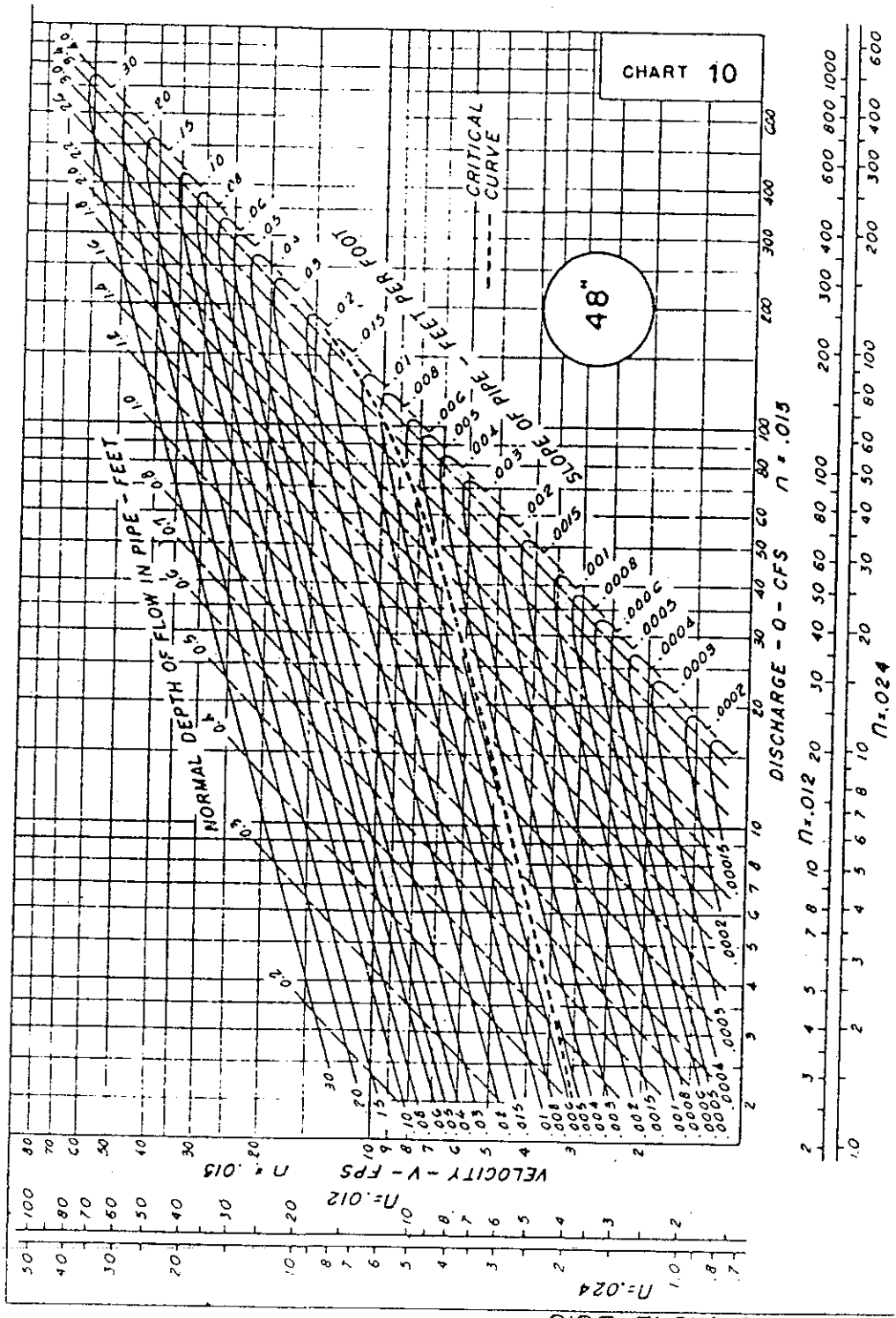
PIPE FLOW CHART
12-INCH DIAMETER



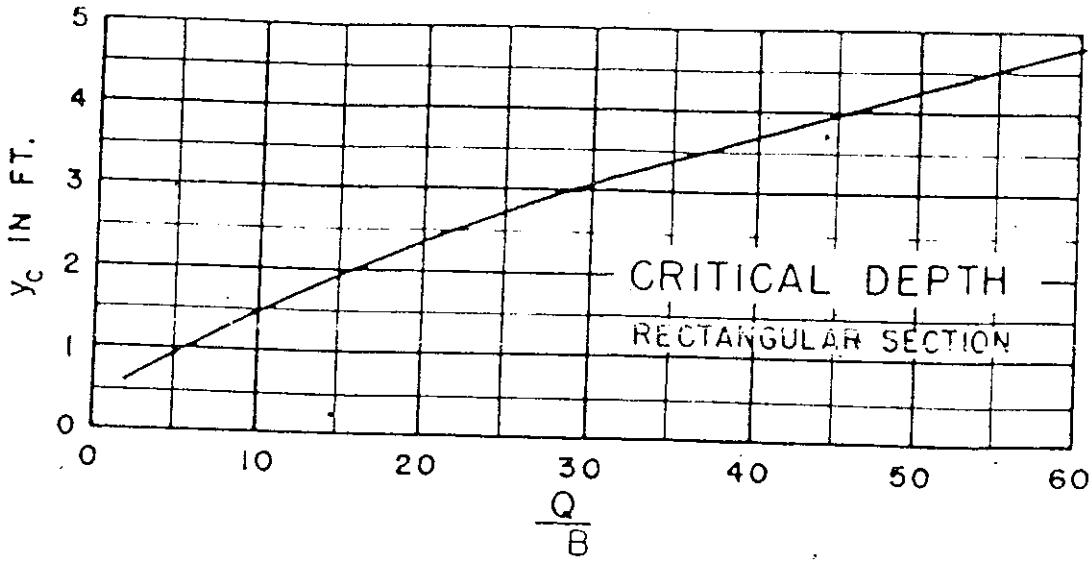
PIPE FLOW CHART
24-INCH DIAMETER

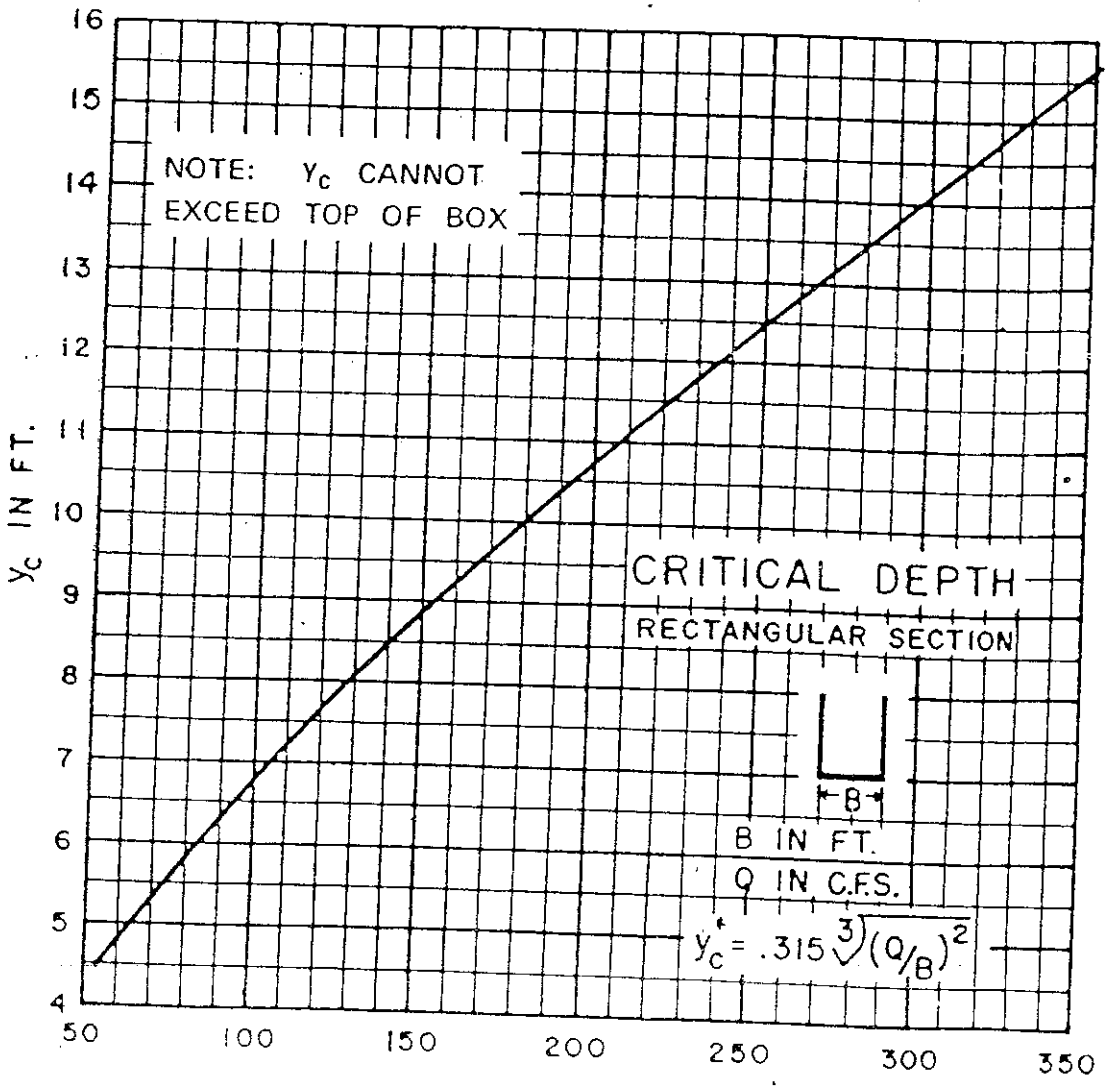


PIPE FLOW CHART
36-INCH DIAMETER



PIPE FLOW CHART
48-INCH DIAMETER





DESIGN CHART 11

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.* From Reference III-3.

d/b[1]	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
02	.00213	.00215	.00216	.00217	00218	.00219	.00220	.00220	00221	.00223
03	.00414	00419	.00423	.00426	.00429	.00431	00433	.00434	00437	.00443
04	00661	00670	.00679	.00685	.00690	.00696	.00700	.00704	00707	00722
05	00947	.00964	00980	.00991	.0100	.0101	.0102	.0103	0103	0106
06	.0127	.0130	.0132	.0134	.0136	.0137	.0138	.0140	.0141	0145
.07	.0162	.0166	.0170	.0173	.0176	.0177	.0180	.0182	.0183	.0190
.08	.0200	.0206	.0211	.0215	.0219	.0222	.0225	.0228	.0231	.0240
09	.0240	.0249	.0256	.0262	.0267	.0271	.0275	.0279	0282	.0296
10	.0283	.0294	.0305	.0311	.0318	.0324	.0329	.0334	.0339	.0358
11	.0329	.0342	.0354	.0364	.0373	.0380	.0387	.0394	.0400	.0424
.12	.0376	.0393	.0408	.0420	.0431	.0441	.0450	.0458	0466	.0497
13	.0425	.0446	0464	.0480	.0493	.0505	.0516	.0527	.0537	.0575
.14	.0476	.0501	.0524	.0542	.0559	.0573	.0587	.0599	.0612	.0659
15	.0528	.0559	.0585	.0608	.0628	.0645	.0662	.0677	.0692	.0749
16	.0582	.0619	.0650	.0676	.0699	.0720	.0740	.0759	0776	.0845
17	.0638	.0680	.0717	.0748	.0775	.0800	.0823	.0845	.0867	.0947
18	.0695	.0744	.0786	.0822	.0854	.0883	.0910	.0936	0961	.105
19	.0753	.0809	.0857	.0900	.0936	.0970	.100	.103	106	.117
20	.0813	.0875	.0932	.0979	.102	.106	.110	.113	.116	.129
21	.0873	.0944	.101	.106	.111	.115	.120	.123	.127	.142
.22	.0935	.101	109	.115	.120	.125	.130	.134	.139	.155
.23	.0997	.109	.117	.124	.130	.135	.141	.146	.151	.169
24	106	.116	.125	.133	.139	.146	.152	157	.163	.184
25	.113	.124	.133	.142	.150	.157	.163	.170	.176	.199
26	.119	.131	.142	.152	.160	.168	.175	182	189	.215
27	.126	.139	.151	.162	.171	.180	.188	195	.203	.232
.28	.133	.147	.160	.172	.182	.192	.201	.209	.217	.249
.29	.139	.155	.170	.182	.193	.204	.214	.223	.232	.267
.30	.146	.163	.179	.193	.205	.217	.227	.238	.248	.286
31	.153	.172	.189	.204	.217	.230	.242	.253	.264	.306
32	160	.180	.199	.215	.230	.243	.256	.269	281	.327
33	.167	.189	.209	.227	.243	.257	.271	.285	.298	.348
34	174	.198	.219	.238	.256	.272	.287	301	315	.369
35	181	.207	.230	.251	270	287	.303	318	334	.392
36	190	.216	.241	.263	.283	.302	.319	.336	.353	.416
37	196	.225	.251	.275	.297	.317	.336	.354	.372	.440
38	203	.234	.263	.289	.311	.333	.354	.373	.392	.465
39	210	.244	.274	301	326	.349	371	392	412	.491
40	218	.254	.286	.314	341	.366	.389	.412	433	.518
41	225	.263	.297	.328	357	.383	.408	432	455	.545
42	233	.279	.310	.342	373	401	.427	453	478	.574
43	241	282	.321	356	.389	418	447	474	.501	604
44	249	292	.334	.371	405	437	467	496	524	634

[1] For d/b less than 0.04, use of the assumption R = d is more convenient and more accurate than interpolation in the table.

DESIGN CHART 12 (a)

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula* - Continued, from Reference III 3

d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.45	.256	.303	.346	.385	.422	.455	.487	.519	.548	.665
.46	.263	.313	.359	.401	.439	.475	.509	.541	.574	.696
.47	.271	.323	.371	.417	.457	.494	.530	.565	.600	.729
.48	.279	.333	.384	.432	.475	.514	.552	.589	.626	.763
.49	.287	.345	.398	.448	.492	.534	.575	.614	.652	.797
.50	.295	.356	.411	.463	.512	.556	.599	.639	.679	.833
.52	.310	.377	.438	.496	.548	.599	.646	.692	.735	.906
.54	.327	.398	.468	.530	.590	.644	.696	.746	.795	.984
.56	.343	.421	.496	.567	.631	.690	.748	.803	.856	1.07
.58	.359	.444	.526	.601	.671	.739	.802	.863	.922	1.15
.60	.375	.468	.556	.640	.717	.789	.858	.924	.988	1.24
.62	.391	.492	.590	.679	.763	.841	.917	.989	1.06	1.33
.64	.408	.516	.620	.718	.809	.894	.976	1.05	1.13	1.43
.66	.424	.541	.653	.759	.858	.951	1.04	1.13	1.21	1.53
.68	.441	.566	.687	.801	.908	1.01	1.10	1.20	1.29	1.64
.70	.457	.591	.722	.842	.958	1.07	1.17	1.27	1.37	1.75
.72	.474	.617	.757	.887	1.01	1.13	1.24	1.35	1.45	1.87
.74	.491	.644	.793	.932	1.07	1.19	1.31	1.43	1.55	1.98
.76	.508	.670	.830	.981	1.12	1.26	1.39	1.51	1.64	2.11
.78	.525	.698	.868	1.03	1.18	1.32	1.46	1.60	1.73	2.24
.80	.542	.725	.906	1.08	1.24	1.40	1.54	1.69	1.83	2.37
.82	.559	.753	.945	1.12	1.30	1.47	1.63	1.78	1.93	2.51
.84	.576	.782	.985	1.18	1.36	1.54	1.71	1.87	2.03	2.65
.86	.593	.810	1.03	1.23	1.43	1.61	1.79	1.97	2.14	2.80
.88	.610	.839	1.07	1.29	1.49	1.69	1.88	2.07	2.25	2.95
.90	.627	.871	1.11	1.34	1.56	1.77	1.98	2.17	2.36	3.11
.92	.645	.898	1.15	1.40	1.63	1.86	2.07	2.28	2.48	3.27
.94	.662	.928	1.20	1.46	1.70	1.94	2.16	2.38	2.60	3.43
.96	.680	.960	1.25	1.52	1.78	2.03	2.27	2.50	2.73	3.61
.98	.697	.991	1.29	1.58	1.85	2.11	2.37	2.61	2.85	3.79
1.00	.714	1.02	1.33	1.64	1.93	2.21	2.47	2.73	2.99	2.97
1.05	.759	1.10	1.46	1.80	2.13	2.44	2.75	3.04	3.33	4.45
1.10	.802	1.19	1.58	1.97	2.34	2.69	3.04	3.37	3.70	4.96
1.15	.846	1.27	1.71	2.14	2.56	2.96	3.34	3.72	4.09	5.52
1.20	.891	1.36	1.85	2.33	2.79	3.24	3.68	4.09	4.50	6.11
1.25	.936	1.45	1.99	2.52	3.04	3.54	4.03	4.49	4.95	6.73
1.30	.980	1.54	2.14	2.73	3.30	3.85	4.39	4.90	5.42	7.39
1.35	1.02	1.64	2.29	2.94	3.57	4.18	4.76	5.34	5.90	8.10
1.40	1.07	1.74	2.45	3.16	3.85	4.52	5.18	5.80	6.43	8.83
1.45	1.11	1.84	2.61	3.39	4.15	4.88	5.60	6.29	6.98	9.62

DESIGN CHART 12 (b)

পল্লি সদন
উপ-বিভাগীয় প্রকৌশলী
নকশা সার্কেল-২
বাপাউবো, ঢাকা-১২১৫

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.*—Continued, from Reference III-3.

d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	6.81	7.55	10.4
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	7.33	8.14	11.3
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	7.89	8.79	12.2
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	8.47	9.42	13.2
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	9.08	10.1	14.2
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	9.72	10.9	15.2
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	10.4	11.6	16.3
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	11.1	12.4	17.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	12.4	13.2	18.7
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	12.5	14.0	19.9
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	13.3	14.9	21.1
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	15.0	16.8	23.9
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	16.7	18.7	26.8
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	18.6	20.9	30.0
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	20.6	23.1	33.4
2.50	2.07	1.58	7.52	10.6	13.7	16.8	19.8	22.7	25.6	37.0
2.60	2.16	4.90	8.14	11.6	15.0	18.4	21.7	25.0	28.2	40.8
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	27.4	31.0	44.8
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	29.9	33.8	49.1
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	32.6	36.9	53.7
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	35.4	40.1	58.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	41.5	47.1	68.9
3.40	2.90	7.97	14.2	21.0	27.9	34.8	41.5	48.2	54.6	80.2
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	55.5	63.0	92.8
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	63.5	72.4	107
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	72.1	82.2	122
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	96.9	111	164
5.00	4.39	16.7	33.1	51.5	70.3	89.2	108	126	145	216

DESIGN CHART 12 (c)

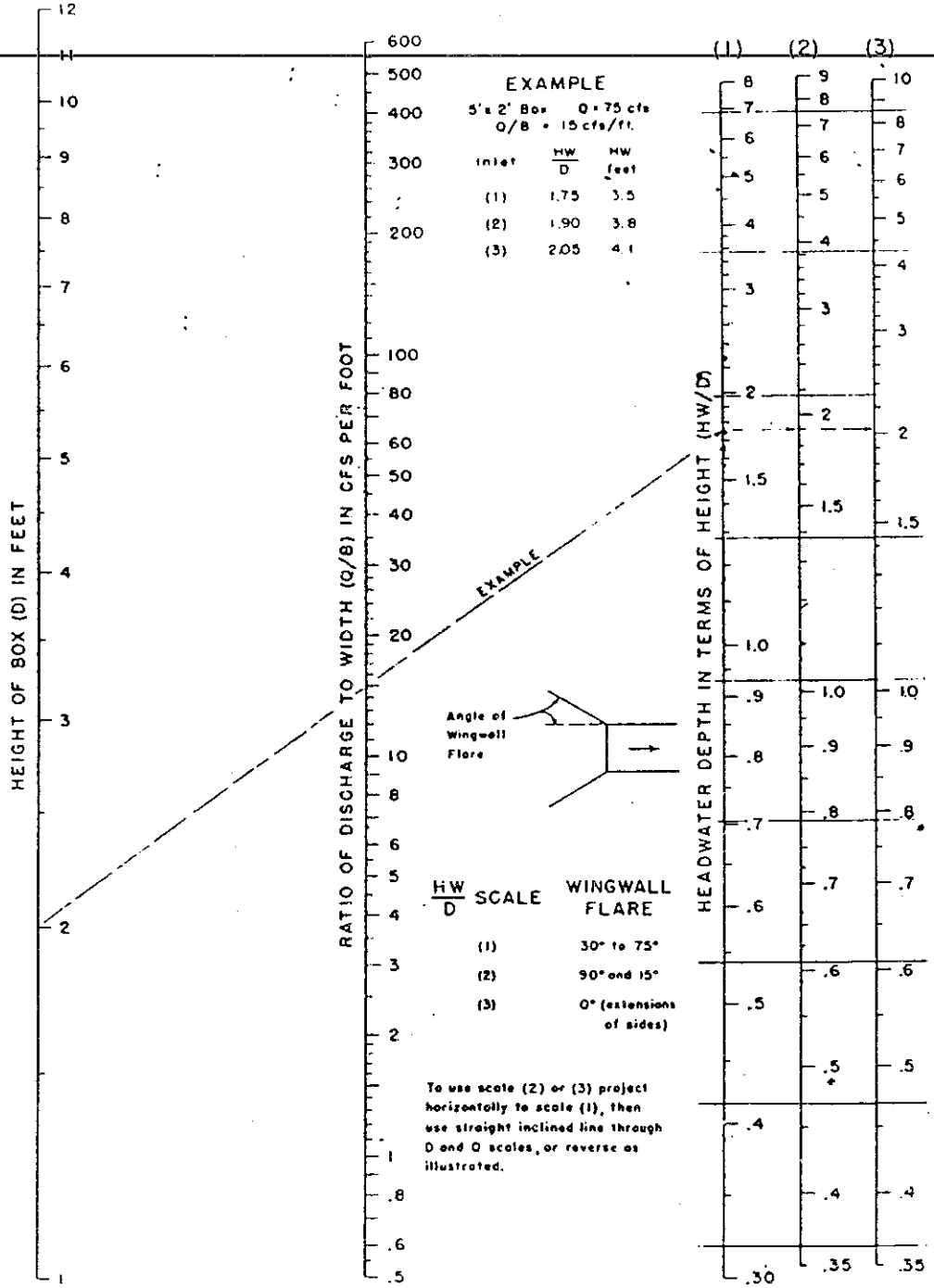
Table III-2.—*Uniform flow in circular sections flowing partly full.* From Reference III-3.

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius

Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1706	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

DESIGN CHART 13



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN 1963

CHART 14. DESIGN MONOGRAPH - FLOW THROUGH CULVERTS

Taken from : Hydraulic Charts for the Selection of Highway Culverts.

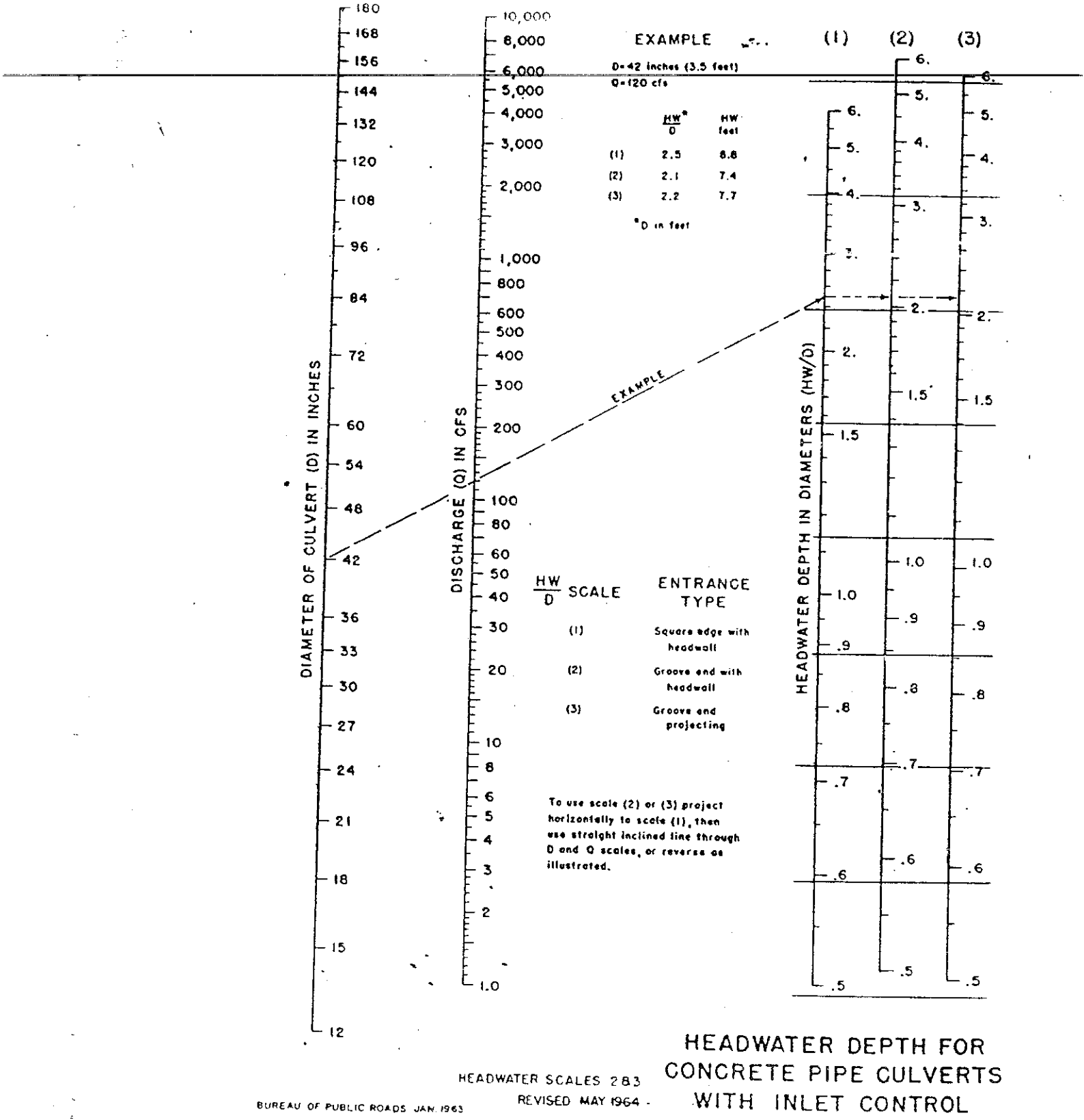
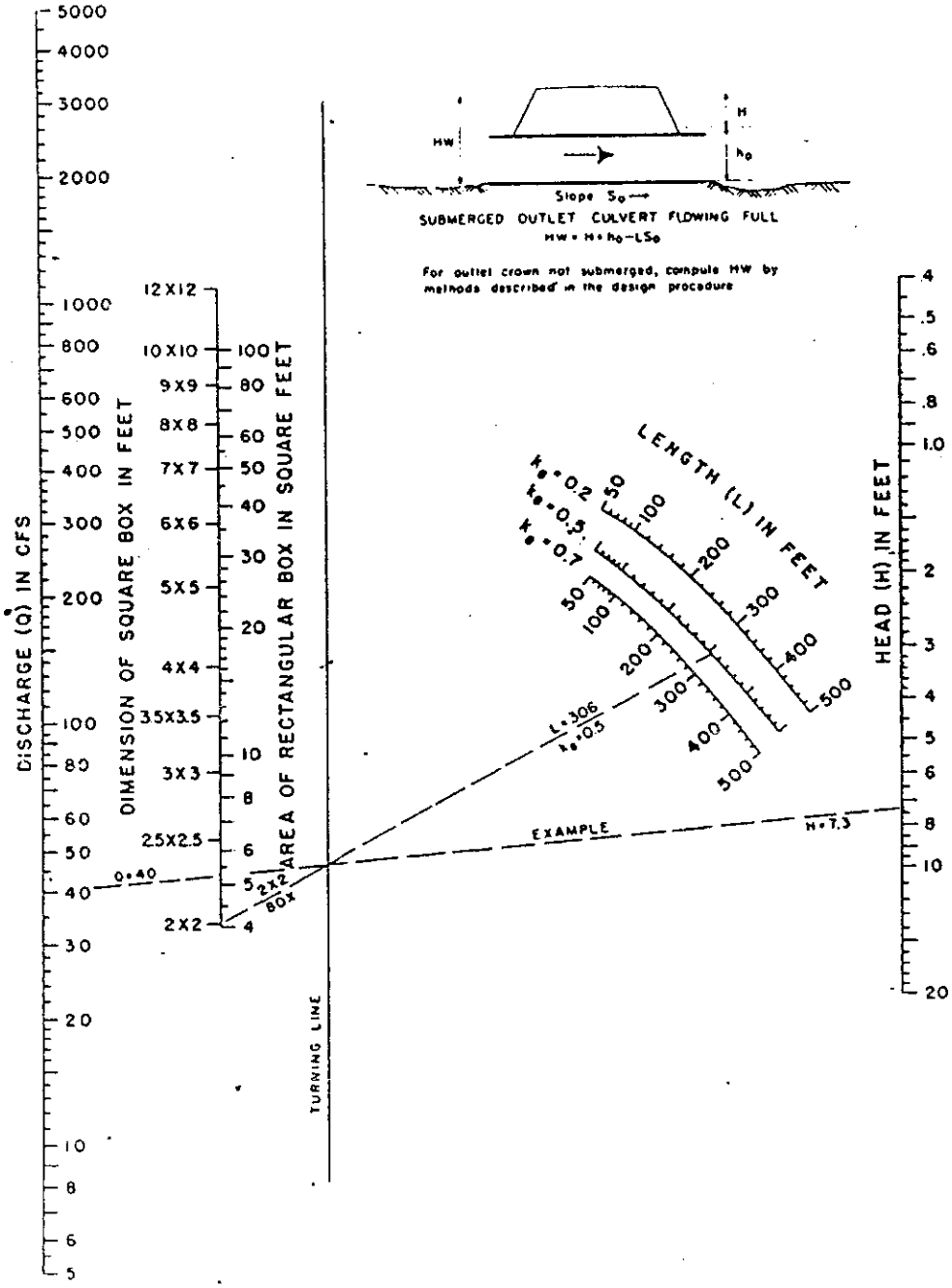


CHART 15. DESIGN MONOGRAPH - FLOW THROUGH CULVERTS

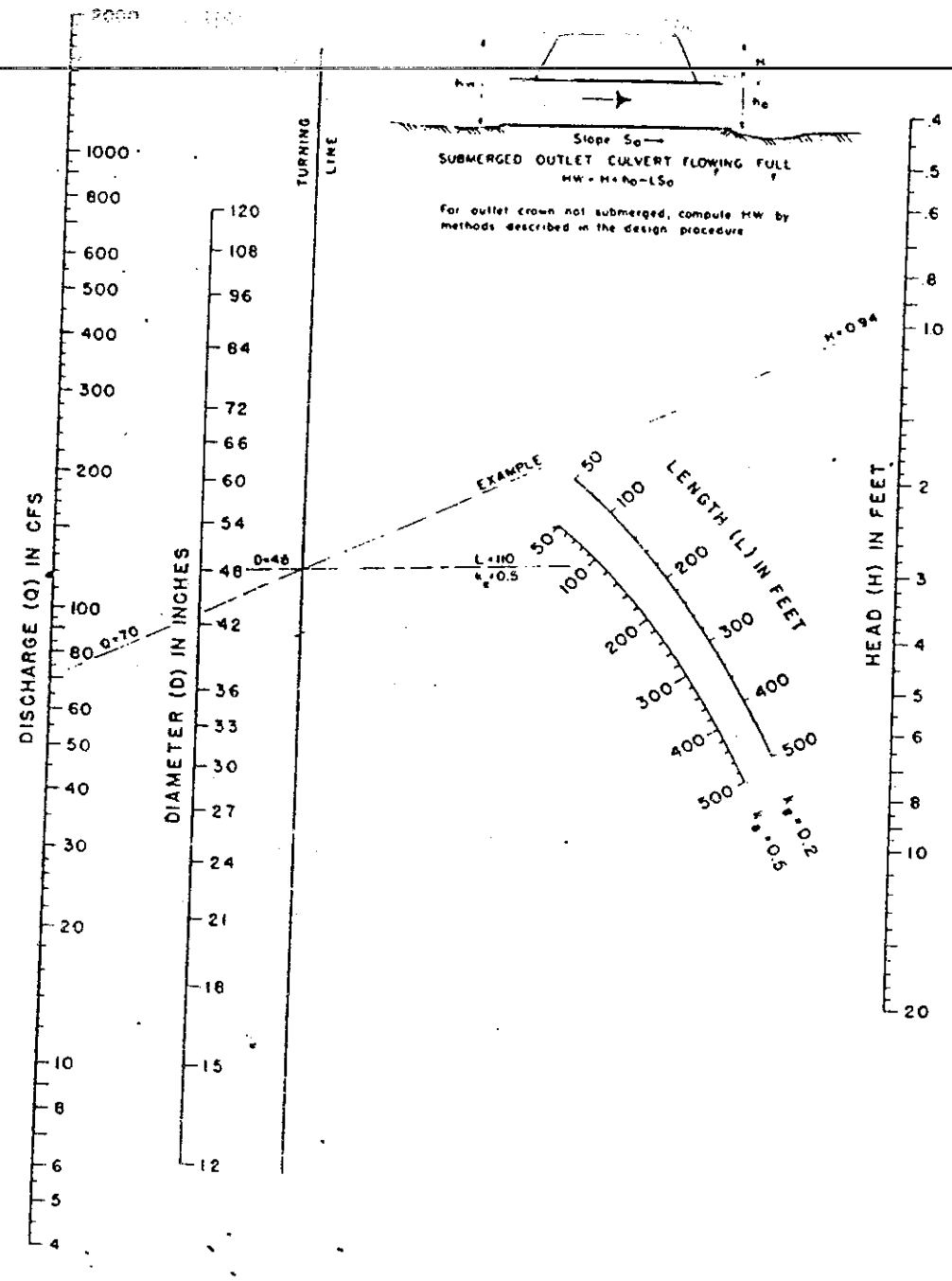
Taken from : Hydraulic Charts for the Selection of Highway Culverts.



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

CHART 16. DESIGN MONOGRAPH - FLOW THROUGH CULVERTS.

Taken from : Hydraulic Charts for the
Selection of Highway Culverts.



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

CHART 17. DESIGN MONOGRAPH - FLOW THROUGH CULVERTS.
Taken from : Hydraulic Charts for the
Selection of Highway Culverts.

Appendix A. -TABLES

Table 1.--Manning roughness coefficients, n [1]

	Manning's n range [2]
I. Closed conduits:	
A. Concrete pipe	0.011–0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2⅔ by ½-in. corrugation (riveted pipe): [3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021–0.018
(2) Flow 0.8 depth	0.021–0.016
(3) Flow 0.6 depth	0.019–0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012–0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009–0.011
F. Brick	0.014–0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015–0.017
2. Wood forms, smooth	0.012–0.014
3. Steel forms	0.012–0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017–0.022
2. Natural floor	0.019–0.025
I. Laminated treated wood	0.015–0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined [4] (straight alinement): [5]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013–0.017
2. Trowel finish	0.012–0.014
3. Float finish	0.013–0.015
4. Float finish, some gravel on bottom	0.015–0.017
5. Gunite, good section	0.016–0.019
6. Gunite, wavy section	0.018–0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015–0.017
2. Random stone in mortar	0.017–0.020
3. Cement rubble masonry	0.020–0.025
4. Cement rubble masonry, plastered	0.016–0.020
5. Dry rubble (riprap)	0.020–0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017–0.020
2. Random stone in mortar	0.020–0.023
3. Dry rubble (riprap)	0.023–0.033
D. Brick	0.014–0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011–0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017–0.020
2. Irregular section	0.022–0.027
III. Open channels, excavated [4] (straight alinement,[5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In gravelly soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides clean, gravel bottom	0.025–0.030
5. Sides clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high stage	0.10–0.14
IV. Highway channels and swales with maintained vegetation [6] (values shown are for velocities of 2 and 6 f.p.s.):	Manning's n range [2]
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07–0.045
b. Length 4–6 inches	0.09–0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18–0.09
b. Length about 24 inches	0.30–0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14–0.08
b. Length about 24 inches	0.25–0.13
B. Depth of flow 0.7–1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05–0.035
b. Length 4 to 6 inches	0.06–0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12–0.07
b. Length about 24 inches	0.20–0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10–0.06
b. Length about 24 inches	0.17–0.09
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.017
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of n by	*0.004*
VI. Natural stream channels: [5]	
A. Minor streams [8] (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	*0.01–0.01*
2. Irregular sections, with pools, slight channel meander; increase values given in 1a–e about	*0.01–0.01*
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas:	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees: [10]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush: [10]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100–150 per acre:	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited [8] if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028–0.033

Footnotes to table 1 appear at the top of page 101

Footnotes to Table 1

[1] Estimates are by Bureau of Public Roads unless otherwise noted.

[2] Ranges indicated for closed conduits and for open channels, lined or excavated, are for good to fair construction (unless otherwise stated). For good quality construction, use larger values of *n*.

[3] *Friction Factors in Corrugated Metal Pipe*, by M. J. Webster and L. R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HY9, Sept. 1959, Paper No. 2148, pp. 35–67.

[4] For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select *n* by comparison of the specific conditions with the channels tested:

Flow of Water in Irrigation and Similar Channels, by F. C. Scobey, Division of Irrigation, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 652, Feb. 1939; and

Flow of Water in Drainage Channels, by C. E. Ramser, Division of Agricultural Engineering, Bureau of Public Roads, U.S. Department of Agriculture, Tech. Bull. No. 129, Nov. 1929.

[5] With channel of an alinement other than straight, loss of head by resistance forces will be increased. A small increase in value of *n* may be made, to allow for the additional loss of energy.

[6] *Handbook of Channel Design for Soil and Water Conservation*, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station; published by the Soil Conservation Service, U.S. Department of Agriculture, Publ. No. SCS–TP–61, Mar. 1947, rev. June 1954.

[7] *Flow of Water in Channels Protected by Vegetative Linings*, by W. O. Ree and V. J. Palmer, Division of Drainage and Water Control, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 967, Feb. 1949.

[8] For calculation of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey, to obtain data regarding values of *n* applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of *n* tabulated have been derived from data reported by C. E. Ramser (see footnote 4) and from other incomplete data.

[9] The tentative values of *n* cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of *n* must be increased to provide for the additional loss of energy caused by bends. The increase may be in the range of perhaps 3 to 15 percent.

[10] The presence of foliage on trees and brush under flood stage will materially increase the value of *n*. Therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. For trees in channel or on banks, and for brush on banks where submergence of branches increases with depth of flow, *n* will increase with rising stage.

Table 2.—Permissible velocities for channels with erodible linings, based on uniform flow in continuously wet, aged channels [1]

Soil type or lining (earth; no vegetation)	Maximum permissible velocities for—		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	F.p.s.	*F.p.s.*	*F.p.s.*
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and boulders	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

[1] As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

Table 3.—Permissible velocities for channels lined with uniform stands of various grass covers, well maintained [1] [2]

Cover	Slope range	Permissible velocity on—	
		Erosion resistant soils	Easily eroded soils
	Percent	*F.p.s.*	*F.p.s.*
Bermudagrass	0–5	8	6
	5–10	7	5
	Over 10	6	4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0–5	7	5
	5–10	6	4
	Over 10	5	3
Grass mixture	0–5	5	4
	5–10	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0–5	3.5	2.5
Common lespedeza [3] Sudangrass [3]	[4] 0–5	3.5	2.5

[1] From *Handbook of Channel Design for Soil and Water Conservation* (see footnote 6, table 1, above).

[2] Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.

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

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

Table 4.—Factors for adjustment of discharge to allow for increased resistance caused by friction against the top of a closed rectangular conduit [1]

D/b	Factor
1.00	1.21
.80	1.24
.75	1.25
.667	1.27
.60	1.28
.50	1.31
.40	1.34

[1] Interpolation may be made. See derivation of factors on p. 8.

Table 5.—Guide to selection of retardance curve

Average length of vegetation	Retardance curve for—	
	Good stand	Fair stand
6–10 inches	C	D
2–6 inches	D	D

CHAPTER IV

STRUCTURAL DESIGN

A.H. BHUIYAN

Contents

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1.0	General Design Requirements	1
2.0	Computation of Design loads and pressures	3
3.0	Structural elements of a regulator	7
4.0	Loading condition and design concepts	8
	4.1 Stability analysis	8
	4.2 Structural design	9
5.0	Example on stability and structural design of Mrigi Khal regulator.	21

STABILITY & STRUCTURAL DESIGN OF R.C.C. REGUALTOR/SLUICE.

1.0 General Design Requirements :

Any structure shall be so designed that it shall remain stable against external loads & pressure and different members of the structure are strong enough to resist external load, forces and internal stresses. Considering this the structural design of regulator shall be undertaken after stability analysis.

a) Stability - The stability design of the structure should fulfil the following four requirements :

- It must be safe against overturning at any horizontal plane
- It must be stable against sliding or shearing force on any horizontal plane, including planes in foundation materials
- It must be stable against water uplift
- It shall not overstress the underlying foundation material or soil.

The factor of safety against the above stability requirements shall be taken as follows :

Overturning	:	Normal Case	: 1.50
		Extreme Case	: 1.30
Sliding	:	Normal	: 1.50
		Extreme	: 1.30
Uplift	:	1.10	
Bearing	:	Normal	: 2
		Extreme	: 3

b) Structural - An efficient design of a reinforced concrete structure requires that the materials be economically selected, proportioned and arranged to carry the required loads & pressure without developing stresses which are in excess of the allowable working stresses. Corrrect estimation of all types of loads should be made before taking up the design proper.

The structural design requirements may be summarised as below :

- The sectional area of concrete should be sufficient to take care for the compressire stresses caused by bending or direct load.

- Sufficient longitudinal reinforcements should be provided to take care for the tensile stresses caused by bending or direct load.

- Provision of sufficient anchorage for all reinforcement should be provided so that all bond stresses and anchorage requirements are met.

- Sufficient shear reinforcement should be provided to keep the shearing stresses within permissible limits.

The allowable working or permissible stresses to meet the above design requirements may be taken as follows for the type of material and quality control during constructioin as are usually available.

Flexural Stress :

Concrete - $f'c = 2500$ psi, $fc = 0.4\ f'c$.
Steel - $fs = 18000$ psi.

Bond Stress :

Top bar - $Ut = (1.7 \sqrt{f'c})/D < 160$ psi

Bottom bar - $Ub = (2.4 \sqrt{f'c})/D < 160$ psi

Shear Stress :

Flexural - $1.1 \sqrt{f'c}$ (max $5 \sqrt{f'c}$ with shear reinforcement)

Punching - $2 \sqrt{f'c}$.

Bearing Stress :

On full area - $0.25 \sqrt{f'c}$

On one third area or less - $0.375 \sqrt{f'c}$

2.0 Computation of Design loads & pressures :

The usual loads & pressures which may act on the regulator/sluice structure shall be correctly listed and computed as below :

i) Dead loads - All permanent & temporary dead loads shall be calculated with the following unit weights -

R.C. Concrete work - 150 lb/cft.

Plain Concrete work - 145 "

Brick work - 120 "

Earth work -

dry - 100 lb/cft
moist - 110 "
saturated - 120 "
submerged - 57.5 "

Water -

Fresh water : 62.4 lb/cft
Sea water : 64 "

Timber : 50 lb/cft

Steel : 490 "

Wearing Course : 150 "

ii) Live Loads :

a) Human Occupacy - The live load for human occupacy shall be taken as follows :

- Operating deck : 150 lb/sft (with stoplog operation)
- Regulator crest : 100 lb/sft (for pedestrian movement)

b) Truck Load - The truck loading shall be selected on the basis of location of structure in relation to the road or embankment it crosses. The following type of truck loading may be selected :

Village road	- H_{10}
Feeder road	- H_{10}
District council road	- H_{15}
High way	- H_{20} S_{16}

- The distribution of truck loading on concrete slab and filled earth shall be as per AASTO design standard code of practice (Plate No. 1)
- The equivalent surcharge load for truck loading shall be a max. of 3'-0" earth load.

iii) Pressures and forces : The various pressure & forces which may act on the regulator/sluice shall be computed as follows :

a) Water pressure :

Horizontal - At any depth, $p = 1/2\ \gamma_w h$

Total pressure, $P = 1/2\ \gamma_w H^2$

Vertical (uplift) - The uplift pressure under the regualtor base slab shall be calculated as per Khosla's or Lane's theory.

b) Earth pressure : The usual active earth pressure which may act on the regulator shall be computed as follows :

Vertical wall with Horizontal Backfill (Plate No. 2)

At any depth $P = Ca\,\gamma\,h$

Total pressure, $P = 1/2\ Ca\ \gamma\ H^2$

$$Ca = \frac{1 - \sin \phi}{1 + \sin \phi}$$

Inclined Wall with Horizontal Backfill (Plate No. 2)

At any depth $P_n = Ca\,\gamma\,h$, P_n = Pressure normal to the back of wall

$P_h = P_n \cos (90^o - B)$, p_h = Horizontal component of pressure

$P_v = P_n \sin (90^o - B)$, P_v = Vertical component of pressure

$$Ca = \left[\frac{\sin (B - \phi)}{(n + 1) \sin B}\right]^2 \times \frac{1}{\sin B} \text{ where}$$

$$n = \frac{\sin \phi}{\sin B}$$

B = Angle between wall & Horizontal

ϕ = Angle of internal friction of soil.

Vertical Wall with inclined Backfill

At any depth, $P = Ca\,\gamma\,h$, where

$$Ca = \cos B \frac{\cos B - \sqrt{\cos^2 B - \cos^2 \phi}}{\cos B + \sqrt{\cos^2 B - \cos^2 \phi}}$$

where, B = angle of inclination of backfill with horizontal

Earth pressure with Saturated Backfill (Plate No. 2)

When the backfill soil will be saturated or submerged by water behind it, the computed pressure shall be based on two components -

Earth pressure due to submerged soil &
Presure due to pore water.

The pressures behind a wall of partly saturated backfill may be computed from the equations as shown in Plate No. 2.

Earth pressure due to uniform surcharge

Earth pressure due to uniform surcharge load may be computed as below :

P surcharge = Ca W = Ca γ h' where, W = uniform surcharge and
h' = equivalent depth of soil

c) Wind, Wave & Earthquake pressures : This pressures need not to be considered for design of minor regulators. However, these pressures should be considered for major & costly structures as and when applicable.

iv) Intensity of pressure on foundation : Under the application of different loads & pressures and under certain loading condition, the intensity of pressure on foundation base shall be computed as below :

When Resultant is in middle third

$$P_1/P_2 = \frac{V}{L}\left(1 \pm \frac{6e}{L}\right)$$

When Resultant is at edge of middle third

$$P_1 = \frac{2 \Sigma V}{L}, \quad P_2 = 0$$

When Resultant is outside middle third

$$P = \frac{2 \Sigma V}{3a}, \quad a = L/3$$

3.0 Structural Elements of a Regulator :

The type of Regulator or Sluice structure normally built in this country consists of either a pipe or a Box Conduit with inlet & outlet transitions. Pipe sluice is selected when a small amount of discharge is to be handled from a small catchment area. The pipe is a locally manufactured R.C.C. construction with maximum internal diameter of 3'-0". The Box sluices are usually made in standard sizes of 5'x6' and 4'x5' in multi-box construction for handling discharge of large drainage areas. They are usually constructed with two central joints separating the transition part with main conduit part of the structure. In large structure, additional joints are provided along the longitudinal direction of wingwall to separate the wingwall and apron slab.

The various structural elements of a regulator/sluice of standard vent size with earthfill construction need to be designed are as follows :-

- Conduit or Barrel
- Abutment wall as extended part of Conduit
- Base slab as extended part of Conduit's base slab
- Pier
- Head wall
- Wingwall (either monolithic to apron slab or usual retaining wall construction)

- Return Wall (either monolithic to R.C.C. Caisson cutoff wall or usual retaining wall construction over sheet pile cutoff wall)

- Apron slab (Simple gravity slab with minimum reinforcement or R.C.C. construction monolithic to wingwall.

- Operating platform or deck slab

- Safety railing

- Cutoff wall either as R.C.C. Caisson or Sheet pile.

4.0 Loading conditions and design concepts :

4.1 Stability Analysis : The stability analysis shall be made for the central part of the structure, Wingwalls & Return walls for checking the safety against overturning, sliding, uplift and bearing pressure. Wingwalls which are built with apron slab in monolithic construction shall be checked for uplift stability only. The return wall constructed on caisson cutoff wall shall be checked for stability as a whole. For the stability analysis, imposed loads & pressures shall require to be computed separately and combined in such way that they produce critical loading at various stage of construction, operation or maintenance of the structure. Usually, all stability analysis shall be done based on following three conditions :

- Construction condition (During or just after construction)
- Operation Condition (Gate closed or open)
- Maintenance Condition

The various loading conditions which shall be considered for the stability analysis of central part of the struture & Wingwalls/Returnwalls are given in Plate No. 3 and Plate No. 4 respectively.

4.2 Structural Design : The various members of the Regulator shall be designed for the maximum bending moment, direct force, shear & bond which arise under critical condition of loading. The various loading conditions & design concept for each of the structural elements are given below :

i) Box Conduit.

Loading condition - The various loadings conditions for structural design are given in Plate No. 4.

Design Concept - The method of moment distribution shall be used to analysis the frame. The design span is the centre line of member support. Under each loading condition computed moments and shears are tabulated separately and each member is designed with maximum moment and shear.

ii) Wingwall/Returnwall (as conventional Retaining wall)

Loading conditions : As in Plate No. 3.

Design Concept

a) Cantilever Retaining Wall :

Stem - It shall be designed as Cantilever slab with full maximum load imposed on the back of wall. The reinforcement shall be placed at earth face with gradual curtailments.

Toe slab - It shall be designed as cantilever slab with net foundation pressure acting upward. The main reinforcement shall be placed at bottom.

Heel slab - As cantilever slab with net foundation pressure acting downwards and main reinforcement at top.

b) Counterfort retaining Wall :

General

For walls above 20 ft. height, the cantilever type of retaining wall requires a greater stem thickness. In such high walls, the cost may be reduced by placing counterforts at suitable intervals on the back side of the wall. The stem behaves as a vertical slab, extending continuously over the counterforts and is loaded by the horizontal thrust. The function of the counterforts is to tie the vertical wall slab to the base of the wall.

Design of Stem

The stem is designed as a continuous vertical slab, spanning over the counterforts. For the end spans the negative moment is taken as $\frac{WL^2}{10}$ and for the inner spans the same is taken as $\frac{WL^2}{12}$ where, L is spacing of counterforts.

The maximum bending moment in the stem wall occurs at its junction with the base slab and decreases proportionately towards top of the stem. The thickness of the stem, however, is often kept uniform throughout the height of the wall, as calculated from the maximum bending moment and reinforcement is curtailed towards top of the stem.

Design of Base slab

The heel is designed as a horizontal slab, supporting the counterforts. It is subjected to bending stresses induced by the backfill and the subsoil reactions to the loading. The toe is designed exactly in the same manner as used for cantilever retaining walls.

Design of counterforts.

The earth pressure acting on the stem is transferred to the counterforts. The total load, to which each counterfort is subjected, is the total earth pressure per foot at any height of stem multiplied by the spacing of the counterforts. The back of the counterfort is in tension and is designed as a cantilever beam of non-uniform depth, fixed at the top of the base slab. The tensile reinforcement is provided along the sloping back and must be sufficiently anchored at both the ends.

Since the stem has a tendency to pull away from the counterfort, horizontal ties must be provided to tie the stem to the counterforts. Similarly, as the counterforts have also a tendency of pulling away from the base slab, the two must be tied by means of vertical ties.

In the above discussion of the wall design the effect of bottom fixity has been neglected. In this regard, Huntington proposed the following modification to the design load case. (Plate No. 6).

- The face slab is designed as a continous vertical slab to resist the lateral pressure as shown in Plate No. 6
- The slab should be designed to resist the negative and positive bending moments shown in the same plate.
- The heel slab is subjected to an equivalent downward force in addition to the forces. This equivalent forces is evaluated to include the effect of bending moment of the toe slab and assumed to be distributed parabolically with a maximum value Wa at top of the heel.

The total of the parabolic load is equal to 2/3 Wha and the centroid is located 5a/8 from the back of the face slab. This equivalent uniform load should be applied to the entire length of the heel slab, from counterfort to counterfort, in combination with other forces, for computing the bending moment and shear forces along the length of the wall.

iii) Wingwall and Apron slab (in monolithic construction)

Loading Conditions - As in Plate No. 3.

Design Concept.

Stem of wingwall is designed with maximum imposed horizontal load from the various loading conditions.

Base slab is designed with end moments from wingwall and foundation pressure. Top reinforcements shall be provided with loading condition-1. Maximum bottom reinforcements shall be provided in the bottom of slab from the other loading conditions whichever is critical.

iv) Gravity apron slab - This shall not require to be usually designed with moments or shear, but shall be provided with minimum reinforcement as follows :

Top face	= 0.0025 bt	t = 15" where t > 15"
Bottom face	= 0.0020 bt	b = 12"

v) Abutment wall as extended part of Conduit.

Loading condition : This shall be designed with maximum imposed load at back face.

Design concept : As the abutment is fixed at base slab and also with conduit sidewall & headwall, this wall shall be analysed with slab of two ends fixed and two ends free with triangular loading. The table as in Plate No. 9 may be used for analysis. In fact, the reinforcement shall be placed at both faces and both directions.

vi) Extended part of base slab.

Loading Condition : The maximum foundation pressure shall be used to design the slab.

Design Concept : The base slab shall be designed as continuous slab over support on piers & abutments with max. +ve & -ve moment of $1/10\ WL^2$ where L = clear span, &

W = foundation pressure. The reinforcement shall be placed at both top and bottom.

vii) Pier.

The pier shall be designed as cantilever slab fixed at base slab and headwall with traingualr loading due to water pressure on one side only. This condition may arise during maintenance work when the one bay need to be closed by stop logs with other bays with flowing water.

viii) Head wall.

The headwall may be designed as continuous slab spanning over the pier & abutment supports with maximum earth pressure at its back and design moment of + $1/10\ WL^2$.

ix) Operating Platform.

The operating platform shall be designed for full DL + LL + hoisting load. The slab shall be designed as continuous slab over pier & abutment supports with Maximum positive and negative moment of $1/10\ WL^2$.

x) Railing.

The railing may be of R.C.C. or G.I. pipe. The design of the same may be standarised with minimum reinforcement.

DISTRIBUTION OF WHEEL LOADS

a. Wheels resting directly on slab

(1) Spans under 12 feet

$E = 1.75S + 3.2$, where E = width of slab over which a wheel load is to be distributed and S = span length.

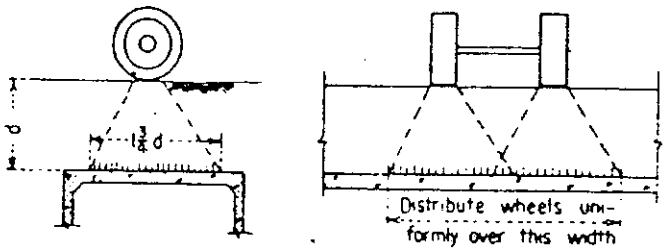
(2) Spans over 12 feet

$$E = \frac{10N + W}{4N}$$

where N = total number of lines (@ 10') permissible on the bridge and W = width of roadway

b. Wheel loads transmitted thru fills.

(1) When the depth of fill is 2 feet or more, concentrated loads shall be considered as uniformly distributed over a square, the sides of which are equal to $1\frac{3}{4}$ times the depth of fill. When such areas from several concentrations overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas.



(2) For single spans the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between end supports or abutments.

(3) In no case shall the stresses for live load as determined by the above be greater than those obtained where the wheel is applied directly on the structure.

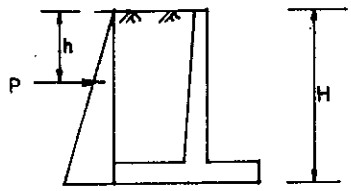
c. In no case shall the total width of distribution exceed the total width of supporting slab.

C. IMPACT

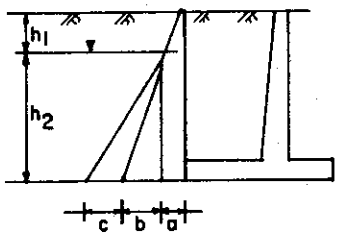
For fills	0'	to	1'-0"	Impact = 30 %
" "	1'-1"	"	2'-0"	" = 20 %
" "	2'-1"	"	2'-11"	" = 10 %
" "	3'-0" and over			" = 0 %

PLATE NO. 1

Earth pressure with horizontal backfill



Earth pressure with saturated backfill (partly)



a = Ca γ moist h_1

b = Ca γ sub. h_2

c = $\gamma_w h_2$

Earth pressure on inclined wall with horizontal backfill

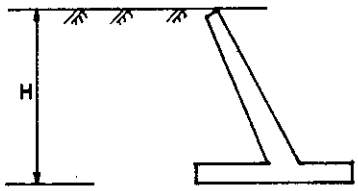
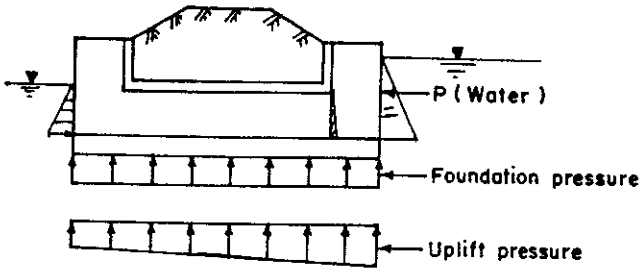
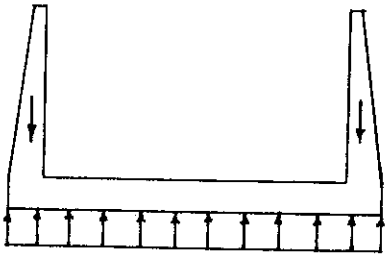


PLATE NO. 2 EARTH PRESSURE ON RETAINING WALLS

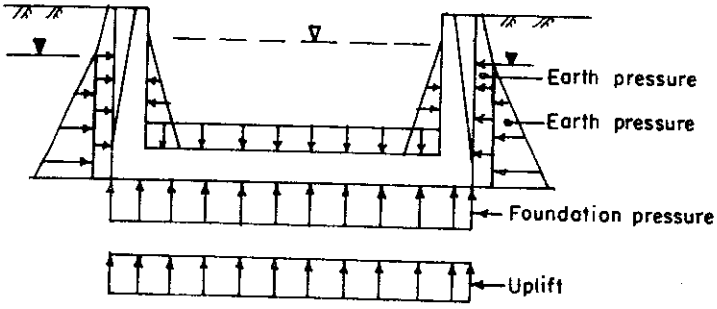
Loading condition for the stability analysis of main part of the Regulator



LOADING CONDITION - I
(Just after Construction, No Backfill)



LOADING CONDITION - 2
(During operation - Gates open)



LOADING CONDITION - 3
(During operation - Gates closed)

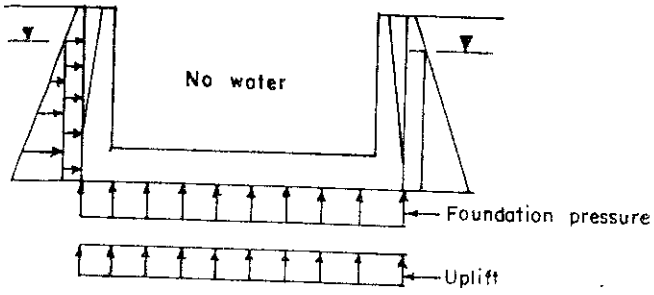
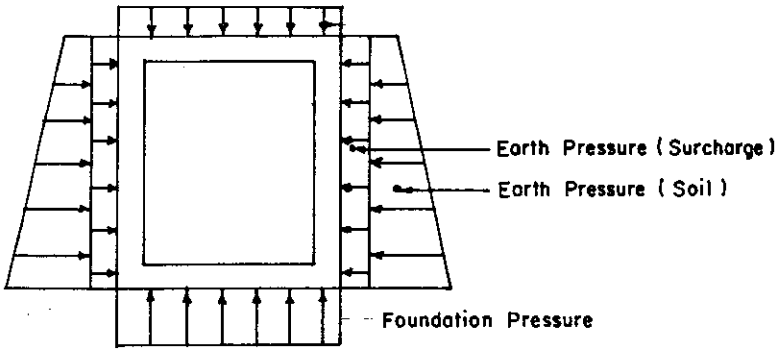


PLATE NO. 3 LOADING CONDITION FOR THE DESIGN OF APRON SLAB & WINGWALL IN MONOLITHIC CONSTRUCTION

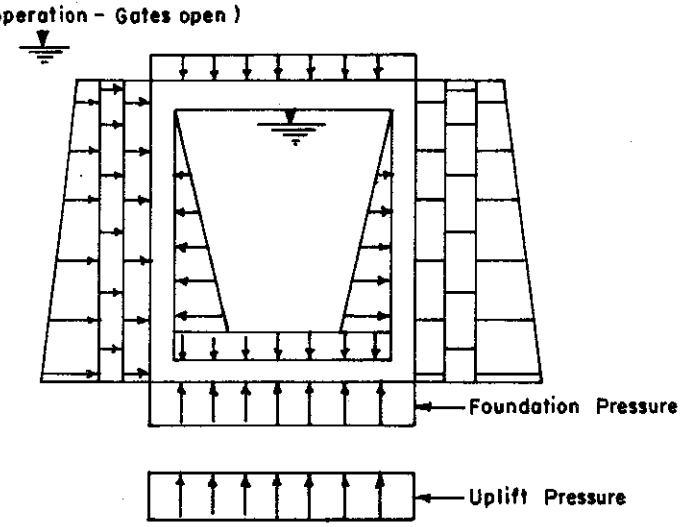
LOADING CONDITION - 1

(After Construction)



LOADING CONDITION - 2

(During operation - Gates open)



LOADING CONDITION - 3

(During operation - Gates - closed)

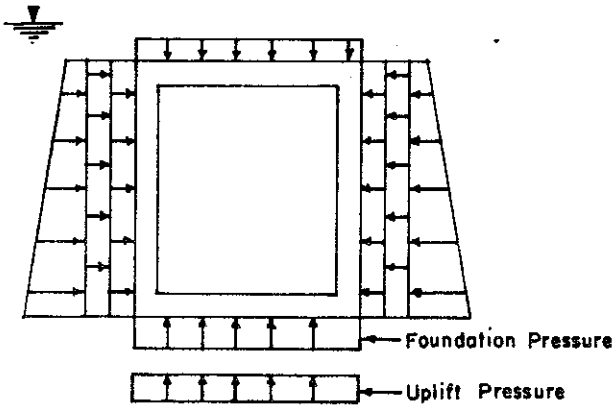
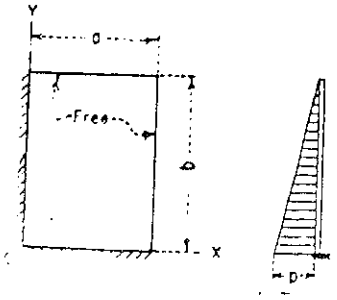


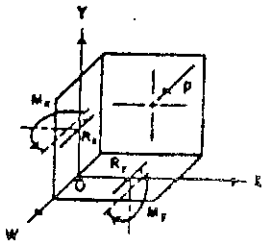
PLATE NO. 4 LOADING CONDITION FOR THE DESIGN OF BARREL

			M_x						M_y					
	y/b	R_x \ x/a	0	0.2	0.4	0.6	0.8	1.0	0	0.2	0.4	0.6	0.8	1.0
a/b = 1/8	1.0	+.0275	+.0007	+.0006	+.0004	+.0002	+.0001	0	0	0	0	0	0	0
	0.8	+.0252	+.0017	+.0011	+.0006	+.0003	+.0001	0	+.0003	+.0002	+.0001	+.0001	+.0001	+.0001
	0.6	+.0501	+.0031	+.0020	+.0011	+.0005	+.0001	0	+.0006	+.0004	+.0002	+.0001	+.0000	-.0000
	0.4	+.0755	+.0046	+.0029	+.0016	+.0007	+.0001	0	+.0009	+.0006	+.0003	+.0001	-.0000	-.0001
	0.2	+.0964	+.0052	+.0031	+.0015	+.0005	+.0000	0	+.0010	+.0006	+.0002	-.0001	-.0003	-.0004
	0	+.0056	0	+.0001	+.0024	+.0008	+.0001	0	0	+.0007	+.0021	+.0038	+.0056	+.0072
		R_y \ R_y	+.0058	+.0008	+.0510	+.0996	+.1819	+.2706						
a/b = 1/4	1.0	+.0070	+.004	+.0015	+.0026	+.0016	+.0007	0	0	0	0	0	0	0
	0.8	+.055	+.0078	+.0052	+.0031	+.0015	+.0005	0	+.0016	+.0011	+.0007	+.0005	+.0004	+.0004
	0.6	+.1026	+.0119	+.0074	+.0039	+.0015	+.0003	0	+.0024	+.0014	+.0007	+.0001	-.0004	-.0006
	0.4	+.1513	+.0150	+.0085	+.0040	+.0011	-.0002	0	+.0030	+.0015	+.0002	-.0009	-.0017	-.0022
	0.2	+.1475	+.0122	+.0060	+.0022	+.0001	-.0006	0	+.0024	+.0008	-.0006	-.0017	-.0026	-.0032
	0	-.0598	0	+.0005	+.0014	+.0024	+.0035	0	0	+.0024	+.0069	+.0121	+.0175	+.0221
		R_x \ R_x	-.0598	+.0133	+.1020	+.1780	+.3009	+.4232						
a/b = 3/8	1.0	+.0140	+.0115	+.0095	+.0066	+.0037	+.0015	0	0	0	0	0	0	0
	0.8	+.0970	+.0180	+.0116	+.0066	+.0029	+.0008	0	+.0036	+.0023	+.0012	+.0004	-.0000	-.0002
	0.6	+.1553	+.0233	+.0133	+.0062	+.0018	-.0002	0	+.0047	+.0024	+.0003	-.0013	-.0026	-.0034
	0.4	+.2050	+.0246	+.0122	+.0045	+.0002	-.0012	0	+.0049	+.0017	-.0012	-.0037	-.0055	-.0067
	0.2	+.1717	+.0157	+.0063	+.0016	-.0005	-.0009	0	+.0030	+.0006	-.0014	-.0028	-.0037	-.0043
	0	+.0044	0	+.0009	+.0024	+.0040	+.0057	0	0	+.0048	+.0119	+.0202	+.0286	+.0351
		R_y \ R_y	+.0144	+.0309	+.1238	+.1911	+.3079	+.4185						
a/b = 1/2	1.0	+.0166	+.0271	+.0167	+.0100	+.0046	+.0013	0	0	0	0	0	0	0
	0.8	+.1402	+.0295	+.0177	+.0090	+.0032	+.0003	0	+.0059	+.0032	+.0010	-.0007	-.0019	-.0026
	0.6	+.1983	+.0334	+.0174	+.0069	+.0009	-.0013	0	+.0067	+.0027	-.0010	-.0041	-.0064	-.0079
	0.4	+.23	+.0309	+.0135	+.0037	-.0011	-.0021	0	+.0062	+.0012	-.0032	-.0067	-.0092	-.0109
	0.2	+.1413	+.0162	+.0058	+.0010	-.0005	-.0004	0	+.0032	+.0004	-.0012	-.0018	-.0018	-.0017
	0	-.0079	0	+.0015	+.0039	+.0064	+.0089	0	0	+.0074	+.0193	+.0318	+.0443	+.0548
		R_x \ R_y	-.0079	+.0573	+.1665	+.2446	+.3698	+.4827						
a/b = 3/4	1.0	+.0974	+.0455	+.0269	+.0107	+.0008	-.0028	0	0	0	0	0	0	0
	0.8	+.1967	+.0487	+.0242	+.0084	-.0002	-.0027	0	+.0097	+.0040	-.0004	-.0039	-.0065	-.0087
	0.6	+.2303	+.0459	+.0195	+.0046	-.0023	-.0036	0	+.0092	+.0020	-.0044	-.0093	-.0128	-.0153
	0.4	+.2383	+.0360	+.0121	+.0010	-.0030	-.0029	0	+.0072	-.0003	-.0060	-.0096	-.0117	-.0132
	0.2	+.1193	+.0160	+.0042	+.0005	+.0006	+.0016	0	+.0032	+.0008	+.0019	+.0050	+.0083	+.0108
	0	-.0194	0	+.0029	+.0070	+.0110	+.0148	0	0	+.0147	+.0352	+.0548	+.0740	+.0894
		R_x \ R_y	-.0194	+.1105	+.2399	+.3236	+.4469	+.5505						
a/b = 1	1.0	+.1917	+.0662	+.0291	+.0056	-.0059	-.0077	0	0	0	0	0	0	0
	0.8	+.2481	+.0613	+.0243	+.0035	-.0056	-.0063	0	+.0123	+.0039	-.0019	-.0059	-.0089	-.0118
	0.6	+.2354	+.0518	+.0173	+.0004	-.0059	-.0054	0	+.0104	+.0009	-.0064	-.0112	-.0144	-.0172
	0.4	+.2269	+.0368	+.0092	-.0015	-.0041	-.0028	0	+.0074	-.0015	-.0062	-.0078	-.0080	-.0084
	0.2	+.1047	+.0150	+.0030	+.0007	+.0021	+.0040	0	+.0030	+.0022	+.0073	+.0148	+.0216	+.0268
	0	-.0224	0	+.0046	+.0103	+.0152	+.0197	0	0	+.0232	+.0515	+.0759	+.0987	+.1157
		R_y	-.0224	+.1598	+.2991	+.3794	+.4909	+.5586						



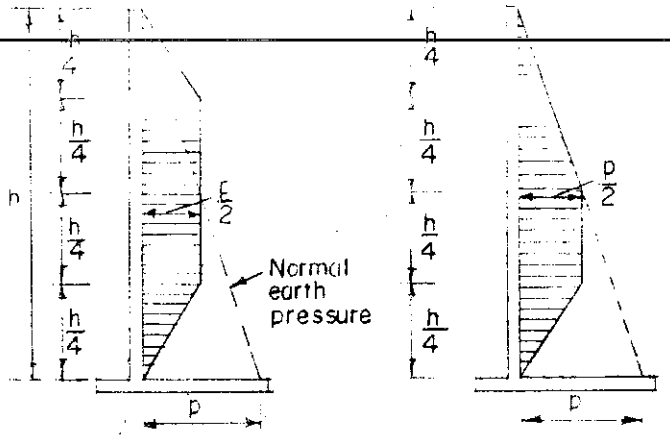
Moment = (Coefficient)(pb^2)

Reaction = (Coefficient)(pb)



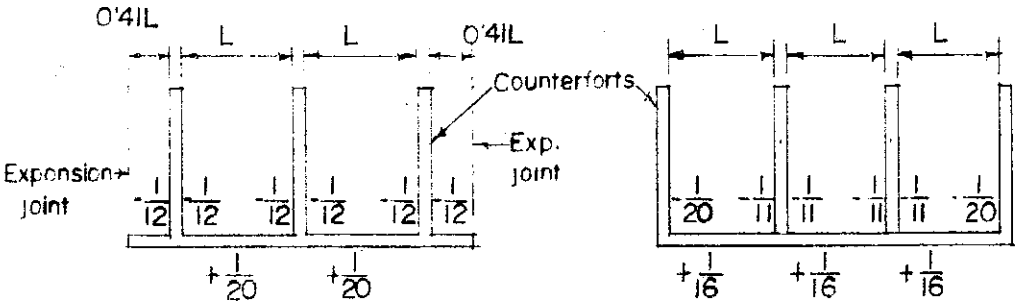
POSITIVE SIGN CONVENTION

PLATE NO.5-*Plate fixed along two adjacent edges, moment and reaction coefficients, Load uniformly varying la*



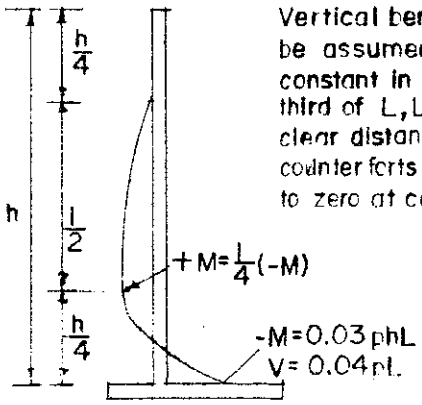
Pressure used for positive moment computation

Pressure used for negative moment computation



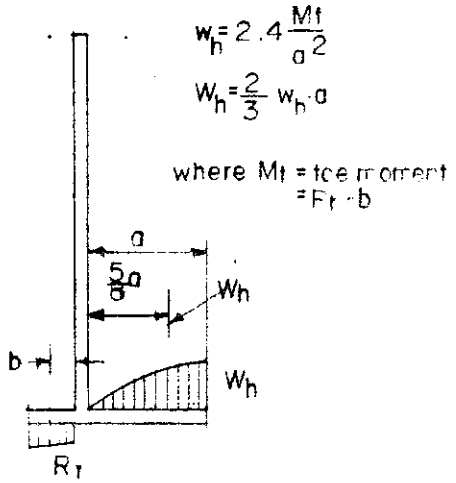
Moment coefficients used in conjunction with the above earth pressures

(a) Horizontal bending in the face slab



Vertical bending may be assumed to be constant in the middle third of L, L being the clear distance between counterforts and reduce to zero at counterforts

(b) Vertical bending in one foot strip of face slab



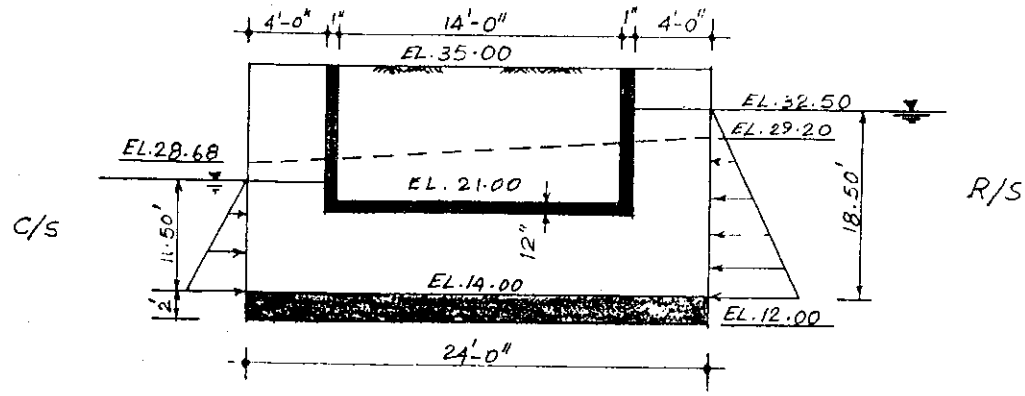
(c) Equivalent load (downward) on heel slab due to moment in the toe slab

PLATE NO6HUNTINGTON'S DESIGN PROCEDURE FOR COUNTERFORT WALL

5.0 Example on stability and structural design of Mrigi Khal regulator (For plan & section refer Plate N0. 7 and 8).

5.1 STABILITY ANALYSIS

CENTRAL PART OF THE STRUCTURE



(a) Detail Loads & Pressures:

(i) Concrete & Earthload

Base slab	: 2'x24'x20.50'x0.15	=147.60 k
Top slab	: 1'x16'x20.50'x0.15	= 49.20 k
Head wall	: 2x1'x14'x20.50'x0.15	= 86.10 k
Pier	: 4x1.25'x4'x21'x0.15	= 63.00 k
	: 2x1.25'x6'x16'x0.15	= 36.00 k
Abutment	: 2x1.50'x6'x24'x0.15	= 64.80 k
	: 4x0.5(1'+1.5')x15'x4'x0.15	= 45.00 k
		491.70 k
Earth(embankment):	19.50'x14'x14'x0.110	= 420.42 k
(Operating platform,Railing,Gate not included)		912.12 k

(ii) Water load

Upstream part	: 11.50'x4'x15'x0.0625	=	43.12 k
Downstream part	: 18.50'x4'x15'x0.0625	=	69.40 k
Central part	: 15'x16'x6'x0.0625	=	90.00 k
			202.52 k ↓

(iii) Uplift : 0.5(16.68+17.20)x24x20.50x0.0625 = 521 k ↑

(iv) Water Pressure

From C/s to R/s : $0.50 \times (11.50)^2 \times 0.0625 \times 15$ = 62 k →

From R/s to C/s : $0.50 \times (18.50)^2 \times 0.0625 \times$ = 160 k ←

(b) Summary of loads and Pressures:

(i)	Concrete & soil load	= 912 k
(ii)	Water load	= 203 k
	Total	=1115 k
(iii)	Uplift	= 521 k
(iv)	Net water pressure	= 160 k -62 k = 98 k

(c) Factor of safety

(i) Uplift = 1115/521 = 2.14 > 1.10 O.K.

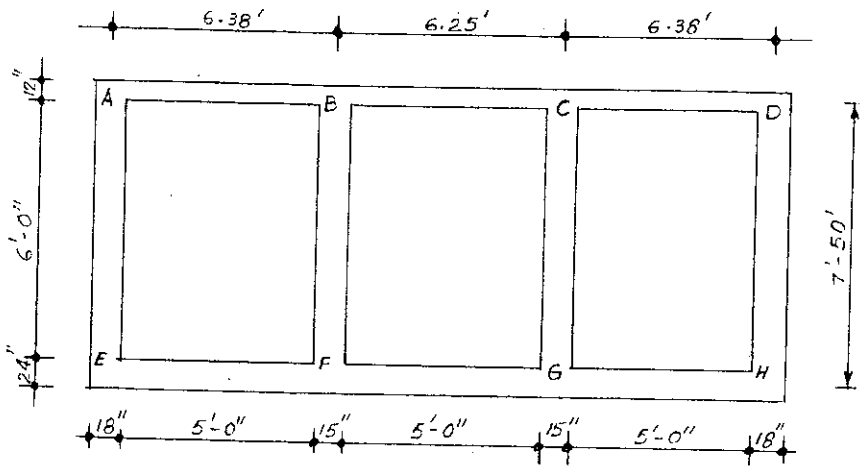
(ii) Sliding = $\frac{0.4(1115-521)}{98}$ = 2.42 > 1.5 O.K.

5.2 STRUCTURAL DESIGN

A. Central Part of the structure

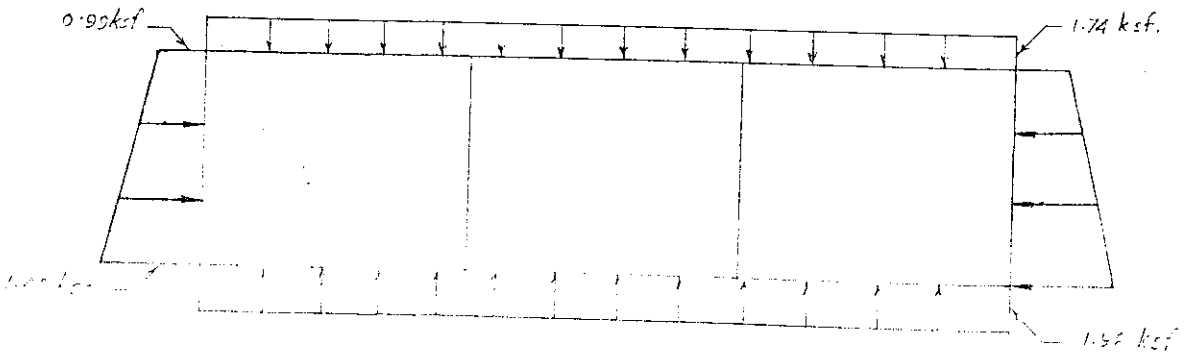
(i) Box Conduit (3 Vents - 5'-0"x6'-0" size)

Frame dimensions and Distribution Factors



Joint	Member	Length(inch)	t	t^3/L	Dist.Factor
A/D	AB/DC	76.56	12"	22.57	0.258
	AE/DH	90.00	18	64.80	0.742
B/C	BA/CD	76.56	12	22.57	0.273
	BC/CB	75.00	12	22.57	0.273
	BF/CG	90.00	15	37.50	0.454
E/H	EA/HD	90.00	18	64.80	0.264
	EF/HG	76.56	24	180.56	0.736
F/G	FE/GH	76.56	24	180.56	0.453
	FB/GC	90.00	15	37.50	0.094
	FG/GF	75.00	24	180.86	0.453

Critical Loadings on frame (Loading Condition - After Construction)



On top slab:

Embankment earth = 14 x 110 = 1540 lb/Sft
Self weight of top slab = 1 x 150 = 150 "
H 10 truck (from page-25) = = 47 "
1737 lb/Sft. = 1.74 K/Sft

On Bottom Slab:

Pier = 2x1.25x6x150 = 2250 lb/ft.
Abutment = 2x1.50x6x150 = 2700 lb/ft.
4950/20.5' = 242 lb/Sft

Load from top slab =1737 "
1979 lb/Sft = 1.98 k/Sft.

(Bottom slab not included for structural design)

On Abutment Slab:

At Centre of top slab = 990 lb/Sft = 0.99 k/Sft } From page

At " of bottom slab = 1403 lb/Sft = 1.40 k/Sft }

Fixed End Moments:

$$M_{AB} = M_{BA} = M_{CD} = M_{DC} = \frac{1.74 \times 6.38^2}{12} = 5.87 \text{ Rft.}$$

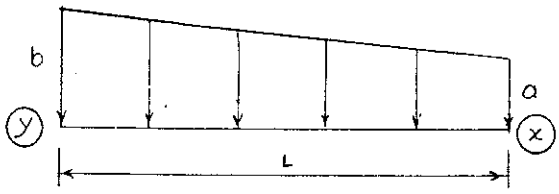
$$M_{BC} = M_{CB} = \frac{1.73 \times 6.25^2}{12} = 5.63 \text{ k'}$$

$$M_{EF} = M_{FE} = M_{GH} = M_{HG} = \frac{1.97 \times 6.38}{12} = 6.68 \text{ k'}$$

$$M_{FG} = M_{GF} = \frac{1.98 \times 6.25^2}{12} = 6.41 \text{ k'}$$

$$M_{AE} = M_{DH} = \frac{7.5^2}{60}(3 \times 0.99 + 2 \times 1.40) = 5.41 \text{ k'}$$

$$M_{EA} = M_{HD} = \frac{7.5^2}{60} (3 \times 1.40 + 2 \times 0.99) = 5.80 \text{ k'}$$

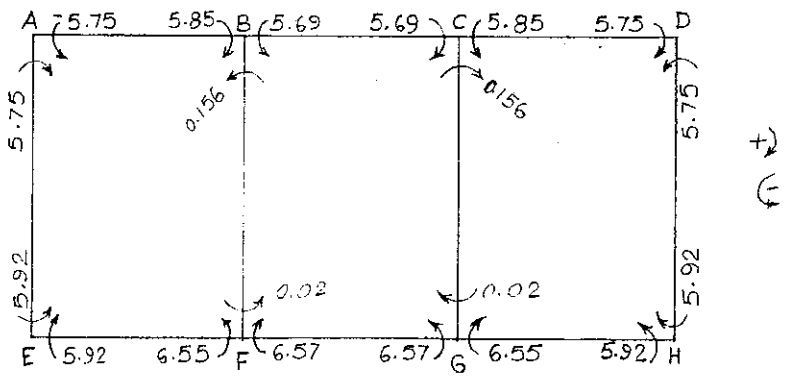


$$Mx = L^2/60 \ (3a + 2b)$$

$$My = L^2/60 \ (3b + 2a)$$

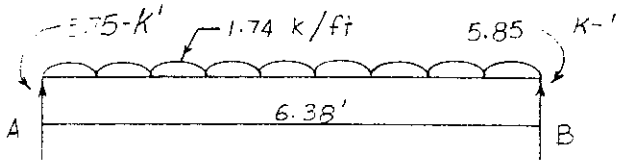
MOMENT DISTRIBUTION:

JOINT	A and D		B and C			E and H		F and G		
MEMBER	AB/DC	AE/DH	BA/CD	BC/CB	BF/CG	BA/HD	BF/HG	FE/GH	FB/GC	FG/GF
Distribution factor	0.254	0.742	0.273	0.273	0.454	0.264	0.736	0.453	0.094	0.453
F.E.M	-5.87	+5.41	+5.87	-5.63	0	-5.80	+6.68	-6.881	0	+6.41
D.M	+0.117	+0.341	-0.066	-0.066	-0.109	-0.264	-0.648	+0.123	+0.026	+0.123
C.O.M	-0.033	-0.132	+0.059	+0.033	+0.013	+0.1709	+0.061	-0.032	-0.0504	-0.061
D.M	+0.0419	+0.123	-0.029	-0.029	-0.048	-0.061	-0.171	+0.065	+0.014	+0.065
C.O.M	-0.014	-0.030	+0.021	+0.014	+0.007	+0.061	+0.032	-0.085	-0.024	-0.032
D.M	+0.011	+0.033	-0.011	-0.011	-0.019	-0.024	-0.068	+0.064	+0.013	+0.064
TOTAL	-5.75	+5.75	+5.85	-5.69	-0.156	-5.92	+5.90	-6.55	-0.022	+6.57



Frame Analysis for Shear and Moment:

Top Slab:



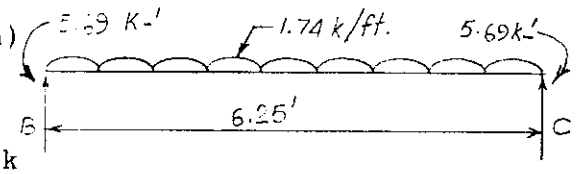
$$1.74 \times \frac{6.38^2}{2} - 0.1 = R_A \times 6.38$$

Or, $R_A = 5.53^k$, $R_B = 1.74 \times 6.38 - 5.53 = 5.57^k$

Location of Zero Shear $= \frac{5.53}{1.74} = 3.20$ (from R_A)

$$+M_{Max} = 5.53 \times 3.20 - 5.75 - 1.74 \times \frac{3.20^2}{2}$$

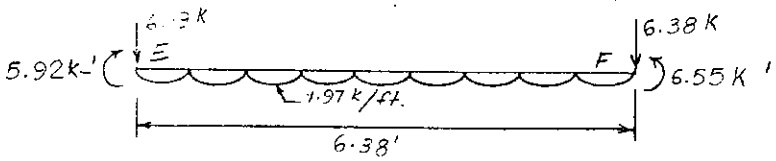
$= 3.04^{k-1}$ ------- (Bottom tension)



$$R_B = R_C = \frac{1.74 \times 6.25}{2} = 5.44^k$$

$$+M_{max} = \frac{1.74 \times 6.25^2}{8} - 5.69 = 2.81^{k-1} \quad \text{(Bottom tension)}$$

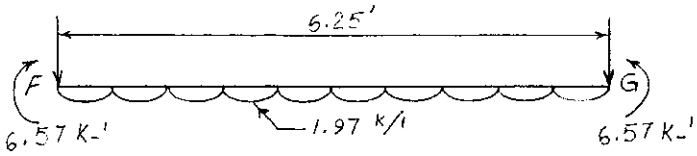
Bottom slab:



$$1.97 \times \frac{6.38^2}{2} + 0.63 = R_F \times 6.38 \quad \therefore R_F = 6.38^k, \; R_E = 1.97 \times 6.38 - 6.38 = 6.19^k$$

Location of Zero Shear $= \frac{6.19}{1.97} = 3.14'$ (from R_E)

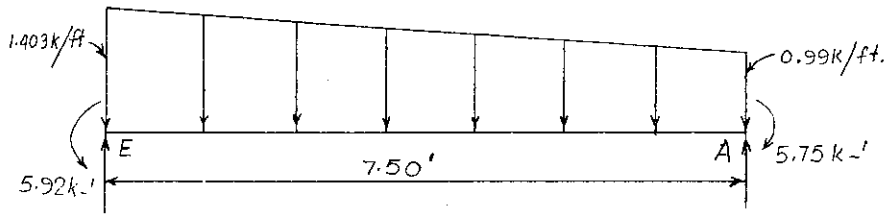
$$+M_{max} = 6.19 \times 3.14 - 5.92 - 1.97 \times \frac{3.14^2}{2} = +3.81 \ k' \quad \text{(Top tension).}$$



$$R_F = R_G = \frac{1.97 \times 6.25}{2} = 6.16 \ k$$

$$+M_{max} = \frac{1.97 \times 6.25^2}{8} - 6.57 = 3.05 \ k' \quad \text{(Top tension)}$$

Abutment



$$0.99 \times \frac{7.5^2}{2} + \frac{1}{2} \ 0.113 \times \frac{7.5^2}{3} - 0.17 = R_A \times 7.5$$

$$\therefore \ R_A = 3.86^k \qquad R_E = 7.85 - 3.86 = 3.99^k$$

location of Zero Shear form right support

$$0.99 \times \frac{1}{2} \times x \times \frac{0.113}{7.5} x = 3.86$$

or, $0.113x^2 + 14.84x - 57.9 = 0$

$\therefore$ By trial $x = 2.52'$

$$+M_{max} = 5.75 + 0.99x \frac{2.52^2}{2} + \frac{1}{2} \left(\frac{0.113}{7.5} \times 2.52\right) x 2.52 \frac{2.52}{3} - 3.86 \times 2.52$$

$= 8.94 - 9.73 \; k'$

$= -0.79 \; k'$ (Exposed Face Tension)

Member thickness and Reinforcement

Top slab

Max. Moment $= 5.85 \; k'$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{5.85}{0.157}} = 6.1"$$

$t = 6.1 + 3.5 = 9.6" < 12"$ O.K.

$$\text{Top Reinforcement} = \frac{M}{fsjd} = \frac{5.85x12}{18x.87x8.5} = 0.53 \text{ in}^2$$ use 5/8"Ø @ 6" c/c

$$\text{Bottom Reinforcement} = \frac{M}{fsjd} = \frac{3.04 \times 12}{18x87x8.5} = 0.27m^2$$ Use 5"/8 Ø @ 12"c/c

Bottom Slab

Max Moment $= 6.57 \; k'$

$$d = \sqrt{\frac{6.57}{0.157}} = 6.47"$$

$t = 6.47 + 3.5 = 9.97" < 24"$ O.K.

$$\text{Top Reinforcement} = \frac{M}{fsjd} = \frac{3.81 \times 12}{18x.87x20.5} = 0.143 \text{ in}^2$$ use 5/8"Ø @ 12" c/c

$$\text{Bottom Reinforcement} = \frac{M}{fsjd} = \frac{6.57 \times 12}{18x.87x20.5} = 0.25 \text{ in}^2$$ use 5/8"Ø @ 12" c/c

Abutment

Max. Moment = 5.92 k'

$$d = \sqrt{\frac{5.92}{157}} = 0.14''$$

t = 6.14 + 3.5 = 9.64" < 18" O.K.

$$As\ (Earth\ face) = \frac{5.92 \times 12}{18 \times 0.87 \times 20.5} = 0.23\ in^2$$ use 1/2" Ø @ 10" c/c

As (Exposed face) USE 5/8" Ø @ 10" c/c

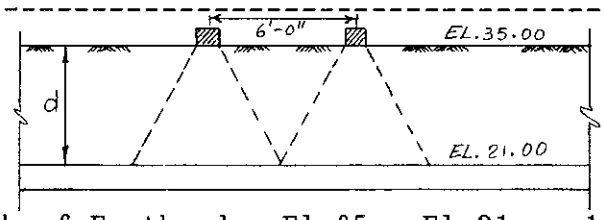
Pier

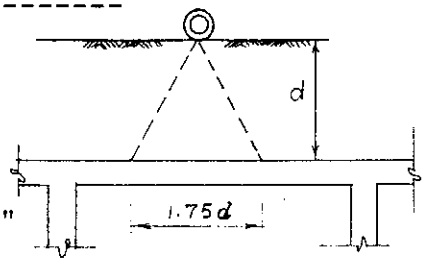
Vertical imposed load = 5.85 k + 5.44 k
= 11.29 k

load taken by 15" wide pier per 12" strip without any reinforcement = 0.25 fc' A
= 0.25 x 2.5 x 15 x 12
= 113 k ≫ 11.29 k O.K.

USE : 5/8" Ø @ 10" c/c in both face (From table-1)

(a) Distribution of Wheel loads on Embankment Earth:





Depth of Earth, d = El.35 - El.21 = 14' -0"

Length within which wheel loads distribute :

Along the embankment = 1.75d = 1.75 x 14 = 24.5 ft.
Transverse = 1.75d + 6 = 1.75 x 14+6 = 30.5 ft. use 14'-0"

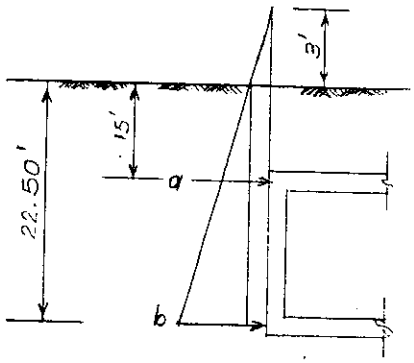
(Distributed length along the transverse direction shall not exceed the available length of 14'-0")

~~Distributed Area = 24.5 x 14.0 = 343 sft.~~

Load of Rear wheels = 0.8 x 10 x 2 = 16 k

Load/Sft = 16 x 1000/343 = 47 lb

(b) Earth Pressure behind the abutment of Box Conduit



Loading: After Construction

Earth Pressure Coefficient at Rest = 0.50

At any depth earth pressure with h' of soil surcharge, p = Ca γ (h+h')

At a, p = 0.50 x 110 (15 + 3) = 990 lb/sft.

At b, p = 0.50x110 (22.5+3) = 1403 lb/sft.

(ii) Extended Part of Conduit slab:

The extended part of conduit base slab supports the piers and abutments. The critical condition of loading is during operation when the u/s stop log is closed for retention of irrigation water.

Foundation pressure calculated = 1700 psf.
(excluding bottom slab)

Total uplift = 4x62.4 = 250 Psf.

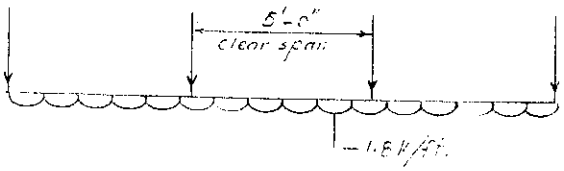
Sub. wt. of slab = 2x876 = 175 "

Net uplift = 75 "

Total load = 1775 Psf = 1.80 k/sft.

$$\pm M = \frac{1.80 \times 5^2}{10} = 4.5^{k-1}$$

$$V = \frac{1.15 \times 1.80 \times 5}{2} = 5.18^{k}$$



$d_{moment} = \sqrt{4.5/0.141} = 5.65"$ OK

$d_{Shear} = \frac{5.18 \times 1000}{55 \times 12} = 7.85"$ OK

$AS = \frac{4.5 \times 12}{18 \times 0.87 \times 20.5"}$ d= 24" - 3.5" = 20.5"

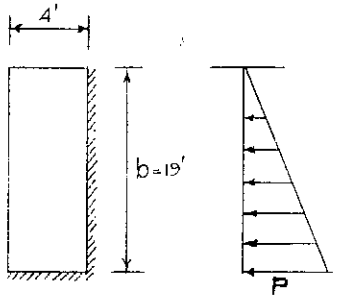
$= 0.17 \text{ in}^2$ use Min. Reinf.

Top face : 5/8" Ø @ 10" c/c

bottom face : 5/8" Ø @ 10" c/c

(iii) Extended part of abutment:

The extended part of conduit abutment is a slab cantilevered in two directions with fixing at bottom slab and headwall. The slab shall be designed with uniformly varying earth pressure as shown below. In addition, the extended wall shall be designed as cantilever beam above the conduit abutment with earth pressure behind the headwall.



$P = Ca\gamma b = 0.41.110 \times 19 = 857$ Psf.

a/b = 4/19 = 1/4-75 use 1/4 ratio table for simplicity.

Coefficient for Cx = + 0.0150 ¦ taken the max values (plate No.5)
Cy = + 0.0221 :

$M_x = Cx\ pb^2 = 0.0150 x 857 x 19^2 = 4641$ lb'

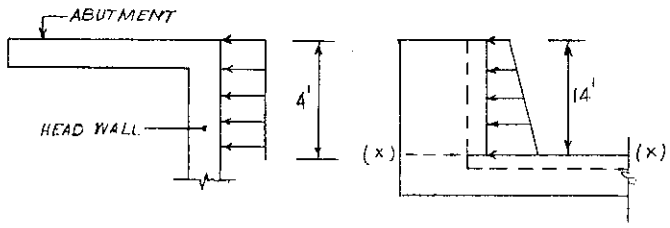
$M_y = Cy\ pb^2 = 0.221\ x 857 x 19^2 = 6837$ lb'

Check $d = \sqrt{6837/141} = 6.96"$

t = 6.96" + 3.50" = 10.46" < 18" O.K.

$$As\ (Vertical) = \frac{6837 \times 12}{18000 x 0.87 x 14.5} = 0.36\ in^2$$ use 5/8" ϕ@10" c/c

$$As\ (horizontal) = \frac{0.36 \times 4641}{6837} = 0.24\ in^2$$ use 5/8" ϕ @ 12" c/c



Pressure top of wall = 0.41 x 110x3 = 135 Psf.
" bottom of " = 0.41 x 110x(14+3) = 767 Psf.

Shear P = 1/2(135+767) x 14x4 = 25256

= 6314 x 4 = 25256 lb

Moment M_{xx} = (135 x 14 x 7+0.5x14x767) x 4

= (13230+ 5369)4

= 18599 x 4 = 74396 lb

Check

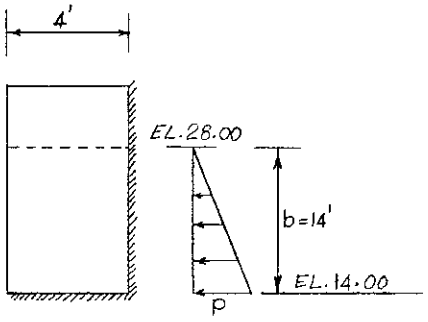
$$d_{shear} = \frac{25256}{18 \times 55} = 25.51" < 54"$$ O.K.

$$d_{moment} = \sqrt{\frac{74396 \times 12}{141 \times 18}} = 18.76"$$ O.K.

$$AS = \frac{74396 \times 12}{18000 \times 0.87 \times (60-6)} = 1.06 \text{ in}^2 \quad \text{use } 4\text{-}5/8''\phi$$

(iv) Extended Part of Pier

Pier is designed on the same concept of Extended Abutment. During maintenance, while the one bay is closed by stop log, water pressure acts on the pier. Slab from the other bay. In this case it is assumed that while the water level is at max. El. of 28.00, maintenance work is desired. The loading therefore is as follows:



$p = \gamma wb = 62.4 \times 114 = 874$ Psf.

$a/b = 4/14 = 1/3.5$ use $a/b = 3/8$ Coefficients (Plate No.5)

$Cx = 0.0246$, $Cy = +.0354$ (used max. Co-efficients)

$Mx = 0.0246 \times 874 \times 14^2 = 4214$ lb'

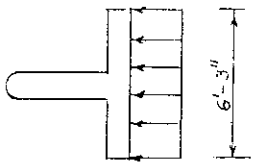
$My = 0.0354 \times 874 \times 14^2 = 6064$ lb'

$d = \sqrt{6064/141} = 6.56'' < (15''-2.5'' = 12.5'')$ O.K.

$AS_x = 12 \times 4214/18000 \times 0.87 \times 12.5 = 0.24 \text{ in}^2$ use $1/2''\phi$ @ 10" c/c

$$AS_y = \frac{0.24 \times 6064}{4214} = 0.35 \text{ in}^2 \quad \text{use } 5/8''\phi \text{ @ } 10'' \text{ c/c}$$

Designing as cantilever beam above x-x:



From the pressure diagram as before,

$P = 6314 \times 6.25 = 39463$ lb

$M = 18599 \times 6.25 = 116244$ lb

Check d

$$d_{shear} = \frac{39463}{15 \times 55} = 48" < 54" \text{ O.K.}$$

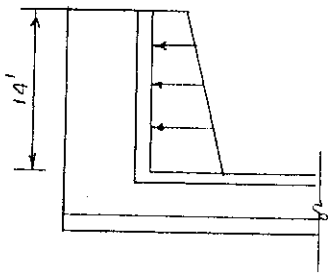
$$d_{moment} = \sqrt{\frac{116244 \times 12}{141 \times 15}} = 25.68" < 54" \text{ O.K.}$$

$$AS = \frac{116244 \times 12}{18000 \times 87 \times 5}$$

$$= 1.65 \text{ in}^2 \quad \text{use } 6\text{-}5/8"\phi$$

(v) Head wall

The head wall shall be designed as a continuous slab supporting over piers and abutments with earth pressure against it as shown below.



From above, $P_{bottom} = 767$ Psf. (as before)

$$M = PL^2/10 = 767 \times 5^2/10 = 1918 \text{ lb'}$$

Check $= \sqrt{1918/141} = 3.69 < (12" - 3") = 9"$ O.K.

$$+ AS = \frac{1918 \times 12}{1800 \times 0.87 \times 90} = 0.16 \text{ in}^2$$ USE Min.Reinf.as below

From Table-1, Horizontal bar : Earth face = 1/2" @12" c/c

Exposed face = 5/8" @12" c/c

Vertical bar : Both face = 1/2" Ø @ 12" c/c

(vi) Operating Platform

Clear span = 5'-0"

Design of 12" strip slab as beam under the hoist of gate:-

Load:- Dead load 6" slab = 75 Psf (Uniform load)

Gate and hoisting load $2/2^{k} = 1^{k}$ (Conc. load)

Moment - Uniform load = $wL^2/10 = 0.075 \times 5^2/10 = 0.19\ k'$

Conc.load = PL/4 (max) = 1x5/4 = $\frac{1.25}{1.44}\ \frac{k-1}{k-1}$

$$d = \sqrt{\frac{1.44}{0.141}} = 3.19", \quad t = 3.19 + 1.25 = 4.44"$$

< 6" O.K.

$$AS = \frac{1.44 \times 12}{18 \times 0.87 \times 4.75} = 0.23\ in^2$$

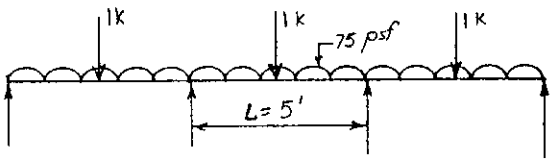
use 3-3/8"Ø within 12" width (both top & bottom)

Design of operating slab:

DL = 75 Psf.

LL = 150 "

225 "



Moment = $225 \times 5^2/10 = 563$ lb - 1

$$A_s = \frac{563 \times 12}{18000 \times 0.87 \times 4.75} = 0.09\ in^2$$

use Min.Reinf. as below

Temperature of Min. Reinf. = 0.0025 bt = 0.0025 x 6 x 12

= 0.18 in^2, use 3/8" Ø @ 6" c/c (both top & bottom)

(vii) Railing

The railing may be standardized with following sections and reinforcements.

Railing bar - 5" x 5" section with 4-3/8"ϕ longitudinal bar and 1/4"ϕ stirrup @ 6" c/c

Railing post - 6"x6" Section with 4-1/2"ϕ Vertical bar & 1/4" ϕ stirrup @ 6" c/c

B. Wingwall and Apron (Monolithic Construction)

The design of U-type D/S Wingwall-Apron for a typical section has been given below. The apron has been divided into three parts as shown in the sketch with all pertinent dimensions. The following loading conditions and pressures have been adopted.

i) Loading conditions

Loading condition - A (After construction, No Backfill)

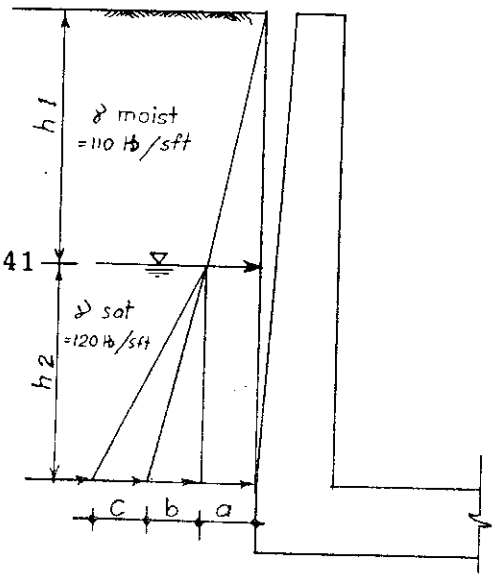
Loading condition - B (After construction, Backfill completed)

Loading condition - C (During operation, Gate closed with heading up of water).

ii) Earth Pressure

Angle of Internal friction of Backfill Soil, $\phi = 25^{o}$

Rankin's Active Earth Pressure coefficient, $Ca = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.41$



Earth pressure above water

$$Pa = Ca\ \gamma\ moist\ h_1$$

$$= 0.41 \times 110 \times h_1 = 45h_1$$

Earth pressure below water

$$P_b + P_c = Ca\ \gamma sub\ h_2 + \gamma Water\ h_2$$

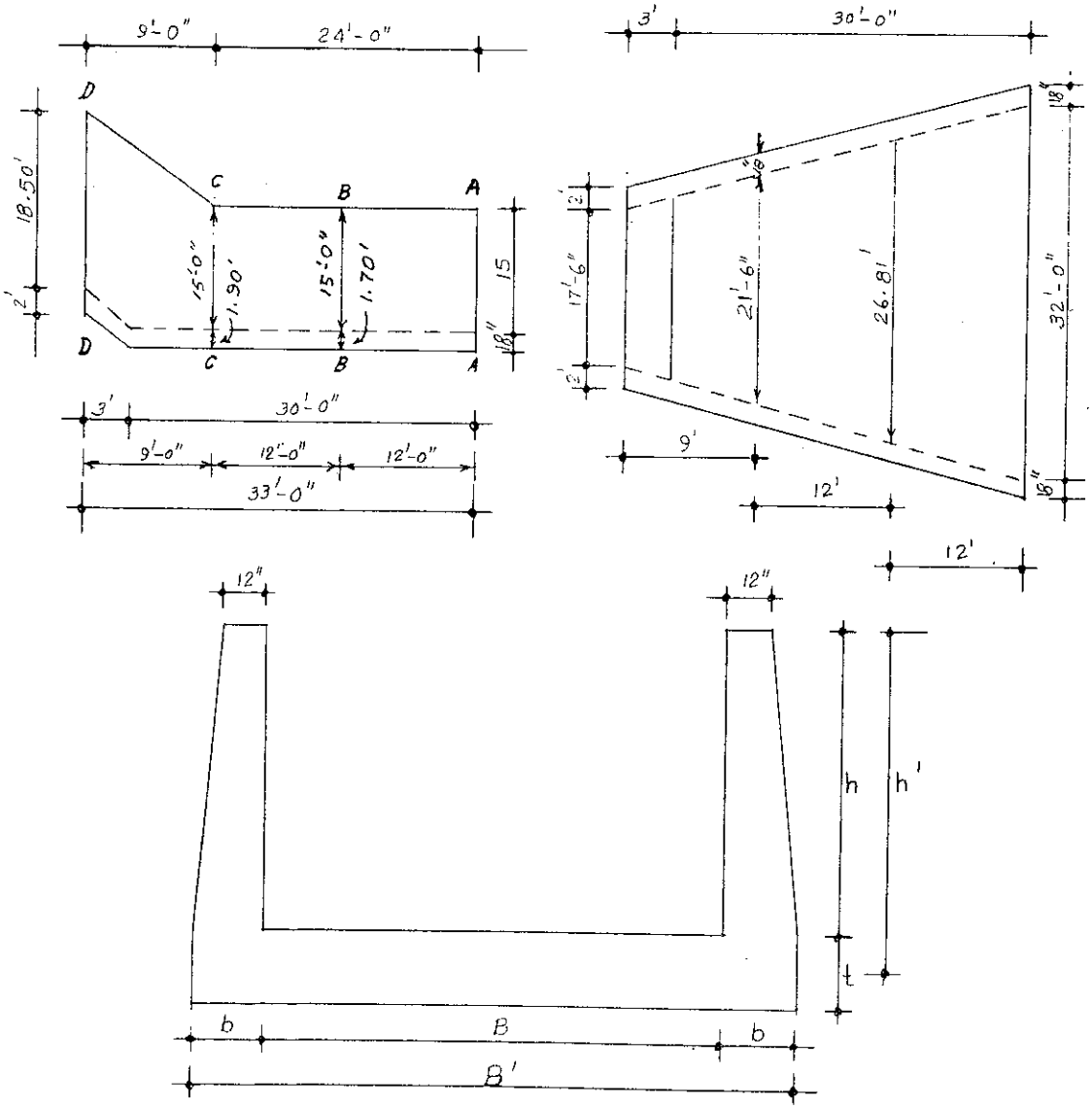
$$= 0.41 \times 57.6\ h_2 + 62.4 \times h_2 = 86\ h_2$$

iii) Uplift pressure and Saturation line

Uplift under the regulator floor has been computed in Hydraulic Design and the Hydraulic Grade line (H.G.L.) has been shown in the profile of Regulator. It is assumed that backfill soil will be saturated upto hydraulic grade line.

SECTION	h	t	h'	B	b	B'
A-A	15'	1.50'	15.75'	32'	1'6"	33'-6"
B-B	15'	1.70'	15.85'	26.81'	1'-6"	28.31'
C-C	15'	1.90'	15.95'	21.50'	1'-6"	23.00'
D-D	18'-6"	2.00'	19.50'	17.50'	2'-0"	19.50'

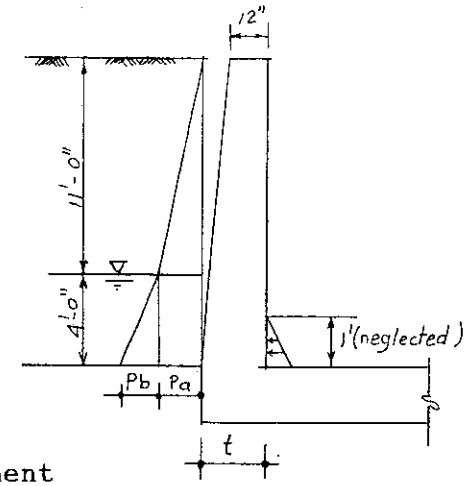
D/S APRON & WINGWALL



(a) Wingwall Design (Sec B-B)

Loading

The critical loading condition for wingwall design is loading condition C in which backfill is partly saturated.



Shear and Moment

$P_a = 45h_1 = 45 \times 11 = 495$ Psf

$P_b = 86h_2 = 86 \times 4 = 344$ Psf

Total Shear : 0.5 x 11 x 495 = 2723 lb.

0.5 x 4 x 344 = 688 "

4 x 495 = 1980 "

5391 lb.

Moment : 2723 x (4+ 1/3 x 11) = 20876 lb.

688 x 1/3 x 4 = 917 "

1980 x 2 = 3960 "

25,753 lb.

Stem thickness and reinforcement

$d_{shear} = 5391/12 \times 55 = 8.17"$

$$d_{moment} = \sqrt{\frac{25753}{141}} = 13.51"$$

Use t = 18", d = 18"-3.375" = 14.625"

$$AS = \frac{25753 \times 12}{18000 \times 0.87 \times 14.625} = 1.35 \text{ in}^2 \quad \text{use } 3/4"\phi \text{ @ } 3\ 1/2" \text{ c/c}$$

Check bond:

$$\mathscr{M} = \frac{V}{\Sigma ojd} = \frac{5391}{8.07 \times 0.87 \times 14.625} = 52.50 \text{ Psi O.K.}$$

$$\Sigma O = \frac{12}{3.5} \times 3.14\ D = 8.07"$$

Bar Curtailment

Half and one fourth of the bars may be curtailed for economy, details not shown here.

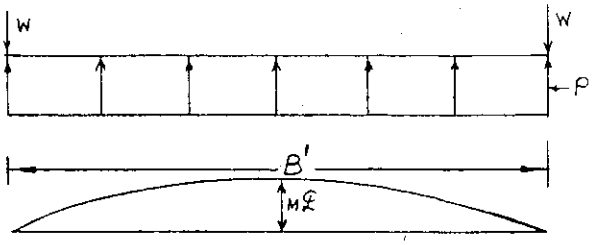
Temperature and Distribution Steel

From Table-I, Soil Face horizontal bar = 5/8"ϕ @12" c/c
Exposed Face vertical bar = 1/2"ϕ @18" c/c (Binder bar)
Exposed Face horizontal bar=3/4"ϕ @12" c/c

(b) Apron Design (Sec.B-B)

The top and bottom reinforcement of apron shall be designed for the maximum moment calculated from critical loading conditions as below:-

Loading Condition - A



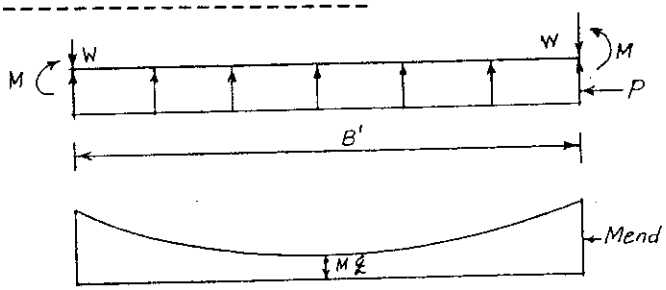
W = 0.5 (1+t) x h x 150 = 0.5 (1+1.5) x 15 x 150 = 2820 lb

P = 2W/B' = 2 x 2820/28.31 = 199 lb/sft

$M = P B^2 \; 1/8 = 199 \times 28.31^2 \; 1/8 = 19936$ lb

= 20 k-ft. (top tension)

Loading Condition - B



Here W = 2820 + 0.5(1.5-1) x 15 x 110 = 3645 lb.

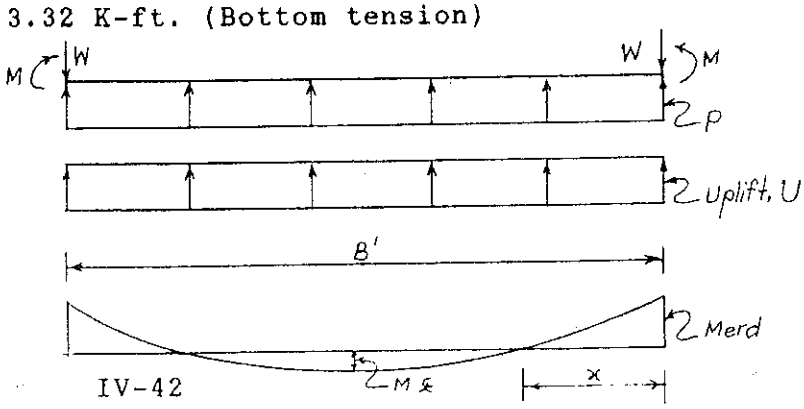
$$P = \frac{2 \times 3645}{28.31} = 258 \text{ Psf.}$$

$$M_{end} = 1/6 \; Ca \; h^3 = 1/6 \times 0.41 \times 110 \times (15.75)^3 = 29.37 \text{ K-1}$$

$$M_L = M - P B^2 \; 1/8 = 29.37 - \frac{0.26 \times (28.31)^2}{8}$$

= 29.37 - 26.05 = 3.32 K-ft. (Bottom tension)

Loading Condition - C



W = 3645

P = 25 8 lb/sft | As before

Uplift = (H.G.L. El. 15.88 - Bottom El. of slab) x 62.4

= (15.88 - 11.50) x 62.4 = 270 lb/sft.

Submerged weight of Apron = 1.70 (150-62.4) = 149 Psf

Net uplift = 270-149 = 121 Psf (U)

P + U = 258 + 121 = 379 Psf (w)

End Moment	0.5 x 11 x 45 x 11= 2723 x(4.75 + 1/3 x 11)	=22919 lb.
	45 x 11 x 4.75 = 2351 x 4.75/2	=5584 "
	4.75 x 86 x 0.5 x 4.75 = 970 1/3 x 4.75	=1536 "
		30039 lb.
		= 30 K-ft.

$M_E = M - WB'^2/8 = 30 - 0.38 \times 28.31^2/8$

= 30 - 38 = -8K (top tension)

Location of zero moment : $30 - 0.38 x^2 1/2 = 0$

or, X = 12.56'

Critical condition for Design:

It is seen from different loading conditions that maximum top tension at centre occurs in loading condition - A when there was no backfill. The maximum bottom tension at centre occurs in loading condition - B when backfill is completed. Top tension in apron slab at loading condition - A can be avoided by progressive backfilling during the construction of wingwalls. As such the following design has been done for loading conditions B and C only.

Thickness of Member and Reinforcement

Maximum moment at end = 30 K-ft.

$$d = \left(\frac{30}{0.141}\right)^{1/2} = 14.59", \quad t = 14.59" + 3.50"$$

$$= 18.09" \text{ O.K.}$$

Bottom Reinforcement

M (max.) = 3.32 K-ft. (From loading condition B)

$$As\ (centre) = \frac{3.32 \times 12}{18 \times 0.87 \times 16.90}$$

$$= 0.15\ in^2 \text{ use as below}$$

d = (1.70 x 12) - 3.50

= 16.90"

M_{end} (max.) = 30.00 k-ft. (From loading condition C)

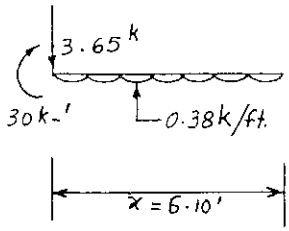
M critical for design = 30.00 - 1/3 x 1.5 x 3.65

= 28.18 K-ft.

$$As\ (end) = \frac{28.18 \times 12}{18 \times 0.87 \times 16.90} = 1.28\ in^2 \quad \text{use } 3/4"\phi @\ 3\ 1/2"\ c/c$$

Location of 50% curtailment of bar

50% of End Moment = 0.5 x 30 = 15 K



$$30 + 0.38\,x^2/2 - 3.65\,x = 15$$

$$\text{or, } 60 + 0.38\,x^2 - 7.30\,x = 30$$

$$\text{or, } 0.38\,x^2 - 7.30\,x + 30 = 0$$

$$\text{or, } x^2 - 19.21 \times 78.95 = 0$$

By trial x = 6.10 ft.

50% bar can be curtailed at (6.10 + 1.5/2 + 16.90/12) = 8.26' from end of Apron.

Top reinforcement

$M_{\mathcal{C}}$ (max.) = 8^{k-1} (From loading condition C)

$$As\ (centre) = \frac{8 \times 12}{18 \times 0.87 \times 16.90} = 0.36\ in^2$$

use 3/4" Ø @ 12" c/c as Min. reinf.

Longitudinal Reinforcement

Use longitudinal bar as minimum reinforcement from table-1 as below:-

Top face = 3/4" Ø @ 12" c/c

Bottom face = 5/8" Ø @ 12" c/c

C. Return wall (Monolithic with caisson)

The stem thickness and reinforcement of return wall are same as calculated in the design of wingwall. The base of the returnwall is the top slab of caisson whose vertical reinforcements need to be embedded in the caisson top slab/base slab of returnwall.

D. Caisson Cutoff Wall

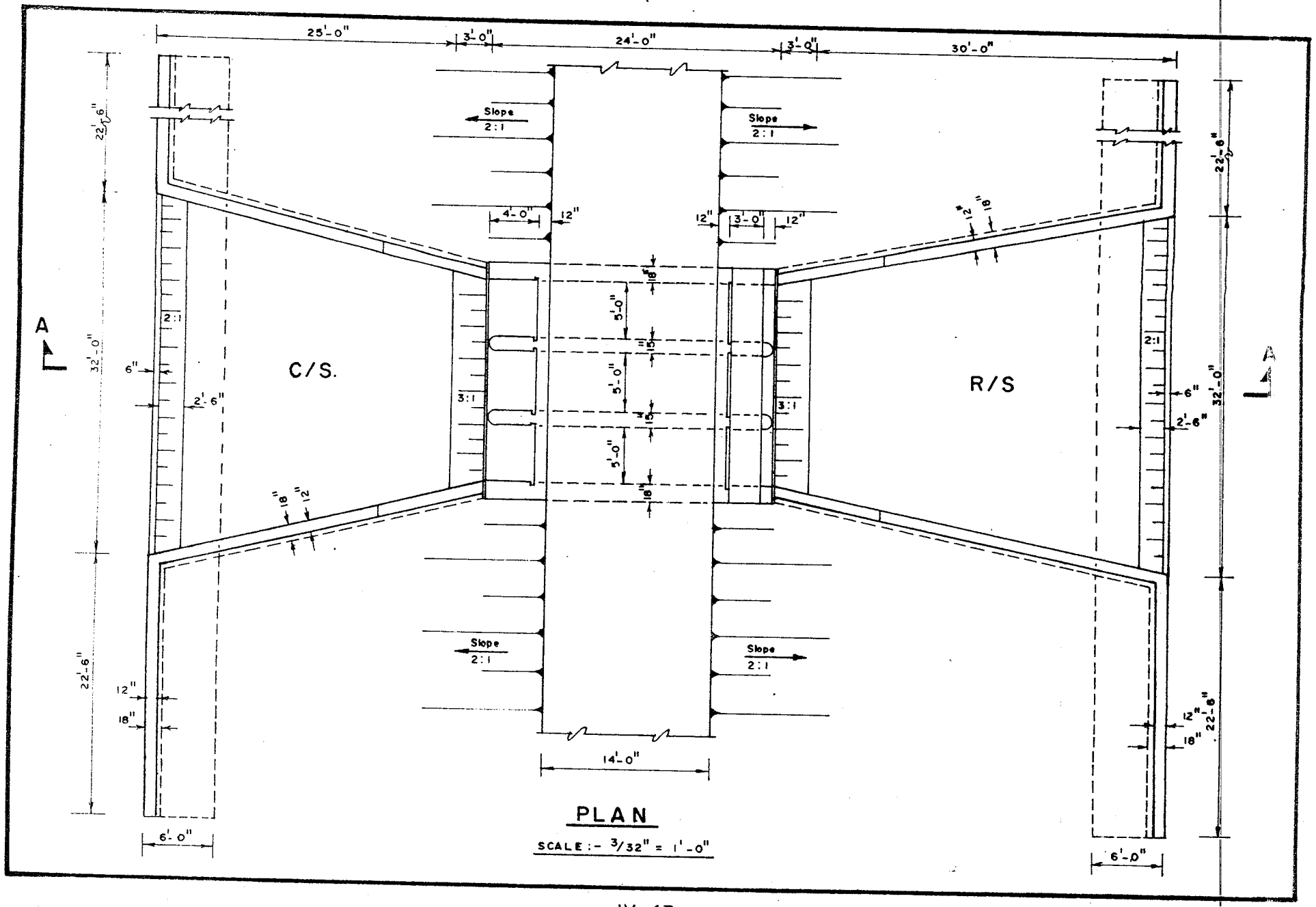
R.C.C. Caisson with about 4'x6' chamber size and minimum reinforcement of 1/2" Ø @ 12" c/c has been standardized and found satisfactory for cutoff Wall.

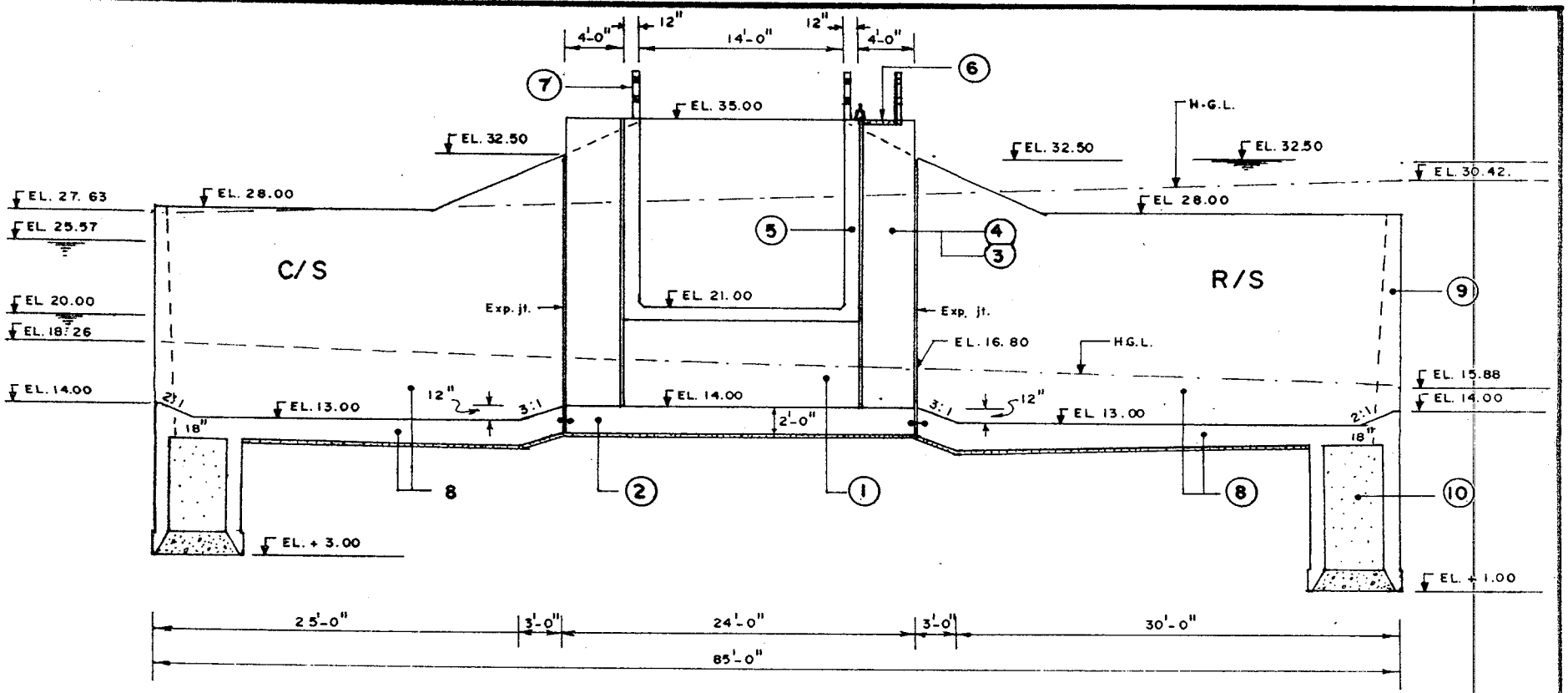
Table - 1

TEMPERATURE/SHRINKAGE OR MINIMUM REINFORCEMENT

REFERENCE : USBR CODE

Thickness	Location	Quantity of Reinforcement	Recommended Dia and Spacing of Bar
A. Length less than 30' - 0"			
12"	Earth Face	$0.0010 \times 12" \times 12" = 0.144\ in^2$	1/2" @ 12" c/c
- do -	Exposed Face	$0.0020 \times 12" \times 12" = 0.288\ in^2$	5/8" @ 12" c/c
15" or greater	Earth Face	$0.0010 \times 15" \times 12" = 0.18\ in^2$	1/2" @ 10" c/c
- do -	Exposed Face	$0.0020 \times 15" \times 12" = 0.36\ in^2$	5/8" @ 10" c/c
B. Length greater than 30'-0"			
12"	Earth Face	$0.0015 \times 12" \times 12" = 0.216\ in^2$	1/2" @ 10" c/c
- do -	Exposed Face	$0.0025 \times 12" \times 12" = 0.36\ in^2$	5/8" @ 10" c/c
15" or greater	Earth Face	$0.0015 \times 15" \times 12" = 0.27\ in^2$	5/8" @ 12" c/c
- do -	Exposed Face	$0.0025 \times 15" \times 12" = 0.45\ in^2$	3/4" @ 12" c/c


25'-0"
3'-0"
24'-0"
3'-0"
30'-0"
22'-6"
32'-0"
22'-6"
Slope
2:1
4'-0"
12"
3'-0"
18"
5'-0"
15"
C/S.
R/S
2:1
3:1
6"
2'-6"
14'-0"
6'-0"
A
A
PLAN
SCALE :- 3/32" = 1'-0"



LONGITUDINAL PROFILE THROUGH A-A

SCALE :- 3/32" = 1'-0"

LEGEND :-

1. Box Conduit
2. Extended Part of Conduit Base Slab
3. Extended Part of Conduit Abutment
4. Extended Part of Conduit Pier
5. Head wall
6. Operating Platform
7. Railing
8. Wing wall & Apron (Monolithic)
9. Return wall
10. Caisson cut off wall

NOTE :-

Protection work not shown

PLATE NO. 8.

CHAPTER V.

FOUNDATION DESIGN

M.A. Azam.

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1.0 BACKGROUND :

Bangladesh Water Development Board is associated with the implementation of Flood Control, Irrigation and Drainage Projects of both large and small dimension. Activities of the Board expands over the whole country. Flood control embankments of few projects have been constructed long ago. After the implementation of embankments, problems of drainage arises and many sluices, regulators, syphones have been constructed along with bridges of long and short spans. It may be mentioned here that BWDB has already implemented about 8,500 Nos. of sluices, regulators and bridges. In addition, the Board completed construction of about 200 Nos. of water control structures main function of which is to retain water during dry period.

During the execution of all these structures serious foundation problems had been encountered in many cases. During the early period of implementation of Coastal Embankment Project, much stress on foundation was not considered as scope of soil exploration was very limited. At present there is a Ground Water Circle wholly responsible for collection of sub-soil data. Soil Testing Laboratory is responsible for analyzing the data. However, specific opinion regarding the type of foundation is not incorporated in the report and in some cases no information of soil parameters other than bore-logs are supplied. As such, design of foundation treatment becomes very difficult.

1.1 GENERAL TYPE OF SOIL :

In the northern part of Bangladesh foundation soil is better and generally sandy soils are found at shallow depths in most of the cases and as such special treatment is not required for normal structure. Moreover, these region have no tidal effect and foundation level is normally above ground water table. In the coastal zone foundation soil creates major problems. Soil of Satkhira and Khulna is very poor and needs lot of attention and careful judgement during foundation design. In some parts of Chittagong which is also in coastal embankment project, soil condition is very poor and "0" SPT value has been encountered even at a depth of 80'-0" from ground level. In location of poor soil special care is to be taken for design of foundation.

2.0 TYPICAL FOUNDATION TYPE :

In order to give the foundation engineer the first approach in the philosophy of selecting a foundation, the principal type of foundations will be outlined.

Isolated Footings : Footings are understood to be formed by a rigid rectangular base of stone or concrete of dimensions width 'B' and length 'L' in which the ratio L/B will not exceed 1.5. The foundation of structure will support the column load. Single footing foundations, in general, will be used only in soils of low compressibility and in structure where the differential settlements between columns may be controlled by the superstructure fexibility.

Continuous footings : When it is necessary to control within certain limits the magnitude of differential settlements between columns supported on footings, and when soil deposits of medium or low compressibility are encountered, it is recommended to use continuous footings. Continuous footings are arranged by joining two or more columns together with beams.

2.1 RAFT FOUNDATIONS :

When the loads are so large that continuous footings will occupy close to 50% of the structure, it is more economical to use a continuous mat covering the entire area. The total load in this case may be assumed uniformly distributed in the area covered by the building. The soil reaction is determined on the basis of safe bearing capacity. This type of foundation may by used generally in soil deposits of medium compressibility. In certain cases, the surface raft foundation may be used in soils of high and very high compressibility, where large total settlements may be allowed. This type of foundation may be used efficiently in reducing differential settlement.

2.2 COMPENSATED FOUNDATION :

In soil deposits of medium, high and very high compressibility and low bearing capacity, compensated foundations are indicated. This type of foundation requires a monolithic box foundation. In the design of compensated foundation, it should be borne in mind that the soil should be considered as a material of two phases

namely a solid and a liquid phase. Therefore, in a compensated foundation, the compensation is made by adding two effects:

i) Substitution of the submerged weight of solids and the buoyancy effect by the weight of liquid displaced. The volume of the concrete box forming the foundation structure and basements will displace a weight of liquid that, according to Archimedis principle, will contribute in floating the foundation upto that value, reducing the load applied to the solid phase. The load taken by solid phase will however, deform the soil because of the change in effective stress induced in the soil structure. It should be investigated from the point of bearing capacity as well as total and differential settlements.

A compensated foundation is designed usually with a stiff foundation structure, therefore differential settlements are controlled rather easily. The load will be compensated by means of an excavation sufficiently deep to permit the obtention of the necessary load capacity and reduction of vertical displacements. Theoretically, if one could make a substitution of load without changing effective stresses and hydraulic pressures, no vertical displacements would take place at the ground surface.

The foundation concept of this type of foundation is to achieve the minimum change in effective stresses during excavation and construction of the foundation structure.

2.3 FOUNDATION WITH FRICTION PILES :

When a compensated foundation is not sufficient to support the load with the allowable total settlement, friction piles may be used. This case may be present in deposits of high or very high compressibility extending to great depth. The piles will reinforce the upper part of the soil where a higher compressibility is encountered. The applicability of this foundation calls for a soil that varies from very high compressibility at the upper part of the deposit to medium or low compressibility at the bottom. Total settlement of this type of foundation depends greatly on the way the friction piles are driven, their spacing and length.

2.4 FOUNDATION WITH POINT BEARING PILES :

When the load is higher than a friction pile can take, then it will be required to find deep seated hard stratum of low to very low compressibility and high shear strength, where piles can be driven to point bearing.

Two main cases of point bearing pile is encountered. The first case is recognized when the hard stratum of considerable thickness is found underlain by materials of medium compressibility. In this case after solving the problem of point bearing of the piles in the hard stratum, there still exists the problem of investigating if the lower compressible soil stratum will have a safe bearing value and also if the total differential settlements will be within allowable limits.

Second type of point bearing pile foundation is observed when the point bearing piles rests in a firm deposit of low compressibility extending to a great depth. The piles driven in firm stratum develop lateral friction contributing to the total bearing capacity. The bearing capacity of the piles will depend mainly of the mechanical properties on shear strength of the deposits in which they are driven, spacing of the piles, length of penetration into the bearing stratum and on the state of density and confinement of such stratum. The total and differential settlements of these foundations may be estimated computing the compression of the strata underlying the point of piles.

2.5 PIER FOUNDAITON :

This type of foundations are used to support very heavy loads in burried soil deposits of very low compressibility. Their load capacity is a function of the mechanical properties of the soil under the base of the pier and of the confining stress of the bearing stratum. Actually, the bearing capacity of such an element is determined as a deep seated isolated footing.

2.6 SAND PILE FOUNDATION :

This type of foundation is used to increase the load capacity of the soil by reducing its compressibility and increasing its shear strength capacity properties. This type of pile made by used in loose or medium, dense sand deposits. The improvement of the sub soil is a function of the volume of sand introduced at the time

these elements are installed. Usually first a hole is driven in the ground, then sand is introduced and is highly compacted in layers using a heavy ram. The sand elements will take the load because of the lateral confinement given by the subsoil.

3.0 SELECTION OF PROPER FOUNDATION :

The foundation of a structure is that element which provided the ultimate support for the structure and its static and dynamic loads. The selection of the type of foundation and foundation structure depends first on the ability of the foundation engineer to recognize the mechanical behaviour of the subsoil materials encountered in nature at the specific site in question and second, the mechanical behaviour of the type of foundation structure to be used, the loads to be supported and the allowance on total and differential settlements. The selection should always be the most economical type of foundation that can be used, fulfilling the requirements of allowable total and differential settlements in conjunction with sub-soil condition encountered. The following type of foundation is suggested as reference.

i) When the soil is of the expansive type a raft foundation or short piles foundation may be contemplated. Total and differential settlements should be carefully considered.

ii) In sandy clayey silts, continuous footings or raft foundations are indicated.

iii) On river planes where the finest alluvial sediments are encountered, compensated foundations may be used and in cases, use of piles or caissons may prove to be necessary.

iv) Soils consisting of fine or very fine sediments like silts and clays in deltaic area are encountered. These deposits may be of medium to high and very high compressibility. Compensated foundation with or without friction piles may be used in compressible deposits extending to great depth.

In case of hydraulic structures in coastal zone base of the foundation is in most cases below ground water table which reduces allowable bearing pressure. Moreover, due to surface pumping during construction, soil becomes more disturbed and does not reflect the condition as shown in bore logs. Box part i.e, middle part of hydraulic structures are kept separated from u/s apron and d/s apron part as there is considerable difference of loading. The separated part is considered as expansion joint and PVC water stop is provided to check seepage. If the differential settlement is more than 1 inch, it may crack P.V.C. water stop and structure may face danger of failure. So, special consideration is taken for foundation design so that settlement does not exceed allowable limit.

4.0 SHALLOW AND DEEP FOUNDATION :

Foundation as we encountered with small scale structures is shallow. A shallow foundation is one which is placed immediately beneath the lowest part of the superstructure which it supports.

The deep foundation is one in which the foundation must be placed at considerable depth below the lower part of the superstructure. Obviously, there is no sharp dividing line between the two classes. Generally foundation that has a width "B" equal to or greater than the depth of the foundation "D_f" is considered a shallow foundation. Shallow foundation includes spread footings and mats which transfer structural loads to the underlying soil at a relatively shallow depth below the lowest part of the superstructure. Deep foundation includes piles and caissons which transfer structural loads through a foundation member to bear on within soil at a considerable depth below the structure.

When soils occuring immediately below the structure are unsuitable for support of shallow foundations, it is sometimes possible to use shallow foundations after the unsuitable soil is excavated and replaced by compacted fill, or after the unsuitable material is improved or stabilized in some manner by densification or by pre-compression. Pre-compression for support of shallow doundation is a relatively new, useful and flexible tool for the soil engineer provide that tool is used with skill and judgement.

5.0 REQUIREMENTS FOR A SATISFACTORY FOUNDATION :

A satisfactory foundations for any given structure must meet the following :

a) It must be adequately safe against structural failure.

b) Foundation must not settle differentially by an amount which could damage the structure during its life time.

(e) Foundation must be feasible both technically, economically and practical to build.

6.0 BEARING CAPACITY CONCEPT :

The bearing capacity of a soil is defined as the maximum load per unit of area which the soil can support without rupture. It is necessary to investigate both base shear resistance and settlement for any structure. In many cases settlement criteria will control allowable bearing capacity. A recommendation for the allowable bearing capacity to be used for design is based either on settlement considerations or on the ultimate bearing capacity. The ultimate bearing capacity is divided by a suitable factor of safety based on the type of soil, accuracy of imput data.

Ultimate bearing capacity of the soil is divided by a safety factor to obtain the safe bearing capacity. A safety factor is used to safeguard against :

(a) Natural variations in the shear strength of the soil

(b) Uncertainties in the accuracy or reliability of theoretical or empirical methods for calculating bearing capacities.

(c) Minor local deterioration in the bearing capacity of the soil during or subsequent to construction.

(d) Excessive settlement caused by yielding of the soil when the foundation is approaching failure in shear.

Of the above, the variation in soil conditions provides the main reason for requiring an adequate safety factor. A high degree of judgement is required of the foundation engineer in selecting design values of shear strength of the soil on sites where there is a wide scatter of tests results. A safety factor of 2.5 to 3 is generally used to cover the variations or uncertainties listed above. A value of 2 would only be adopted on sites where very uniform soil condition are found. Low safety factors of 1.5 to 2 are also used in the design of temporary works such as cribs supporting construction plant, or for calculation of the allowable bearing pressure for earthworks where the effects of settlement are not detrimental to temporary works or adjoining property.

It should be noted that safety factors used in calculations determine safe bearing pressures are not normally intended to cover accidental increases in structural loading.

6.1 BEARING CAPACITY EQUATIONS :

In the past design of shallow foundation was mainly empirical and it was based largely on allowable bearing pressures for various soil types. The study of foundation behaviour in the field amplified by laboratory research has led to more rational approach. It is now recognized that the results of a site exploration and soil test form the basis of an analysis of the stability and probable movements of the proposed foundation and superstructure.

One of the early set of bearing capacity equations was proposed by Terzaghi (1943). He modified a bearing capacity equation developed for an infinite strip by Prandt (1920) based on theory of plasticity. No exact mathematical approach has been devised for the analysis of bearing capacity failure in soil. Many methods have been formulated but all involved some simplifying aproximations regarding soil properties and the movements which takes place in soil mass. An analysis of the condition of complete bearing capacity failure can be made of assuming that the soil behaves like an ideally plastic material. Equations were further developed by Meyerhof (1951, 1963), Hansen (1957, 1970), Buisson and others.

Formula given by Terzaghi is $qd = CN_c + D_f N_q + 1/2 \; B N$.

Where C = cohesion of soil

N_c, N_q & N are dimensionless bearing

Capacity factors that depends on "O" (Angle of internal friction). Terzaghi gives the values of N_c, N_q, N in the form of curves. This equation is for the case of general shear failure when the soil was more compact initially and settlement of footing develops full plastic zones. But for local shear failure for soft and loose soil, Terzaghi gives modified values for bearing capacity factors as N_c, N and N_q. Generally for "N" value 5 or less, local shear failure is considered.

Semi empirical equation are given by Terzaghi for circular and square footing.

Circular footing of diameter "B"

$$q = 1.2\ C\ N_c + DN_q + 0.3\ B\ N$$

Square footing of size "B"

$$q = 1.3\ C\ N_c + DN_q + 0.4\ BN$$

The above equations are not applicable for very deep foundations.

Since bearing capacity depends on shape and depth of footing, the inclination of load, Hansen introduced factors to account for them.

$$q = C\ N_c\ S_c\ dcic + D_f\ N_q\ S_q\ d_q\ i_q + 1/2\ B\ N\ S\ D\ i$$

Where c = cohesion of soil
= surcharge
d = depth of factor
i = inclination factor
s = shape factor

These values are given in table 5 and 6.

There are many other methods which are not being considered here.

It may be recognized that the ultimate bearing capacity is obtained by three independent terms. The 1st term is function only of the soil shear strength parameters C & 'O', 2nd term is a function of the vertical confining effective stress at the footing grade elevtion, and third term is a function of the width of the footing on the average of the soil mass under footing. The position of water table affects only second and third term. If the water table is at the foundation level the reduction factor will be taken as 0.50. It is for this reason footing under water table experienced less bearing capacity.

7.0 CHOICE OF FOUNDATION :

If sub-soil is loose and bearing capacity is not adequate to withstand the load of the structure, lot of consideration regarding foundation is to be taken into account. Soil condition at a particular site is not generally found ideal and economic foundation design is not possible for all cases. This observation applies not only to design but also to construction problems. Very often the economic solution may not be feasible from point of view of working condition.

Foundation soil may be composed of medium dense sand, or may be very soft in top underlying dense strata or there may be a dense soil in between comparatively weaker soil. There may be other combinations. Considering all such cases, foundation differs for same type of structure from place to place.

Very often in coastal areas loose sand of shallow depth overlying deep layers of soft clay is found. In such cases the nature of foundation to be adopted for it depends upon the loading and type of structure. For light structure, whose foundation width is very narrow and there is no interference between two foundations, individual soil footings offer the most economical solution. But when the load intensities are very heavy and sand layer is very thin, the settlement of clay layer below will contribute to large differential settlements of magnitudes exceeding the allowable limit. So alternative ways should be considered. It may be necessary to carry the structure

on bearing piles passing through the clay strata to dense sand below.

Stiff crust underlain by soft clay may be found. The top crust may be formed due to over consolidation. Foundation on such ground depend upon load. If the structure is comparatively light it is most advantageous to locate the foundation i.e, individual footings on hard soil. Settlement in clay layer under hard soil is to be checked. But if the load is very high, deep foundation will be necessary.

If there is loose fills resting on hard strata, there is no question of individual footing. In such cases pile foundation or pier foundation is the solution.

7.1 FOOTING FOUNDATIONS AND BEARING VALUE :

The most important step in the design of a footing foundation is the evaluation of greatest pressure that can be applied to the soil beneath the footing without causing excessive settlement. The pressure is known as allowable bering value "qa". Determination of this allowable soil pressure is the most critical step in design process and there are number of different approaches to this problems.

Before the advent of soil mechanics, the method for choosing the allowable soil pressure were based on experience and inadequate knowledge of the properties and behaviour of soils. Designers have assembled records of foundation successes and failures and

correlated with the character of the soil on which the foundation is rested and pressure exerted by foundation on the soil.

With the development of soil mechanics it was observed that many of the significant factors in foundation were not taken into account such as compressibility of the sub-soil, the character of the structure itself including its size, shape, depth, loads and its ability to withstand settlement. Allowable soil pressure is verified on the basis of the load test. The ultimate bearing capacity of the soil can be estimated from load settlement curve.

8.0 ALLOWABLE SOIL PRESSURE ON SUB SOIL :

As the load on a given foundation increases, the soil underneath deforms and the foundation settles. At low pressure the settlement may increase indirect proportion to the load but with increasing load, the rate of increase of settlement usually increases. The relation between settlement and average load per unit of area may be represented by a load settlement curve shown in fig. 3. If the soil is fairly dense or stiff, the settlement curve is similar to curve 1 and if the soil is loose or fairly soft, the settlement curve is similar to curve 2.

If the load becomes sufficiently great, the movement of settlement may be excessive or uncontrollably large and the foundation is said to have broken into the ground or to have experienced a bearing capacity failure.

Every foundation, therefore, satisfy two conditions.

a) The factor of safety of foundation with respect to breaking into the ground should not be less than about 3 which is the maximum factor of safety.

b) The deformation of the base of the structure due to unequal settlement should not be great enough to damage the structure.

The allowable soil pressure on cohesionless soil is usually determined from settlement consideration.

8.1 RATIONAL DESIGN METHOD :

The rational approach to foundation design essentially is one of trial evaluation -- revision technique. A trial design is assumed on the basis of experience. It is analyzed to determine how well it meets the requirements at a minimum cost. The use of this method requires extensive, accurate information regarding the soil and the superstructure. The soil data required are the depth, the thickness of soil strata, the location of ground water, the physical properties of soil, particularly, their strength and compressibility. The variations of the soil from one part of the site to another govern the way in which the analysis of bearing capacity and settlement are made.

9.0 BEARING CAPACITY AND SAFETY FACTORS :

The ultimate bearing capacity may be determined from theoretical analysis or in some cases from properly interpreted plate load test. In finding the ultimate bearing capacity the average soil properties are used for uniform deposits. For the variable erratic deposits, the poorest strength is employed. Safety factor is selected depending on how accurately the soil conditions are known, and the type of the loading. The required minimum is usually 3. When there is reasonably accurate soil and loading information and or there is large component of live load that is unlikely to develop, a factor of safety 2 may be employed with total loads. Similarly, if there are questionable condition, a safety factor of 2 is advisable. For temporary structure, a small value of safety factor is employed.

Now discussion will be concentrated on footing on sand, and clay and footing in soil intermediate between sand and clay.

9.1 FOOTING ON SAND :

In order to establish reliable criteria for design of footings on sand, the allowable soil pressure must be correlated with properties and conditions that have a significant influence on the behaviour of sand under load. These conditions are the

relative density of the sand and position of water table on the base of footing.

$$\text{Relative density } Dr = \frac{emax - e}{emax - emin} \times 100$$

emax = void ratio in longest state

e = inplace void ratio

emin = void ratio in densest state

The relative density has a decisive influence on the angle of internal friction and the shape of the load settlement curve. The position of water table with reference to the base of footing also has an influence on both the ultimate bearing capacity and settlement. The allowable load for cohesive soil under settlement consideration is determined by means of consolidation test on undisturbed samples and computing settlement by Terzaghi's theory and thereby limiting the settlement within permissible limits, for sandy foundations, undisturbed samples being not possible, the allowable load by standard Terzaghi's curves (Terzaghi & Peck, 1948) based upon situ penetration tests and load tests. These curves (Fig. 4) have been drawn on the supposition that settlement of the footing is 2.5 cm and differential settlements 1.25 cm and that the water table exists at a depth equal to twice the base width "B" of the footing below the foundation level. Below water table, Terzaghi assumes that the submergence of sand located beneath the base of a footing would approximately double the settlement. On this basis, if the depth base ratio D/B of the depth of foundation and base width is small like that of basement, the value could be reduced by 50%.

If the depth ratio is close to unity, the values are to be reduced by 33%. Meyerhof (1956), has however pointed out that due to watertable the bearing capacity should be reduced linearly from 0 to 0.5, if the water table rises from 2B to zero below the base of footing.

This treatment of determination of bearing capacity does not take into account that reduction in the values of penetration resistance due to groundwater. Thus the allowable load obtained by settlement criteria, which is the guiding criteria in the case of sand, gives conservative results leading to uneconomical design of superstructure.

10.0 EFFECT OF WATER TABLE ON PENETRATION RESISTANCE :

Holtz and Gibbs (1957) in their Laboratory Study observed that in the coarse sand "N" values have decreased by 25% whereas in fine sand by 50 - 65 percent. If a saturated sand located beneath a footing is very loose, (N<5) the footing should be supported on piles, or sand may be compacted.

In order to determine the allowable soil pressure 'qa' by means of the chart (Fig. 4) the effect of submergence on settlement must be considered. According to theory the submergence of sand located beneath the base of a footing should approximately double the settlement provided the base is located at or near the surface of the sand.

If Df/B is small, the values obtained in chart should be reduced by 50%, whereas if the depth breadth ratio is close to unity, the values in the chart need be reduced only by one third, because, the effect on the settlement of the weight of the soil surrounding the footing partly compensate for that of the increase due to saturation.

Where excessive values are encountered in above soil under saturation condition those should be corrected to equivalent values by applying the following relationship.

$$Ns = 15 + \frac{N - 15}{2}$$

Ns = computed equivalent value to be adopted in designs

N = actually observed number of blows

Some engineers do not accept that it is necessary to halve the bearing pressures if the water table is close to foundation level. The consider that the test results themselves automatically allow for reduction in bearing pressure. Certainly some N-values when plotted with depth do show a reduction of about 15 percent when they pass from a dry or damp sand to a saturated sand. However, there is no conclusive evidence to suggest that it is safe to reject the established procedure of halving the bearing pressure for foundations at or below the water table.

11.0 FOOTING ON HOMOGENEOUS CLAY :

The allowable soil pressure on clay, as well as that on sand, should satisfy the two requirements, that factor of safety against bearing capacity failures should be adequate and the settlement produced by the load should be within tolerable limits.

The Net ultimate bearing capacity "qd" per unit area of footing resting near the surface of clay may be calculated from

$$qd\ net = 5c(1 + 0.2\ Df/B)(1 + 0.2\ B/L)$$

where B = width of footing

L = length of footing

Df = depth of foundation

Df should not exceed 2.5 B. For a circular footing, diameter D = B = L. Under normal condition the factor of safety of footing on clay shall not be smaller than 3. For computing bearing capacity of clay, it is necessary to determine the undrained shear strength of the clay below the proposed footing using 2" or 3" tube samples secured between the base of the footing and depth below base equal to width of footing. The average stength "c" of the soil can be estimated from unconfined compression test. The procedure is valid provided the sub-soil does not contain within the significant depth a layer of clay softer than clay defined by value of 'c' on which the allowable soil pressure is based. The value of ultimate bearing capacity is calculated and divided by factor of safety to find the allowable bearing capacity.

The allowable soil pressure 'q_a' on a clay can be assumed to be equal to one third of the value of qd net determined from equation. Whether this requirement is satisfied, depends primarily on whether the clay is normally loaded or pre-compressed.

If the footing rests on a normally loaded clay, the magnitude of both total and the differential settlement can be very large. Fortunately footing foundation on normally loaded clay, are rare exceptions. In most localities, even soft clay, are pre-compressed to some extent, either by dessication or by temporary lowering of water table.

11.1 FOOTING FOUNDATION LOCATED ON FIRM SOIL ABOVE SOFT CLAYS :

Let us consider a case of footing resting on a firm Stratum A, located above a soft Stratum B. If the upper boundary of the soft stratum is located close to the base of the footing, the footing may break through the firm layer into the soft deposit. They can be avoided by giving the footing such dimension that the pressure on the upper boundary of the soft Stratum B does not exceed the allowable bearing value of the soil in that Stratum. This can be done by Bousines's or Newmark chart. The total footing load can be assumed uniformly distributed over the base of a truncated pyramid where side sloped from the edges of the footing to the upper surface of B at an angle of 60° with horizontal.

পলি দাশ
উপ-বিভাগীয় প্রকৌশলী
নকশা সার্কেল-২
বাপাউবো, ঢাকা-১২১৫

If the upper boundary of the soft stratum 'B' is locaed at a considerable depth below the base of the footing, failure by breaking into ground can not occur since stratum 'A' acts like a thick mat or raft that distributes the entire weight of the structure almost uniformly over the surface of 'B'. The firm stratum supporting the footing acts like a natural raft that distributes the load from the footing over the softer stratum. Thus the seat of settlement is located well below the seat of soft clay.

11.2 STABILITY ANALYSIS FOR FOUNDATION ON CLAY :

Regarding the analysis of stability of foundations on clay it is necessary at the outset to recognize the basic difference in the behaviour of soft clays and stiff over consolidated clays. In soft clays positive pore pressures are developed when the load is applied in a relatively short period. Hence the critical condition generally arises immediately after the loading. In over consolidated clays the pore pressures are generally small and sometimes negative, hence long term stability must be considered. In soft clays, influence of creep is compensated by dissipation of pore pressure. There are many information of failure by creep in foundations resting on soft saturated clays.

Cohesive soils are virtually incompressible in undrained shear and the angle of internal friction is close to zero. Hence the basic assumptions underlying theory of plasticity are rigorously valid.

There is uncertainly in the stability analysis of the dispersion of load through a sandy layer overlying a soft clay layer. According to work of Yuan Teheng (1957) the influence of the sand layer on the ultimate bearing capacity could be assessed approximately by increasing the bearing capacity by a factor 1/(1 - 0.3 h/b) where h is the depth of sand layer and b is the width of footing. Experience suggests on the other hand that dispersion is considerably greater. Model test has indicated that dispersion angle is at least 30 to 45 for ultimate failure. However, it is understood that sand in model under small confining pressure behaves quite differently from sand layers at considerable depth under field conditions.

12.0 SETTLEMENT ANALYSIS :

The allowable magnitude of the vertical displacements is vital information for the foundation engineer. With this knowledge and information on subsoil conditions, it is possible for him to select from the economical point of view the proper type of foundation.

When a mass of soil is subjected to a foundation load the soil undergoes some settlement. Settlement of soil under load does not occur instantenously. Although some settlement takes place as soon as the load is imposed on the soil, usually the greater part develops slowly.

During the foundation design it is to be considered that :

a) The soil should not fail in shear under the external loads.
b) The settlement and differential settlements of the structure resulting from the compressibility and displacements of the soil below, should not endanger the safety or utility of the structure.

The most important causes of the settlement of structures are those involving the mechanical properties of the soil under loads which are good enough to produce shear distortion and consolidation. This can be predicted with reasonable limits by an analysis of the stress developed in the soil and of the reaction of the soil to stresses. Any settlement calculation at best be approximated. In addition, all the settlement calculations are generally based on assumption that foundation loads are known in magnitude and distribution.

12.1 EVALUATION OF SETTLEMENTS :

The basis for all settlement calculations in practice at present is the assumption of one dimensional compression of all layers. The assumption is accurate enough for deep seated layers. In case of very shallow and small sized foundation, the settlement may be due to lateral bulging of the soil immediately below the foundation. In such cases, there is no simple and reliable method of estimating settlement. However, in this cases, the bearing capacity criteria mostly governs the design.

Computations of Settlement of Clay.

If the clay is saturated, the settlement may be computed by the equation.

$$S = \frac{H}{1 + eo} \; ec \log_{10} \frac{Po + \Delta P}{Po}$$

where S = Settlement of the layer in cm

Δ P = Increase in vertical stress in the layer in Kg/cm2

H = Thickness of the layer in cm

eo = Initial void ratio

Po = Initial vertical pressure in Kg/cm2

ec = compression index of soil layer

Value of p always corresponds to the middle of the layer. This equation is applicable only to normally consolidated soils i.e, the maximum previous consolidation pressure is equal to present initial overburden presure. If the soil is pre-consolidated, a more realistic value of settlement may be computed by using the pre-consolidation pressure Po' rather than the existing overburden pressure Po. In the equation. Terzaghi and Peck proposed that if the ratio of $\frac{\Delta P}{(Po' - Po)} < 0.5$, the compression of stratum would be from 10 to 25 percent of the computed value (using overburden pressure Po) and if the ratio $\frac{P}{(Po' - Po)} > 1$ the computed value of settlement could be used. (Po' = consolidation pressure from e - log P curve).

12.2 PRIMARY CONSOLIDATION SETTLEMENTS :

Primary consolidation settlement occurs during the time of an excess pore-pressure gradient caused by a stress change in the stratum of interest. Additional settlements termed secondary compression continue for some additional time.

12.3 SECONDARY COMPRESSION :

When the excess pore-pressure due to consolidation has been dissipated, the change of void ratio continues but generally at a reduced rate. During secondary compression some of the highly viscous water between the points of contact is forced out from the grain. In many soil deposits secondary compression is so small that it can be neglected.

12.4 HOW TO ESTIMATE WHETHER THE SOIL IS PRE-CONSOLIDATED

Casagrande (1936) developed a method for estimating the maximum past effective stress from the laboratory compression curve of an undisturbed sample. He developed this method from observations on slow cyclic compression tests on undisturbed clay samples. The basic steps in Casagrande's method are illustrated in Fig. 7.8 and described below.

1. Plot the laboratory compression curve on semi-log paper.
2. Determine by eye the sharpest curvature.
3. Draw a tangent at the curvature point.
4. Draw a horizontal line through the largest point and bisect the angle α produced.

5. Project the straight-line portion of the laboratory curve to its intersection with the line bisecting the angle. This point of intersection approximaters the maximum past effective stress to which the soil has been subjected.

If the maximum past effective stress is approximately equal to existing effective stress, then the soil is normally consolidated. If the maximum past efective stress is greater than the existing effective stress, then the soil is over consolidated. Settlement computations for pre-consolidated soil are almost similar. If $Po + \Delta P < Po'$ the amount $P1 = PO + \Delta P - Po$ is used together with PO to compute the compression.

Compute settlement is less in pre-consolidated soil than normally loaded soil. If e-log P curve does not clearly distinguish pre-consolidation other factors should be examined. The natural moisture content is one indicator of pre-consolidation. If it is closer to plastic limit (PL) than to liquid limit (LL), the clay is almost certainly pre-consolidated. This is based on the reasoning that a pre-consolidated soil is more dense and thus if it is saturated the moisture content will be lower than the same soil in a less dense state.

Consolidation settlements predicted on the above basis have been found to be reliable if the laboratory samples were of good quality.

13.0 LOAD CAPACITY OF PILES :

Introduction : When the surface soil deposits exhibits low bearing capacity and the compressibility of the soil is found high for the required load to be placed on the ground surface, the foundation engineer has to investigated deeper strata to support the proposed load with smaller vertical displacements, therefore making use of a deep foundation with piles and piers. The design and behaviour of pile and pier foundation have always been the concern of foundation engineer and may be considered one of the less investigated fields in soil mechanics. The design of pile foundation, in most cases, is achieved by means of empirical rules and experience supplemented by simple pile tests.

Pile may be of timber, concrete or steel. A pile is defined as a rigid and strong structural member driven by means of hammer causing compaction of soil in the neighbourhood of the pile. The subsoil usually encountered in pile and pier problems is complex, since there elements are driven through stratified subsoil conditions where the layers may have different mechanical properties. Piles should be designed according to their ability to carry load under an allowable settlement in group action, and not by the individual ultimate bearing capacity and vertical displacement obtained in customary single unit tests.

The pile foundation problem will be divided into two parts, the stability problems and elasticity problem. The first concerns the pile or pile group breaking into the ground because it exceeds the shear strength of the soil material where it is

supported. The second problem concerns total and differential settlements of a pile or pile group.

The stability problem is understood by the ultimate load capacity of a pile a) by the bearing capacity and b) by the ultimate friction load capacity.

a) The ultimate bearing capacity is known to be a function of the geometrical dimensions, shape and roughness of the point, confining the pressure at the point, shear strength and density and compressibility of the soil under and around the point of the pile after driving.

b) The ultimate friction load capacity is a function of the shear strength of the soil along the pile shaft. In clays the time element is important. Highly sensitive clays never regain their total strength lost during pile driving and therefore a minimum strength is encountered at certain distance from the pile shaft.

13.1 PILE CAPACITY :

The capacity i.e. load taken by a pile of given length and cross section can be determined from the followings :

- by using the soil properties and determining the supporting strength
- by pile driving formula
- by load test

Whatever may be the procedure, the capacity of individual pile should not exceed the structural strength of pile. Load test is the best and reliable method for determining the ultimage load capacity but this should be limited to large jobs only. Otherwise conservative design may be made by using soil properties and varifying the capacity of pile during actual driving by using pile driving formula. Here, pile capacity depending on the soil properties are mentioned below.

13.2 PILE CAPACITY ON COHESIONLESS SOIL :

Piles are frequently driven in cohesoinless soil through loose cohesionless deposits or soft to very soft cohesive soil into a dense cohesionless soil for transmission of load into the firm strata. In this case the pile acts as end bearing pile and the capacity of pile depends on partly from end bearing by pile tip and partly from the skin resistance of driven into firm strata. The ultimate bearing capacity of pile can be completed as follows :

$$Qult = \pi R^2 (\gamma DNq + 0.6 R\gamma N\gamma) + 2\pi Rhfs \text{ (for round pile)}$$

$$Qult = B^2 (\gamma DNq + 0.4 B\gamma N\gamma) + 4Bhfs \text{ (for square pile)}$$

where Qult = ultimate bearing capacity of individual pile

R = radius of pile for round one
B = breadth of pile for around one
C = cohesion of soil
D = total penetration of pile in soil
h = depth of penetration in supporting layer
ϕ = angle of internal friction of the supporting soil
fs = $\gamma(D-h/2)\tan\phi$

For friction piles in cohesionless soil :

$$Qult = 2 \quad RL(Z + q) K \tan \phi \text{ (round pile)}$$

or $$Qult = 4 BL(Z + q) K \tan \phi \text{ (square pile)}$$

where L = total length of embedment of pile

K = coeff. of lateral earth pressure (0.5 to 1.00 for loose to dense sand)

q = permanent surcharge load

Z = depth of centre of gravity of the embedded portion of the pile.

13.3 PILE CAPACITY IN COHESIVE SOIL :

A friction pile in clay is supported by adhesion between the pile and the soil. The bearing capacity of such pile is approximately equal to the unit adhesion multiplied by the embedded surface area of pile. The adhesion is equal to the cohesive strength of the clay. For friction piles in clay the ultimate bearing can be computed as follows :

$$Qult = CLP$$

where C = cohesion of soil

L = length of pile embedded in the soil

P = perimeter of pile

13.4 PILE GROUP :

It has been recognized that groups of closely spaced piles in cohesive soils settle more than a single pile carrying the same load as the average for the group. Model tests have confirmed that at failure of a group the average load per pile is less than the ultimate bearing capacity of a single pile. Both these

effects becomes more pronouced as the centre to centre spacing, 'S', of the pile is reduced. Realistic design of closely spaced groups is therefore difficult and empirical efficiency formulae are frequently used in calculation of group bearing capacity. Some of the formulae take account of the spacing of piles generally expressed in the form of a ratio to the pile diameter, S/d. Alternatively the over all bearing capacity of the block of soil defined by the pile group is calculated. It is recommended that the group bearing capacity should be calculated both by efficiency formula and by the block method and work the lower of the two values.

In case of pile foundations where piles transmit the load of the structure directly to the underlying soil, partly by end bearing and partly through skin friction around the part of the pile embedded into the bearing layer, the foundation may be treated as deep footing considering one block. The capacity based on the peripherial shear and bearing capacity of the block at the pile ends.

$$Qult = SLP + q_{ult} A.$$

In case of friction piles such as in clay, group capacity is the peripherial shear only.

$$Qult = SLP$$

S = Shear strength = 1/2 qu

L = Length of whole pile embedded in the soil in case of friction pile.

Length of pile embedded in bearing layer in case of bearing pile.

P = Perimeter of area enclosing the piles in group.

A = Area enclosing piles in group.

Qult = Ultimate bearing capacity of soil at the level of pile tips and which may be determined from Terzaghis bearing capacity formula.

13.5 GROUP EFFICIENCY FACTOR :

Since the present knowledge is not sufficient to evaluate the efficiency for different spacing of piles, it is quite conservative to assume an efficiency factor of unity for all practical purposes. It is normally assumed that efficiency falls to unity when the spacing is increased to five or six diameters.

The Converse-Labarre formula is one of the most widely used group efficiency equation and is expressed as

$$Eg = 1 - \frac{\theta\,(n-1)m + (m-1)n}{90\ mn.}$$

where m = number of columns of piles in a group.

n = number of rows
θ = tan - 1 d/s in degree
d = diameter of pile
S = Spacing of pile.

Q g n = Eg n Qu.

When Qgn = Group load carrying capacity of pile.

Eg = Efficiency factor
n = number of piles
Qu = bearing capacity of individual pile.

14.0 SETTLEMENT OF PILE GROUPS IN COHESIVE LAYER :

The total settlements of pile groups may be calculated by making use of comsolidation settlement equations. The problem involved here is to evaluate the increase in stress 'ΔP' beneath pile group when the group is subjected to a vertical load 'P'. The computation of stresses depends on the type of soil through which the pile passes.

i) If the soil is of homogenous clay, the load is assumed to act on a fictious footing at a depth of 2/3rd from the surface and distributed over the sectional area of the group. The load on the pile group acting at this level is assumed to spread out at 2:1 slope.

ii) Where the pile passes through a very weak layer of depth 'D' and lower portion of length D2 is embedded in strong layer, the load is asumed to act at a depth equal to 2/3rd D2 below the surface of that strong layer and spreads at 2:1 slope.

iii) In case of point bearing pile supported on a firm strata, the load is assumed to act at the level of the firm strata and spread out at 2:1 slope.

Allowable settlement for piles should be same as shallow foundation.

15.0 TOLERANCE OF STRUCTURES TO SETTLEMENT :

Every structure needs a foundation to transfer its weight to the underlying soils. The construction of foundation introduces new conditions in the soils. The determination of what amounts of differential settlements can be tolerated by a structure has been studied by evaluation of conditions under which measured settlements have caused distress. The different results found with rigid and flexible structures shows that no rules can be generalized for all structures. No answer can be expected to the question of how much settlement a structure can tolerate. For each type of structure there will be different answer which most consider settlement, and amount and rate of differential settlement as measured by the slope generated by adjacent supports.

Karl Terzaghi states "Differential Settlement must be considered inevitable for every foundation, unless the foundation is supported by solid rock. Consequently, it is necessary to find out by careful inspection of buildings with known settlement records how much distorion the different types of construction can stand without any harm".

The foundation standards of the USSR allot different tolerances for low and tall structures, for wall bearings and framed buildings. Also there are different tolerances for structures on granular and on plastic soils. It has been observed that very rigid structures are most damaged by differential settlements. There are many examples of structures with relatively large

settlements and differential settlements that are maintained in use and probably the two best known examples are the Tower of Pisa and the Palace of fine arts in Mexico.

D.E. Polshin and R.A. Toker summarize the maximum allowable non-uniform settlement of structure based on field observation of 100 sites, and then prepared the USSR code of 1958 as given below.

PERMISSIBLE SETTLEMENT BY USSR CODE :

Structure	On sand or hard clay	On palstic clay	Average settlement
Steel & concrete Frame.	0.002 L	0.002 L	10
End rows of brick clad Frame.	0.007	0.001	15
Multistory brick wall L/H to 3	0.003	0.004 8	for L/H over 2.5
Multistory brick wall L/H over 5	0.005	0.007 10	for L/H below 1.5
One stay mill building	0.001	0.001	

Subsoil deformation computed must be less than the ultimate deformation depending on the strength of the structure.

15.1 PERMISSIBLE MAXIMUM SETTLEMENTS :

The calculated differential settlements are usually not the same as the actual diferential settlements. This is due to the stiffness of the structure which results in a redistribution of foundation loads with settlements and also due to inhomogeneties

in the soil profile. Skempton and Maedonal (1956) has observed that maximum settlement in case of individual footing and 40 to 60% in case of rafts. On the basis of those observations they gave the following table :

Foundation type	Soil	Permissible Settlement in (cm)
Individual Footing	Clay	6
	Sand	4
Raft	Clay	6 - 10
Rigid Box Type Structure	Sand	4 - 6
	Clay	10 and more

The Russian practice is to allow high values from settlement observation studies, it is concluded, that the structures can withstand much higher rotation and differential settlements that it can be provided by Statistical Analysis. (Table 7).

In case of piles also, settlement occurs where foundation is located in hard soil below which there is compressible layer of soil depending upon loading and settlement the compressible layer may have to be penetrated. The procedure for settlement computation of piles is the same as that for rigid footing once the location of footing is established for the pile type. The stress are approximated by sixty degree distribution method.

15.2 CONTROL OF SETTLEMENTS :

Since vertical movements of structures placed on a soil are caused by the imposed loads, two solutions to reduce the settlements are immediately obvious. One method is to pre-load the soil with a load as larger as or larger than the probable future load and allow settlement to take place. The second

method is to excavate as much material as the structure is estimated to weight, then when the structure is built the soil undergoes a net increase in pressure of approximately zero.

The second thought of reducing the total and differential settlement of a foundation by decreasing the net applied load by excavating is called floating. The floating idea is partly applied to water retention structures. The principle of floating foundation is that the net load applied to the foundation should be small enough that virgin consolidation must not occur.

16.0 CONSTRAIN :

In Water Development Board most of the hydraulic structures are constructed in isolated places where pre-loading arrangement is difficult to maintain. Moreover, it is costly affairs. Floating principle does not apply as all the structures are to carry enough surcharge load. The embankments which were constructed long ago has not been properly consolidated due to seepafe flow. Arrangements for pore-water pressure release could not be done by sand drains as embankment may fail. Due to normal load it can not be consolidated at foundation level as a 12-0" height of embankment exerts on 750 lb/sft at bottom which is not sufficient to release pore water pressure and act to pre-compress the soil.

Moreover, as the works are not properly planned due to fund, decision regarding foundation is taken mostly with only 'SPT' value. SPT values are not the only parameter for foundation design. In clay soil detail soil analysis are required before

giving any decision. Excessive settlement in pier-part may destroy P.V.C. resulting in piping & failure.

It is suggested that, in future, soil investigation should be arranged in such a way that during design, complete analysis of soil data is available, otherwise, design done with speculation or mere estimate will not only be uneconomical, but also total cost of exploration will be fruitless.

16.1 RECOMMENDATIONS :

In poor soil, economical foundation can be provided if all the soil parameters such as cohesion, angle of internal friction, Cc value, void ratio etc. are known.

i) Top layer of soft soil may be removed and sand filling is to be done. If the load is small settlement will not be high.

ii) Foundation may be placed in sandy layer and settlement checked for soft layers below sand.

iii) Pile can be provided upto 1st hard layer and settlement is to be checked. If it is within limit there is no need to penetrate clay layer.

iv) If compressible layer exists for a major depth in case of a small structure, top soil is to be removed. The structure will be designed to take care of settlement. P.V.C. water stop should not be used. It will be a rigid structure.

v) Where the G.W.T. is below foundation and at least 15 ft. of soil is loose below foundation possibility of composite pile is to be used. Below G.W.T. Timberpiles may be used over which compacted sand should be provided. Load will be reduced at the top of pile.

vi) There is no need for soil treatment in WRS unless the top soil is very loose.

vii) Surface dewatering should never be used.

DESIGN EXAMPLES :

Example 1.

Main part of 3 vent (5'x6') Mrighi Khal sluice is shown in figure 7 and 8. This structure will be considered to be located in four sites whose bore logs are given in figure No. 9 - 12.

For design of foundation following steps are to be done :

a) To calculate the loads imposing on the structure.

b) To calculate gross and net foundation pressure.

c) Find out the level of foundation, position of ground water table and the existing ground level.

d) Set the design criteria of foundation design.

e) Check each bore hole and collect detail soil report.

f) Calculate bearing capacity and settlement of the structure

g) Find the suitability of foundation on the basis of design criteria.

The calculated total load of the structure as shown in the figure 7 and 8 under major heads are as follows :

D.L. : Concrete & others	= 527 k.
Embankment soil	= 442 k.
L.L. = H10 truck over the embankment	= 20 k.
Total	= 989 k.

The gross foundation pressure on a bearing area of 24'-0"x20'-6" = 492.0 sft. under different condition of the applied load are given below :

D.L.(Concrete & others) only = 1.07 k/sft.

D.L.(Concrete & Embankment soil) = 1.97 k/sft.

Full D.L. + L.L. = 2.01 k/sft.

(As the surcharge is 15'-0", L.L. effect may be neglected).

The net foundation pressure for full D.L. + L.L. condition

$= 2.01 \text{ k/sft} - \gamma D_f$

$= 2.01 \text{ k/sft} - 0.11 \times 15$

$= 2.01 - 1.65 = 0.36 \text{ k/sft.}$

The factor of safety (F.S.) of safe net bearing pressure against the net foundation pressure shall be as follows :

For cohesive soil FS = 3

For cohesionless soil FS = 2

As the foundation is raft, the settlement shall be limited to 2" total and 1" differential.

The ground water table is considered at foundation level as this will give the critical condition of design as full structural load is imposed but bearing capacity is still reduced to saturation of foundation soil.

BEARING CAPACITY : (Ref. Bore Log in Fig 9)

Below the foundation , soil is sandy, and minimum value of "N" is 10. So corresponding Ø value is taken 30°. (This is taken from table 5).

Hansen bearing capacity Equation is adopted.

$$Qult = 0.5 \; \gamma_{sub} \; N\gamma \; S\gamma \; d\gamma + \gamma_{moist} \; D_f \; Nq \; Sq \; dq$$

moist = 110 lb/sft.

sub = 52.60 lb/sft.

B = 20.50 Nq = 18.40 & N = 22.40 for 30°.

$$S = 1 - 0.4 \left(\frac{B}{L}\right) = 1 - 0.4 \left(\frac{20.5}{24}\right) = 1 - .34 = .66$$

d = 1.0

D_f = 15.0

$$Sq = 1 + \left(\frac{B}{L}\right) \tan \emptyset = 1 + \left(\frac{20.5}{24}\right) \tan 30^{o} = 1 + .493 = 1.493$$

$$dq = 1 + 2 \tan \emptyset \, (1 - \sin \emptyset)^{2} \, D/B.$$

$$1 + 2 \tan 30^{o} \, (1 - \sin 30^{o})^{2} \left(\frac{15}{20.5}\right)$$

$$= 1 + 2 \times .577 \, (1 - 0.5^{2}) \times 0.732$$

$$= 1 + 1.154 \times .25 \times .732 = 1.211$$

Incorporating the above values in the same equation.

$$Qult = 0.5 \times 20.5 \times 52.60 \times 22.40 \times .66 \times 1.0 + 115 \times 15 \times 18.40 \times 1.493 \times 1.211$$

$$= 7971.0 + 57387.0 = 65358.0 \text{ lb/sft.}$$

Before back filling and construction of embankment, the safe bearing capacity (without surcharge)

$$q_a = \frac{7971}{2} = 3985 \text{ lb/sft} > 1070 \text{ lb/sft OK.}$$

After back filling and construction of embankment (with surcharge), the net safe bearing capacity $= \frac{qult - \gamma D_f}{F}$

$$= \frac{65358 - 110 \times 15}{2} = 32583 \text{ lb/sft} = 32.58 \text{ k/sft} > 2.01 \text{ k/sft. OK.}$$

SETTLEMENT :

For N = 10, B = 20'-6", net soil pressure for 1" settlement

= 1.0 ton/sft. (from figure 5.)

With water level correction, net soil pressure = 1 k/sft.

For 2" settlement in case of raft, qa = 2 x 1 = 2 k/sft.>0.36 k. O.K.

Safe bearing capacity is controlled by settlement and foundation is safe against allowable bearing capacity failure and also from detrimental settlement.

Example 2.

The same structure placed in a site whose bore log is given in figure 10. Top soil is loose consisting of clay and trace sand upto 20'-0". Below that minimum 'N' value is 6. Foundation

level is at 15'-0" below G.L. So, below the foundation there will be 5'-0" loose layer. This layer should be replaced by sand to get adequate bearing pressure.

It is seen that after removing top 5'-0" below foundation there is one layer of clay located between 30-38 ft. Settlement of this layer is to be checked.

For N value 6, ϕ is taken 28°, and Hansen formula is applied.

$Nq = 14.72$ ¦
¦ From table 5.
$N = 16.72$ ¦

$$Qult = 0.5\ B\ \gamma_{sub}\ N_{\gamma}\ S_{\gamma}\ d_{\gamma} + \gamma_{moist}\ D_f\ Nq\ Sq\ dq$$

$$qult = 0.5 \times 20.5 \times 52.6 \times 16.72 \times .66 \times 1.0$$
$$+ 115 \times 14.72 \times 1.493 \times 1.211 \times 15$$
$$= 5949.62 + 45909.32 = 51858.94 \text{ lb/sft.}$$

Before back filling.

$$q = \frac{5949.0}{2} = 2974 \text{ lb/sft.} > 1070 \text{ lb/sft} = \text{OK.}$$

It is also found safe after construction of embankment.

SETTLEMENT :

Settlement is given by $S = \frac{C_c H}{1 + eo} \log \frac{P_o + \Delta P}{P_o}$

In this example

$C_c = 0.263$ ¦
¦ From test result.
$eo = 1.11$ ¦

$P_o = 0.30$ k.

The existing overburden pressure at mid-height of compressive layer located at 34'-0" below G.L. (19'-0" below structure base).

$P = 15 \times 110 + 19(115 - 62.4) = 1650 + 998 = 2648$ lb/sft.

Net foundation pressure without L.L. = 0.36 - 0.04 = 0.32 k/sft.

Increase in pressure at the middle of compressible layer in short direction considering 2:1 pressure distribution.

$$\Delta P = \frac{0.32 \times 20.5}{20.5 + 19.0} = 0.166 \text{ k/sft. } H = 8'\text{-}0" = 96 \text{ inch.}$$

$$S = \frac{0.263 \times 96}{1 + 1.11} \log_{10} \frac{2648 + 166}{2648} = 11.96 \times 0.026 = 0.31 \text{ inch.}$$

Allowable 2" inch. Hence OK. so the foundation is safe from bearing capacity and as well as settlement consideration.

Example 3.

If the same structure is located in soil type given by figure 11 shallow foundation for the structure is not possible to provide due to excessive settlement of under lying compressible soil.

It is suggested to give 30 ft. timber pile with average dia 8". At the bottom 6'-0" end bearing is taken and out of 24'-0" friction for 20'-0" is considered. Top 4 ft. is omitted for safety.

NUMBER OF PILES :

Cohesion = $40KN/m^2$ = 832 lb/sft. C_c = 0.153.

The ultimate bearing capacity from consideration of end bearing.

$Qult = \pi R^2 (\gamma DNq + 0.6R\gamma N\gamma) + 2\pi Rhfs$

Consider pile length = 30'-0". D = 30 + 15 = 45 ft.

h = 6'-0". fs = (D - h/2)tan ϕ. From soil type $\phi = 28^o$

Nq = 18.70

$N\gamma$ = 15.70 fs = 52.6 (45 - 3) x.53 = 1168 lb/sft.

Qult = 0.34(52.6 x 45 x 18.70 + 0.6 x 0.33 x 52.6 x 15.7)

+ 2 x 3.14 x .25 x 1168 x 6

= 0.34 (44263 + 163) + 11002 = 15104 + 11002 = 26106 lbs.

qa = 26106/3 = 8702 lb 8.7 k.

C = 832 lb/sft. Load/pile = $\pi DCL = 3.14 \times \frac{8}{12} \times 832 \times 20 = 34.83$ k.

F.S. = 3.

qa = 11.61 k. Load carried by each pile = 8.7+11.61 k.=20.31 k

$$\text{No. of pile required} = \frac{\text{Total Load}}{\text{Allowable load per pile}} = \frac{989 \text{ k}}{20.31}$$

= 48.70 = 50 Nos.(say)

Considering efficiency factor 63 Nos. of piles are arranged in group.

$$\text{Group efficiency EQ} = 1 - \frac{\phi (n-1) m + (m-1) n}{90 mn}$$

m = no. of columns, n = no. of rows.

$\phi = \tan^{-1} d/s$. d = dia of pile. s = spacing of pile.

d = 0.67, θ = 11.97

s = 3.10, m = 7, n = 9

$$Eg = 1 - \frac{11.97 \times 8 \times 7 + 6 \times 9}{90 \times 7 \times 9}$$

$$= 1 - \frac{670.37 + 54}{90 \times 63} = 1 - 0.127 = 0.873$$

Consider Eff. factor 0.87

qall = 20.31 x 63 x 0.87 = 1113 k > 989 k. Ok.

Example 4.

The same structure is located in a soil as shown by bore log in figure 12.

Below foundation there is continuous clay & silt soil of 'N' value from 3 to 7 upto 45 ft. So friction pile is suggested.

Consider 30'-0" length of timber pile of av. dia 8".

INDIVIDUAL ACTION :

Qult = CLP

$C = 23\ KN/m^2 = 478$ lb/sft.

qult = 478 x 30 x 2 = 28680 lb.

$$Qall = \frac{Qult}{F.S.} = 9.5 \text{ k/pile.}$$

$$\text{No. of piles} = \frac{\text{Total load capacity of structure}}{\text{Pile capacity}} = \frac{989\ k}{9.5\ k}$$

= 105 Nos. So it is not feasible to provide timber pile.

TRY WITH R.C.C. PILE :

Provide 35'-0" length pile. $\therefore$ Qult = 478 x 35 x 4 = 66920 lb.

Qall = 22.30 k. $\therefore$ No. of pile required = 45. Provide 48 Nos.

Perimeter of group = P = 2(20.5 + 24) = 89 sft.

L = 35'-0". S= 478 lb/sft.

Qult = SLP = (478 x 89 x 35) = 1489 k.

$$Qall = \frac{1489}{3} = 496\ k < 998k.\ \text{Not Ok.}$$

In this case pile length should be increased and pile numbers is to be reduced.

Consider 45'-0" pile

Qult = 478 x 45 x 4 = 86040 lb. Qall = 28.68 k.

No. of pile required = 35. Use 42 Nos.

Group Action :

Qult = SLP = 478 x 89 x 45 = 1914 k. Qall = 638 k < 989 k.

Bearing capacity of pile group at tip has not been considered. This will add to additional capacity of pile group. It is to be checked before finalising the design.

Group efficiency :

S = 3.8

m = 6

n = 7

$\theta = 14.74^{o}$

$$Eff : 1 - \frac{14.74 \times 6 \times 6 + 6 \times 7}{90 \times 6 \times 7}$$

$$= 1 - 0.15 = 0.85.$$

Q total = 28.68.x 42 x .85 = 1023 k > 989 k $\therefore$ Ok.

Settlement of pile groups is not required to be found out as below the piles there is medium stiff sandy soil layer.

Example 5.

In the same site (Bore log figure 11) provide 30 ft. R.C.C. pile with depth of penetration 7 ft. in cohesionless soil.

INDIVIDUAL ACTION :

Minimum N value on the bearing layer = 12, for N = 12, $\phi = 30^{o}$,

From Terzhaghis table value of Nq = 22.5, N = 19.70

fs = (D - 1/2 h) tan ϕ, D = 30 + 15 = 45'-0".

$fs = 52.5(45 - 3)\tan 30^{o}$

$= 52.5 \times 42 \times 0.572 = 1272$ lb/sft.

$Q_{ult} = B^{2} (\gamma DNq + 0.4 B\gamma N\gamma) + 4 Bhf_{s}$

$fs = 1 \times 1(45 \times 22.5 \times 52.6 + 0.4 \times 1 \times 52.5 \times 19.2)$

$+ 4 \times 1 \times 7 \times 1272 = (53257 + 404) + 35616 = 89277$ lb.

Qall = 44.6 k. No. of pile required = 22.2 use 24 Nos.

Capacity per pile = 41.20 k.

Group Action :

Perimeter = (20.5 + 24)2 = 89 ft.

Area of pile group = A = 20.5 x 24 = 492 sft. C = 438 lb/sft.

Ultimate bearing capacity of pile group at tips.

$Qall = \beta BDN + \gamma D N_{f q}$ $\beta = 0.5 - .1 B/L = 0.5 - 0.08 = 0.42.$

$= 0.42 \times 20.5 \times 45 \times 19.7 + 52.6 \times 45 \times 22.5$

$= 7633.0 + 53257 = 60890$ lb/sft.

Ultimate group capacity of pile = SLP + qult xA.

$$= 438 \times 7 \times 89 + 60890 \times 492 = (272874 + 29957880) = 30230 \text{ kip.}$$

$$\text{Allowable group capacity} = \frac{30230}{2} = 15115 > 989 \text{ k. Ok.}$$

$$Eg = 1 - \frac{9.46 \times 5 \times 4 + 3 \times 6}{90 \times 6 \times 4} = 1 - \frac{189.20 + 18}{90 \times 24} = 1 - .090$$

$$= 0.90.$$

$$Qall = 44.6 \times 24 \times .90 = 963 < 989 \text{ k.}$$

$$\text{Again } \tan O = 1/4.3,\ O = 13.09^{o} \qquad Eg = 1 - \frac{13.09 \times 3 \times 6 + 5 \times 4}{90 \times 6 \times 4}$$

$$m = 6, \quad n = 4 \qquad Eg = 1 - 0.118 = 0.88$$

Provide 28 Nos. of piles or 32'-0" length 24 Nos. of piles.

Example 6.

Foundation is located below 18'-0". At foundation level there is medium dense sand. Below the sandy layer there is soft clay of 10'-0" deep. This layer is likely to settle. Laboratory consolidation curve is given. (figure 13).

Assume the following parameters

eo = 1.28, Load/sft. = 1.0 tons
Cc = 0.350. Size of structure 25' x 35' (B x L)
$\gamma_{sat} = 115 \text{ lb/ft}^3$

a) check whether the soil is pre-loaded.
b) Find out settlement.
c) Upper sand layer is 15'-0" deep.

On enquiry, as location of bore-logs were not available, it was decided that there should be another boring. The new boring indicates that at a depth 20 to 30 ft, sand layers were available for a depth 16 to 25 ft with N values 8 to 12. Below sand, there was compressible layer of 26 to 30 ft. How to tackle this poor soil. (Ref. Bore log figure 16).

Solution :

The bridge design was changed. In place of cantilever bridge, simply supported bridge with span 20' - 0" was designed. Length of approach slab provided was 15' - 0" with end support over retaining wall. Retaining wall will be supported over piles.

Maximum load in central pier was calculated to be 185 kips. (including caisson weight and live load)

Bearing area of caisson : 75 sft.

$$qult = 4N^2 B R_w + 12 (100 + N^2) D R'_w$$

$N = 7$ (min), $d = 20' - 0"$

$$qult = 4 \times 49 \times 10 \times .5 + 12 (100 + 49) \times 20 \times 0.50$$

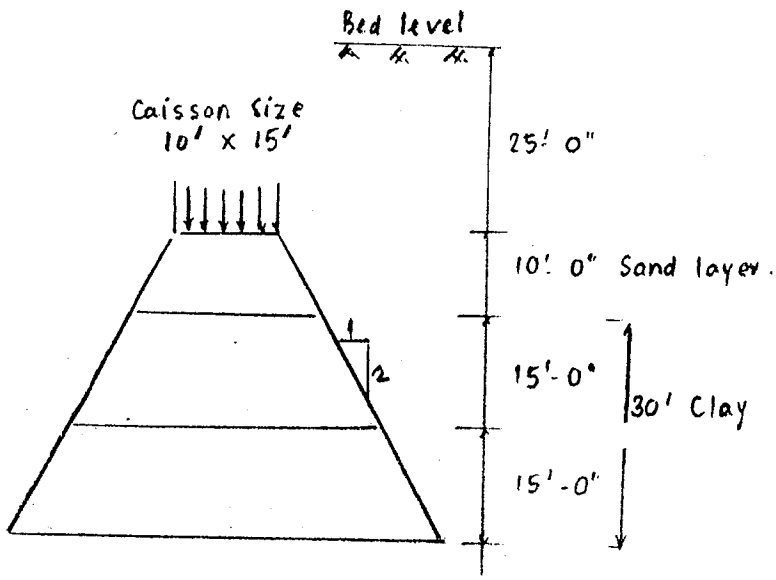
$$= 18.86 \text{ K}$$

$F.S. = 3$, $q_{all} = 6.3$ K.

Total load that can be taken = 6.3 x 75 = 472 K > 185 K, So bearing is enough to take care of vertical load.

Settlement Calculation :

The middle sand layer which has variable thickness from 16 ft to 30 ft was checked. Foundation was placed in such a way that it has at least 10 ft supporting layer.



Po = 50 (115 - 62.4) = 2.63 k/sft.
eo = 1.53
Cc = 0.58

$$P = \frac{185.00}{(10 + 25)\ (15.67 + 25)} = 0.13 \text{ k/sft.}$$

$$S = \frac{0.58 \times 30 \times 12}{1 + 1.53} \log \frac{2.60 + 0.13}{2.60}$$

$$= 82.53 \times .0211 = 1.75"$$

Without Liveload P = 0.113 k therefore S = 1.52 inch

At first support load is 135 k.

P = 0.094 lb/sft.

S = 82.53 x .0156 = 1.28 inch

Solution :

A tangent is drawn at maximum curvature point. A horizontal line is also drawn. The angle is divided and intersection of bisector with virgin curve is found out. Po' = 0.8 ton/sft.

Foundation level is 12' - 0" below G.W.T. At center of clay layer settlement is to be calculated.

$$Po = 115 \times 12 + (115 - 62.4) \times 26 = 2747.6 \text{ lb/sft.} = 1.37 \text{ Tsf}$$

As Po' < Po, it is not pre-loaded.

$$\gamma D_f = 115 \times 12 + (115 - 62.4) \times 6 = 1695.6$$

$$\text{Net load/sft} = 2000 - \gamma D_f = 304 \text{ lb}$$

P at centre of compressive layer

$$P = \frac{0.304 \times 25 \times 35}{(25 + 20) \times (35 + 20)} = 0.110 \text{ k}$$

$$S = \frac{Cc\ H}{1 + e_o} \log_{10} \frac{\Delta P + Po}{Po}$$

$$S = \frac{0.32 \times 120}{1 + 1.28} \log_{10} \frac{110 + 2747.6}{2747.6}$$

$$= 16.84 \times 0.017 = 0.29 \text{ inch.}$$

Beyond central part, load is approximately 0.375 T/sft.

No net pressure, settlement will not occur

Diferential settlement = 0.29 inch < 1.50 inch

So the structure is not likely to create problem.

Example 7.

Same soil condition. Consolidation curve as in figure 14.

Po' = 1.8 Ton/sft.
Cc = 0.188
eo = 1.05
Po = 2747.6 lb/sft. Po < Po'

So the soil is pre-consolidated. So there will be less settlement than calculated. Use Po' instead of Po.

$$\text{therefore } S = \frac{0.188 \times 120}{1 + 1.05} \log_{10} \frac{.055 + 1.80}{1.80}$$

$$= 11.00 \times 0.015 = 0.14 \text{ inch.}$$

So differential settlement is very low.

Pre-consolidated clay causes less settlement.

Although the foundation soil has high bearing capacity, it will cause settlement below.

For checking foundation, two considerations are to be given :

(1) whether foundation has allowable bearing pressure.

(2) whether there will be settlement in compressible layer located below.

Example 8.

Case Study : Madari khal Bridge : Chittagong.

In a bridge site bore-logs were given faulty. A balanced cantilever bridge of span 160' was designed. The 1st bore log indicated that at a depth of 30' - 0" below bed dense soil with minimum N value 20 is found. The deeper layer has higher consistency.

At support there may be small settlement will be within limit. Isolated footing can take care of higher settlement. It is with this consideration isolated short span bridge was designed.

Example 9.

A sluice or regulator consists of three major parts i.e, barrel, u/s transition and d/s transition. The barrel portion is the main part of the structure with high intensity of foundation pressure than the other parts. For this reason for bearing main part is vital. The u/s undergoes settlement, and then there will be differential settlement althrough and this may decrease actual settlement. A joint is provided by P.V.C. water stop in between three parts and if there is excessive settlement it may crack. So, the design will be done in such a way that no excessive settlement i.e. differential settlement takes place.

As an illustration main part of a typical 2 vent sluice is shown in fig. 6. The structure will be considered to be constructed in different type of soil to check the foundation suitability. For the design of foundation following steps are necessary.

1) Calculate loads imposed

2) Calculate gross and net foundation pressure

3) Set the design criteria by which fooundation will be checked

4) Find the level of foundation, position of groundwater table and the existing ground level

5) From geological information collect data for foundation checking

6) Compute bearing capacity and settlement if there is any compressible layer below.

The calculated total load of the structure as shwon in the fig. 6 under major heads are as follows :

DL	=	Concrete & other	:	392 kips
	=	Embankment soil	:	576 kips
LL	=	H 10 truck over the embankment	:	20 kips
		Total Load	:	988 kips

The gross foundation pressure on a bearing area of 38' x 14'-3" = 541.5 sft under dirrerent condition of the applied load are given below :

DL (concrete & others) only	:	0.72	k/sft.
DL (concrete & Embkt. soil)	:	1.79	k/sft.
Total DL + LL	:	1.82	k/sft.

SPT value - minimum 5

Ø value - 28° (from table , if the test values of O is given, use it)

C = O

From Hansen's bearing capacity equation for foundation on granular soil :

$$q_{ult} = 0.5\, B \gamma_s N_\gamma S_\gamma d + \gamma_{moist} D_f N_q S_q d_q$$

where for this example, keeping in mind the critical location of the ground water table (as discussed in earlier)

γ_{moist} = 110 lb/sft (above foundation level)
γ_{sub} = 115 - 62.4 = 52.6 lb/sft (below foundation level)
B = 14' - 3" = 14.25'

For $\phi = 28^o$ $N_q = 14.72$, $N_\gamma = 16.72$ (from table)

$S = 1 - 0.4\,(b/L) = 1 - 0.4\,(14.25/38.0) = 0.85$

$d = 1.00$

$D_f = 12.0$

$S_q = 1 + (B/L) \tan \phi = 1 + (14.25/38.0) \tan 28^o = 1.20$

$d_q = 1 + 2 \tan \phi\, (1 - \sin \phi)^2\, (D/B) \quad (D<B)$

$= 1 + 2 \tan 28^o\, (1 - \sin 28^o)^2\, (12.0/14.25) = 1.25$

Putting the above values into the bearing capacity equation, gives the gross ultimate bearing capacity with surcharge.

$q_{ult} = 0.5 \times 14.25 \times 52.6 \times 16.72 \times 0.85 \times 1.00 + 110 \times 12.0 \times 14.72 \times 1.20 \times 1.25$

$= 5326 \times 29146 = 34472$ lb/sft.

Before back filling and construction of embankment, the safe bearing capacity (without surcharge) is

$q = 5326/2 = 2663$ lb/sft. > 720 lb/sft.

The net foundation pressure for total DL + LL condition

$= 1.82 \text{ k/sft} - \gamma Df$

$= 1.82 - 0.110 \times 12.00$

$= 1.82 - 1.32 = 0.5 \text{ k/sft}$

The factor of safety (Fs) of safe (net) bearing pressure against the net foundation pressure shall be as follows :

For Cohesive soil : Fs = 3

For Cohesionless soil : Fs = 2

As the foundation is raft, the settlement shall be limited to as follows :

Total settlement = 2"

Diff. settlement = 1"

The ground water table usually fluctuates and in the flood season, the structure is submerged partially. Under this condition, it is a question where to select the maximum ground water table. When the structure is submerged, the bearing capacity is reduced and at the same time the load also reduced due to budyant effect. Therefor, it is preferable to assume ground water table at the foundation level because this will give the critical condition of design as full structural load is imposing but the bearing capacity still reduced due to saturation of foundation soil.

The foundation level is assumed at EL. + 8.50 and the existing ground level is at EL. + 20.50 for all the examples, thus, surcharge depth of 12 ft. as shown in the fig. 6.

Example. 10.

<u>Example for foundation analysis on cohesionless soil</u> : Assume that the structure in fig. 6 will be constructed on a soil which has foundation on sandy layer for a considerable depth. Now analyse the foundation i.e, compute the net allowable bearing capacity of the soil from both bearing capacity and settlement consideration and check whether the foundation soil is adequate to support the structure.

<u>Bearing capacity</u> : For bearing capacity calculation, the depth of soil below foundation level should be taken 2B where B is the minimum width of the foundation. In this case the depth of such soil is 28' - 6'. Within this depth the soil properties from collected from laboratory is as follows :

After backfilling and construction of embankment (with surcharge), the net safe bearing capacity

$$\frac{q_{ult} - \gamma D_f}{F} = \frac{34472 - 12 \times 110}{2} = 16576 \text{ lb}$$

$$= 16.58 \text{ k/sft.}$$

$$> 1.82 \text{ k/sft. O.K.}$$

Settlement :

For N = 5 , B = 14' - 3", net soil pressure

for 1' settlement = 0.5 ton/sft = 1 k/sft (from fig. 7.)

With water level correction, net soil pressure = 0.5 k/sft.

For 2" settlement in case of raft, q_a = 2 x 0.5 = 1 k/sft > 0.5 k/sft.

(net foundation pressure)

From the above computaiton, it is seen that safe bearing capacity is controlled by settlement and thus the design or allowable soil pressure is 1 k/sft and the foundation is safe against bearing capacity failure and also from detrimental settlement.

Example. 11.

Example for foundation analysis on cohesive soil : The same structure with the applied load if required to be constructed on a foundation with clay and silty soil above the base level and fine sand below the base having low N value.

From the soil test data, following information is collected.

$C = qu/2$ (for $\phi = 0^{o}$ condition)

$= 11.33/2 = 5.6$ psi $= 815$ psf

$C_c = 0.20$

$e_o = 0.40$

Bearing capacity :

For this type of soil ($\phi = 0^{o}$),

$$qult = 5.14\ C(1 + S_c + d'c) + \gamma D_f$$

where, $C = 815$ psf

$\gamma_{moist} = 110$ lb/sft.

$Df = 12' = 0'$

$$S_c = 0.2B/L = 0.2 \times \frac{14.25}{38} = 0.075$$

$$d_c = 0.4\ D/B = 0.4 \times \frac{12}{14.25} = 0.337$$

$$Qult = 5.14 \times 815\ (1 + 0.075 + 0.337) + 110 \times 12 = 7235\ lb/sf$$

$$Qnet = \frac{Qult - \gamma Df}{F} = \frac{7235 - 110 \times 12}{3} = 1972\ lb/sft > 500\ lb/sft.\ O.K$$

Settlement

The magnitude of settlement due to consolidation of cohesive soil can be calculated by the following equation :

$$S = \frac{CcH}{1 + eo} \log \frac{Po + \Delta P}{Po}$$

In this example, Cc = 0.20, eo = 0.40

The existing overburden pressure at mid height of compresible layer

$$Po = 12 \times 110 + 14.00\ (115 - 62.50) = 2055\ lb/sft.$$

Increase in pressure at the mid height of compressible layer :

$$P = 0.47 \times 14.25 = .237 k/sft$$

$$H = 28'\ -0"$$

Considering 2:1 pressure distributation.

$$\text{Now } S = \frac{CcH}{1 + eo} \log \frac{Po + \Delta P}{Po}$$

$$\text{therefore } \frac{0.20 \times (28 \times 12)}{1 + 0.40} \log \frac{2055 + 237}{2055}$$

$$= 2.27 > \text{allowable } 2' \text{ not OK.}$$

However the distribution has been considered in Shorter direction. Distribution in both the direction will form a truncated prism.

$$\text{So} \quad P = \frac{0.47 \times 14.25 \times 38}{(14 + 14.25)\ (38 + 14)} = 0.173\ k/sft.$$

$$\text{therefore new } S = \frac{(0.20 \times 28 \times 12)}{1 + 0.40} \log \frac{2055 + 173}{2055}$$

$$= 48 \times 0.0355 = 1.70 < 2''$$

In other part load is less than imposed. From differential settlement point it is not OK.

In such cases, the foundation may be improved by pre-consolidation of compressible layer or by transferring the load to hard stratum. This depends upon comparative analysis and technique at the site.

(Net foundation pressure is considered after deducting effect of live load.)

If same type of structure is to be palced in soil as shown in Fig. 6 at 10' -0" below ground level what should be solution. Below foundation there is loose soil for 23' ft depth. After that medium dense sand of vairable "SPT" value is avilable. If timber pile is provided, spacing will be very close. Moreover a better solution is to replace soil. If below foundation 6' -0" soil is removed and replaced by compacted sand, the middle layer will act as compressible layer between two compacted layer of sand. Moreover the bearing value will be high at foundation level. Following data is found from soil analysis.

$Cc = 0.20$
$eo = 0.92$
$qu = 51.08\ KN/m^2$

Net foundation pressure

$= 1.82\ s/sft - 0.11 \times 10$
$= 1.82 - 1.10 = 0.72\ k/sft.$

Excluding live load net P = 0.69 k/sft. Existing over burden at mid height of compressible layer

$$Po = 10 \times 110 + (6 + 8.5) \times 53$$
$$= 1100 + 768 = 1868 \text{ lb/sft.}$$

Increase in pressure at mid height of compressible layer

$$P = \frac{0.69 \times 14.25 \times 38}{28.25 \times 52} = 0.254 \text{ k/sft}$$

$$S = \frac{17 \times 12 \times .211}{1 + 0.92} \log \frac{1868 + 259}{1868}$$

$$= 22.42 \times 0.055 = 1.24 \text{ inch} < 2 \text{ inch.}$$

However to limit settlement to 1", it is better to remove 8' - 0" of soil.

Example 12.

A bridge of span 120 ft. has to be located in a place where foundation soil is very poor (bore log enclosed, Fig. 11) and there is no alternate site with better soubsoil condition is available.

Solution : As the soil is poor the bridge is designed with a few simply supported spans. Any settlement if occurs will affect the structure to a leser degree than continuous bridge.

Load at central column is 190 K.

Soil parameters : N value 4

qu = o.45 Tsf ; C = 450 k/sft.

Av. Pile dia = 8".

Length of pile = 40 ft.

Timber pile will be used with joint. Allowable cohesion = 150 lb/sft.

Load per pile :

Individual action : πDCL

3.14 x 0.67 x 150 x 40 = 12.62 kip.

Number of pile required = 190/12.62 = 15 Nos.

Allow 18 Nos. for arranging the group.

m = No. of column = 6

n = No. of rows = 3

S = C/C spacing = 3.25'

d = Average dia = 8" = 0.67'

GROUP ACTION :

Perimeter of block = 2 (17 + 7) = 48 ft.

Length of pile = 40 ft.

Cohesion = 150 psf.

Allowable = 48 x 40 x 150 = 288 K. > 190 K.

GROUP EFFICIENCY FACTOR :

$$Eg = 1 - \frac{\phi(n - 1)m + (m - 1)n}{90\ mn}$$

$$q = \tan^{-1} d/s \qquad \text{therefore } \phi = 11.64^{o}$$

$$Eg = 1 - \frac{11.64\ (3 - 1)6 + (6 - 1)3}{90 \times 3 \times 6}$$

$$= 1 - 0.095 = 0.905$$

Total allowable load on piles = 12.62 x 18 x .905 = 205 k. > 190 k

So the pile design is OK.

SETTLEMENT :

As it is located in homogeneous clay, load is assumed to distributed at 2/3rd from top of pile.

Thickness of clay layer = 20 ft.

Depth of center of layer = 24 ft. from 2/3rd pile depth.

$$\text{therefore} \quad P = \frac{190}{(12 + 12 + 7) \times (12 + 12 + 17)} = \frac{190}{31 \times 41}$$

$$= 0.150 \text{ k/sft.}$$

$$eo = 1.10$$

$$Cc = 0.28$$

The whole layer is considered homogenous and settlement is considered in one layer only.

$$Po = \gamma_{sub} (26 + 14 + 10) = 52.6 \times 50 = 2630 \text{ lb/sft}$$

$$S = \frac{H}{1 + eo} Cc \log_{10} \frac{Po + \Delta P}{Po} = \frac{20 \times 12}{1 + 1.10} \times 0.28 \times \log_{10} \frac{2630 + 150}{2630}$$

$$= 0.77 \text{ inch.}$$

This is actual settlement. Adjoining column may also settle. So it is considered stable from settlement consideration.

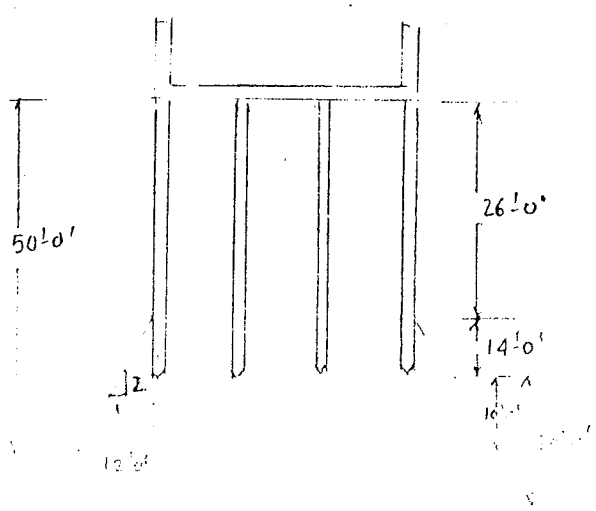


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Table 1: Relative Density of Granular Soil

COMPACTNESS	RELATIVE DENSITY (%)	SPT VALUES (N)	ANGLE OF INTERNAL FRICTION (Ø)	FIELD IDENTIFICATION
VERY LOOSE	0 - 15	0 - 4	0 - 28°	Reinforcing rod can be pushc into soil several feet.
LOOSE	15 - 35	4 - 10	28°- 30°	
MEDIUM	35 - 65	10 - 30	30°- 36°	
DENSE	65 - 85	30 - 50	36°-41°	Difficult to drive 2" x 4" stake with a sledge hammer
VERY DENSE	85 - 100	50 & above	41° and above	

Table 2: Consistency of Cohesive Soil

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH TONS/FT^2	STANDARD PENETRATION N BLOWS/FT	FIELD IDENFIFICATION
VERY SOFT	LESS THAN 0.25	BELOW 2	EASILY PENETRATED SEVERAL INCHES BY FIST
SOFT	0.25 - 0.50	2 - 4	EASILY PENETRATED BY THUMB
MEDIUM	0.50 - 1.00	4 - 8	CAN BE PENETRATED BY THUMB WITH MODERATE EFFORT
STIFF	1.00 - 2.00	8 - 15	READILY INDENTED BY THUMB BUT PENETRATED ONLY BY GREAT EFFORT
VERY STIFF	2.00 - 4.00	15 - 30	READILY INDENTED BY THUMB NAIL
HARD	OVER 4.00	OVER 30	INDENTED WITH DIFFICULTY BY THUMB NAIL

NOTE: The measurement of consistency by means of the standard penetration test can be unreliable.

Table 3: Shape Factors for Terzhagi's Equation

Shape of footing	Continuous	Square	Rectangular	Circular
α	1.0	1.3	$1 + 0.3 \frac{B}{L}$	1.3
β	0.5	0.4	$0.5 - 0.1 \frac{B}{L}$	0.3

Table 4: Bearing Capacity Factors for Terzhagi's Equation

	N_c	N_q	N_γ
5	5.7	1.0	0.0
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	35.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
48	258.3	287.9	780.1
50	347.5	415.1	1153.2

Table 5: Bearing Capacity Factors for Hansen's Equation

ø	Nc	Nq	N	Nq/Nc	tan ø
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.36	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.91	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84

Table 6: Shape and Depth Factors for Hansen's Equations

Shape Factors:

$$S'_c = 0.2\ (B/L)$$

$$S_c = 1 + N_q B/N_c L$$

$$S_q = 1 + (B/L)\ \tan\phi$$

$$S = 1 - 0.4\ (B/L)$$

Depth Factors:

$$d'_c = 0.4\ (D/B) \qquad D \leq B$$

$$d'_c = 0.4\ \tan^{-1}(D/B) \qquad D > B$$

$$d_c = 1 + 0.4\ (D/B) \qquad D \leq B$$

$$d_c = 1 + 0.4\ \tan^{-1}(D/B) \qquad D > B$$

$$d_q = 1 + 2\ \tan\phi\ (1 - \sin\phi)^2 (D/B) \qquad D \leq B$$

$$d_q = 1 + 2\ \tan\phi\ (1 - \sin\phi)^2 \tan^{-1}(D/B) \qquad D > B$$

$$d_\gamma = 1.00 \text{ for all } \phi$$

Table 7: Maximum Differential Settlement Permitted by U.S.S.R. Building Code*

Item No.	*Description of standard value*	*Subsoil*	
		Sand and hard clay	*Plastic clay*
1.	Slope of crane way as well as tracks for bridge crane truck	0.003	0.003
2.	Difference in settlement of civil and industrial building column foundations:		
	(a) for steel and reinforced concrete structures,	$0.002L$	$0.002L$
	(b) for end rows of columns with brick cladding,	$0.007L$	$0.001L$
	(c) for structures where auxiliary strain does not arise during nonuniform settlement of foundations (L = distance between column centers)	$0.005L$	$0.005L$
3.	Relative deflection of plain brick walls:		
	(a) for multi-story dwellings and civil buildings at $L/H \leqq 3$	0.0003	0.0004
	at $L/H \geqq 5$	0.0005	0.0007
	(L = length of deflected part of wall; H = height of wall from foundation footing)		
	(b) for one-story mills	0.0010	0.0010
4.	Pitch of solid or ring-shaped foundations of high rigid structures (smoke stacks, water towers, silos, etc.) at the most unfavorable combination of loads	0.004	0.004

For structures supported on fine-grained soils, settlement takes place only under long time loading. Transient loading, if applied in very short duration or durations, brings little additional settlement. In such cases, the settlement should be calculated only under those loads which remain on the structure for a period, or a number of periods, long enough to cause consolidation settlement of the soil. This load may be referred to as the *service load.* On

* From Polshin and Tokar.

Table 8: Values of Safe Bearing Capacity According to Indian Standard is 1904 - 1961

Coarse sand, compact & dry	45 t/m^2
Medium Sand, compact & dry	25
Fine sand & silt	15
Loose* Coarse and medium sand	25
Loose fine and dry sand	10
Hard or stiff clay	45
Medium clay (indented with thumb nail)	25
Moist clay (strong thumb pressure)	15
Soft clay (moderate thumb pressure)	10
Very soft clay (penetrated several inches by thumb)	5

* If a pointed picket 5 x 5 cm pushed down by a man, penetrates more than 20 cm, the sand is said to be loose.

Table - 9.

Relatively Properties of sand.

State of compaction of sand	Dr	N	Qc	Qd
	<0.2	<4	<20	<30.0
Very loose	0.2 - 0.4			$30^o - 35^o$
Loose	0.4 - 0.6	4 - 10	20 - 40	$35^o - 40^o$
Semicompact	0.6 - 0.8	10 - 30	40 - 120	
Compact	70.8	30 - 50	120 - 200	$40^o - 40^o$
Very Compact		> 50	> 200	$> 45^o$

Dr : Relative Density
N : Number of blows in standard penetration test.
Qc : Unit pressure in penetration cone test in Kg/cm^2 after G,G. Meyerhof (1956).

TABLE 10—COMPARISON OF OBSERVED AND ESTIMATED SETTLEMENTS OF SHALLOW FOUNDATIONS ON SAND AND GRAVEL

Structure	Reference	Foundation width, in feet	Soil type	Average standard penetration resistance in number of blows per foot	Bearing pressure in tons per square foot	Maximum Settlement		Ratio estimated to observed settlement
						Observed, in inches	Estimated, in inches	
T. Edison, Sao Paulo	6	60	Fine clayey sand	15	2.4	0.6	1.9	3.2
Banco do Brasil, Sao Paulo	6,8	75	Fine clayey sand	18	2.5	1.1	1.7	1.5
Iparanga, Sao Paulo	7	30	Fine-medium clayey sand	9	2.3	1.4	3.1	2.2
C.B.I. Esplanada, Sao Paulo	8	48	Fine-medium clayey sand	22	4.0	1.1	2.2	2.0
Riscala, Sao Paulo	7	13	Fine-medium clayey sand	c.20	2.4	0.5	c.1.2	c.2.4
Thyssen, Dusseldorf	12	74	Sandy gravel	25	2.5	0.95	1.2	1.3
Ministry, Dusseldorf	12	52	Coarse gravel	20	2.3	0.85	1.4	1.6
Chimney, Cologne	12	67	Sandy gravel	10	1.8	0.4	2.1	5.3

Static cone penetration (deep-sounding) tests have been used to estimate a compressibility index of sand[13]

TABLE 11—COHESIVE SOIL CASES

Case No.	Structure	Soil Profile Beneath Foundation	Foundation Width, in feet	Average Soil Bearing Pressure, in pounds per square foot	Predicted Settlement, in inches: One Dimensional Consolidation Unmodified	Predicted Settlement, in inches: One Dimensional Consolidation Modified For A Factor + Elastic Compression	Observed Settlement, in inches
1	11-Story Hospital Chicago, Ill.	15 ft Of Soft Clay Av. C = 500 psf Av. Wc = 27% Over Dense Clayey Silt	62	2570	5.0	5.6	4.9
2	Gypsum Rock Storage Pile, Tampico, Mexico	35 ft of Stratified Slightly Organic Clay and Silt WC = 20%-50% Av. C = 400 psf	240	1080	24		24
3	Sand Bin, Savanah, Ga.	14 ft of Organic Clay Av. C = 250 psf Wc = 60%-100% (Top Part Desiccated) Over Med. Dense Sand and Silt	5	800 1000	1.1 1.7	0.6 1.1	0.7 2.4
4	High Rise Apartment Building Near Loop Chicago, Ill.	Preconsolidated Very Dense Clayey Silt Wc = 10%-13%	11	18,000	1.3	0.99	0.6
5	Molasses Tank Near North Side Chicago, Ill.	18 ft of Dense Sand Over 27 ft of Soft Clay Wc = 25% C - 500 psf	42.5	4300	2.9[a]		1.5

[a] Settlement Theoretically 40% Complete

TABLE 12—COHESIONLESS SOIL CASES

Case No.	Structure	Soil Profile Beneath Foundation	Foundation Width, in feet	Average soil Bearing Pressure, in pounds per square foot	Predicted Settlement, in inches: Hough[29] empirical method based on N value	Predicted Settlement, in inches: Terzaghi[30] plate bearing test extrapolation correlated to N values	Observed Settlement in inches
6	Chemical Storage Tank Muscatine, Iowa	38 ft of Medium Dense Sand (N = 18) Over Very Dense Sand	120	3,600	3.7 center 2.4 edge	1.3 center 1.6 edge	0.7 0.5-0.9
7	Dextrose Tanks Kansas City	20 ft of Medium Dense Fine Sand Sand and Silt (N = 14) Over Dense Sand	40	2,800	1.5 edge	0.74 edge	0-1.2 (Av = 0.8)

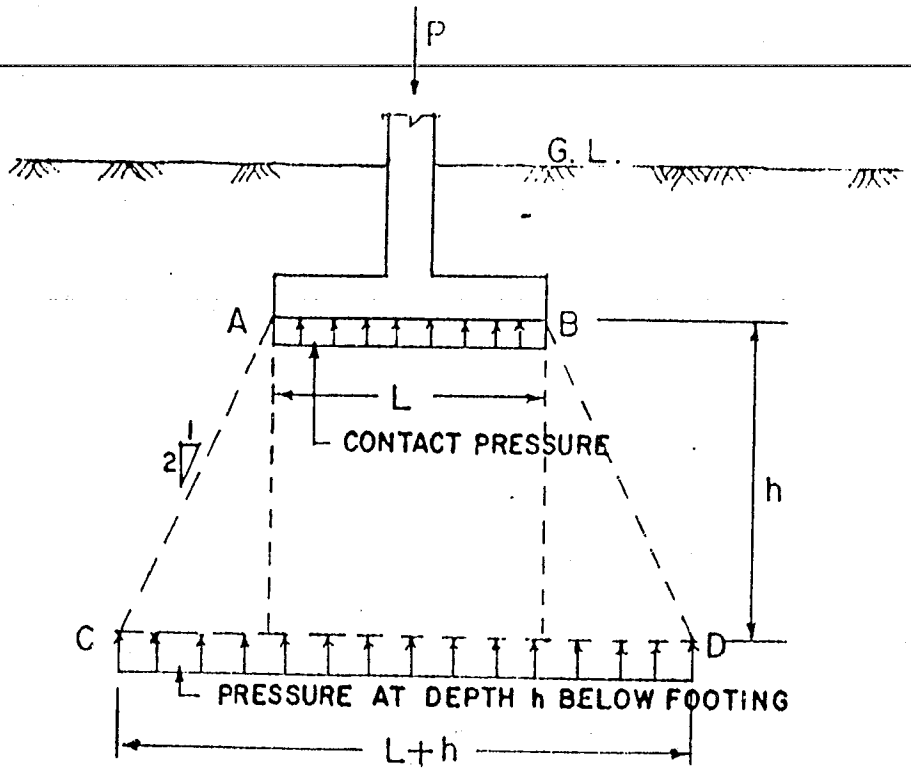


Fig. 1: Approximate Distribution of Pressure within Soil Mass under Footing

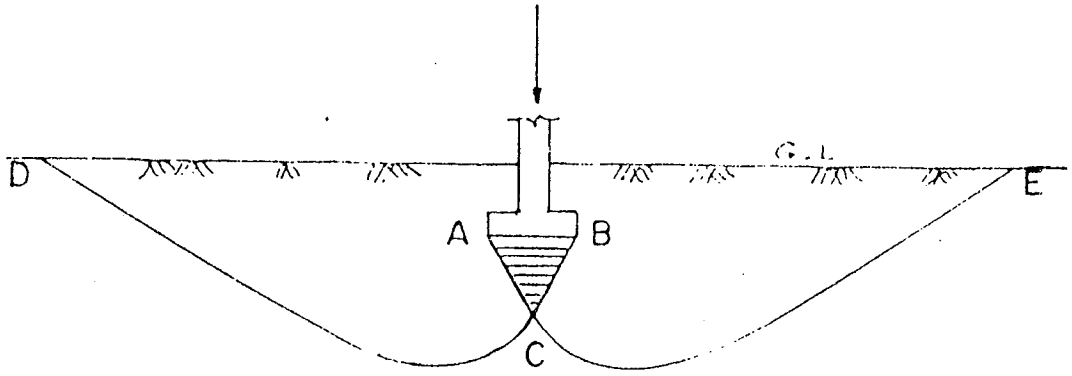


Fig. 2: Shear Failure of Soil under a Footing

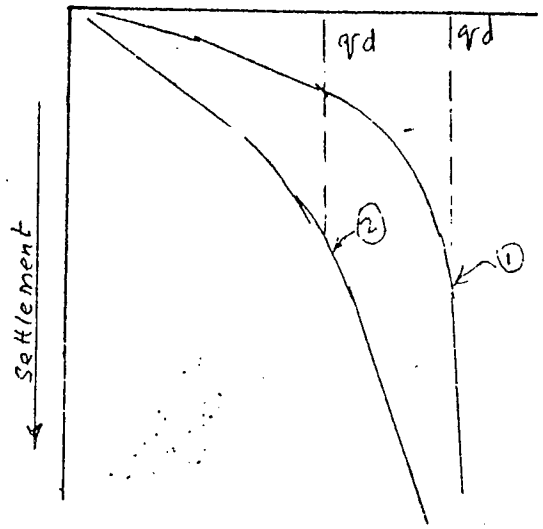


Fig. 3: Load per Unit Area

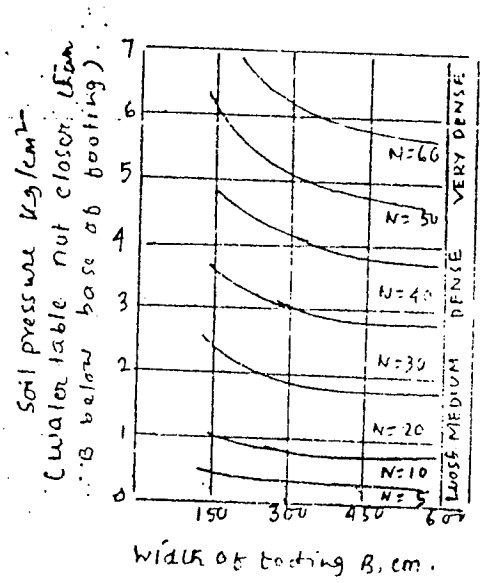


Fig. 4: Soil Pressure Corresponding to 2.5 cm Settlement of Footings on Sand

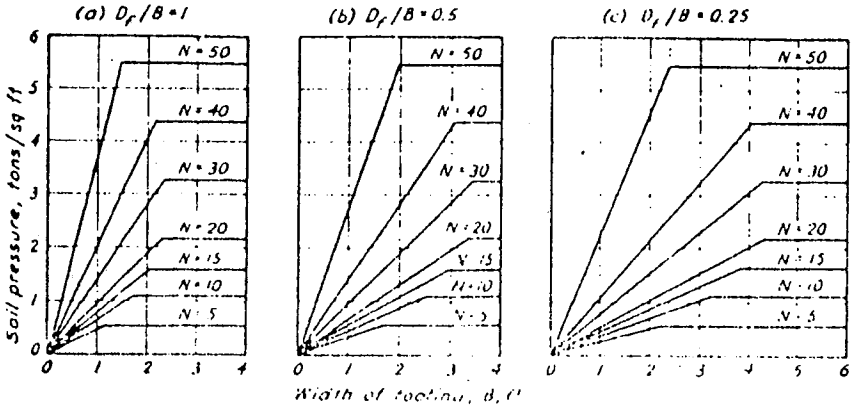


Fig. 5: Allowable Bearing Pressure of Footing on Sand from Settlement Consideration

The above fig. shall be used under the following conditions :-

- The ground water level is in depth equal to the least dimension of the footing below the foundation level.
- The foundation is uniformly loaded
- Maximum settlement under the indicated load will be 1 inch and differential settlement will be no greater than ¾ inch.

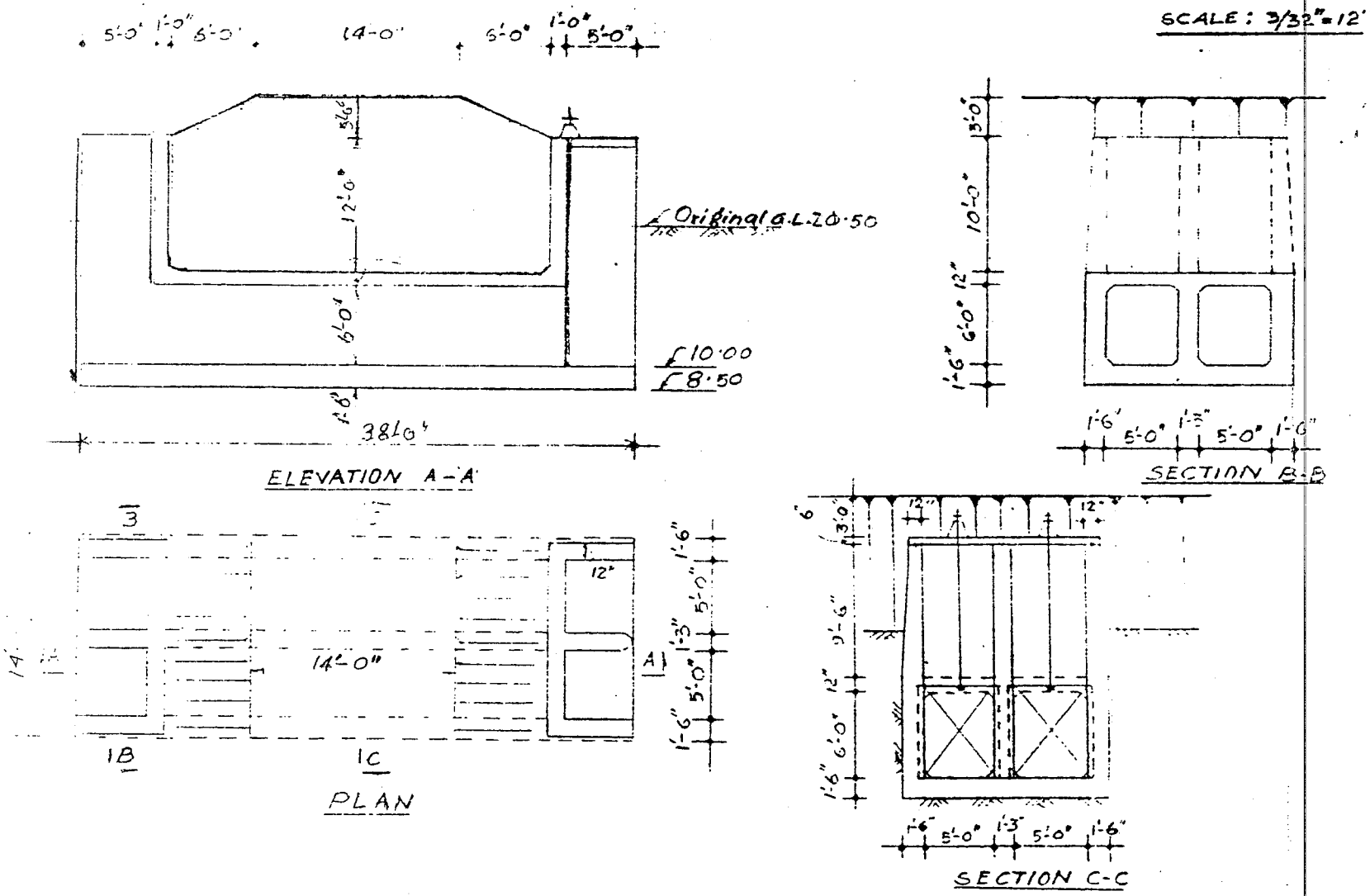


Fig. 6: Main Part of a Typical 2 Vents (5' - 0" x 6' - 0") Sluice

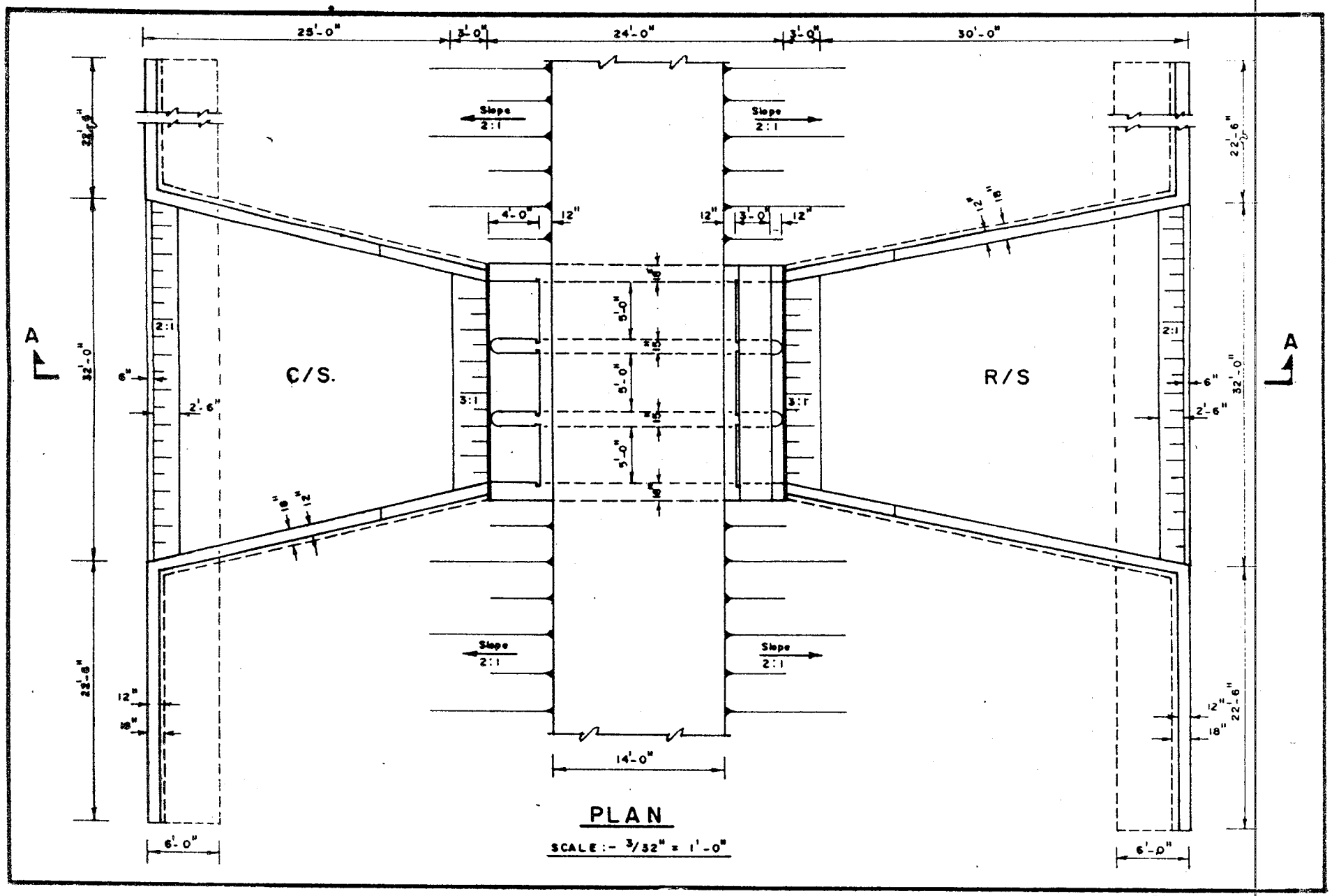
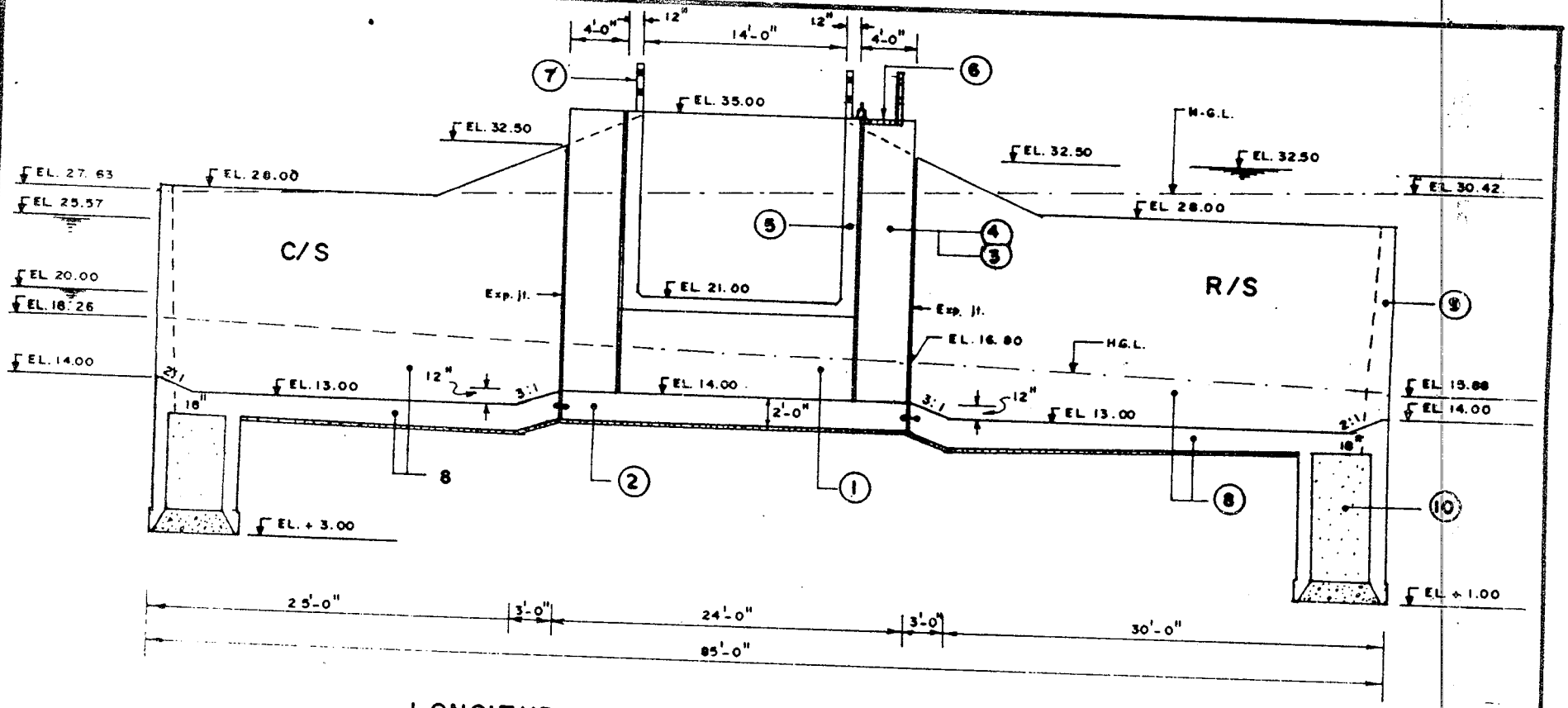

25'-0"
3'-0"
24'-0"
3'-0"
30'-0"
Slope 2:1
Slope 2:1
4'-0"
12"
12"
3'-0"
12"
C/S.
R/S
A
A
2:1
3:1
3:1
2:1
2'-6"
2'-6"
6"
6"
32'-0"
32'-0"
22'-6"
22'-6"
5'-0"
5'-0"
5'-0"
18"
18"
18"
12"
18"
12"
18"
12"
18"
12"
18"
14'-0"
6'-0"
6'-0"
PLAN
SCALE :- 3/32" = 1'-0"

Fig. 7



LONGITUDINAL PROFILE THROUGH A-A

SCALE :- 3/32" = 1'-0"

LEGEND :-

1. Box Conduit
2. Extended Part of Conduit Base Slab
3. Extended Part of Conduit Abutment
4. Extended Part of Conduit Pier
5. Head wall
6. Operating Platform
7. Railing
8. Wing wall & Apron (Manolithic)
9. Return wall
10. Caisson cut off wall

NOTE :-

Protection work not shown

Fig. 8

PROJECT :-
LOCATION :-
DRILLED BY :-

HOLE NO.
GROUND LEVEL
GROUND WATER LEVEL

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG	DESCRIPTION OF MATERIALS	GENERAL: PERMEABILITY	DENSITY	COLOUR	MOISTURE	PLASTIC	BLOWS ON SPOON PER 6 INCH PENETRATION 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	REMARKS
D1						LOOSE	GREY	MOIST	NON PLASTIC	4	4	5	6		
		10													
D2				MEDIUM TO FINE SAND		MED DENSE				4	4	6	7		
D3										5	6	6	8		
		20		trace Silt											
D4										6	7	7	8		
D5				trace mica						6	7	8	8		
		30													
D6										6	8	8	9		
D7						STIFF	LIGHT BROWN		MED. PLASTIC	5	7	8	10		
		40		CLAY trace											
D8										6	7	8	9		
D9				fine Sand						5	6	7	8		
		50													
D10										6	6	7	8		
D11										6	7	8	9		
		60		MED. TO FINE little Silt trace mica		MED. DENSE	Greyish Brown		NON PLASTIC						
D12										7	0	11	13		
D13				FINE SAND little Silt trace mica						8	10	11	12		
		70													
D14										9	11	18	15		
D15										10	12	14	16		
		80													
D16				CLAY trace fine Sand		V. ST			MED. PLAN	11	13	16	16		

INDEX
DISTURBED SAMPLES
UNDISTURBED SAMPLES

FIG. - 9

PROJECT :-
LOCATION :-
DRILLED BY :-

HOLE NO.
GROUND LEVEL
GROUND WATER LEVEL

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG	DESCRIPTION OF MATERIALS	GENERAL: PERMEABILITY	GENERAL: DENCITY	GENERAL: COLOUR	GENERAL: MOISTURE	GENERAL: PLASTICITY	BLOWS ON SPOON PER 6 INCH PENETRATION (0/6, 6/12, 12/18, 18/24)	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	REMARKS
D_1				CLAY, TRACE FINE SAND		V. SOFT	GREY	MOIST	MED. PL.	0 0 1 1		
D_2		10								0 0 1 1		
D_3				CLAY, TRACE, FINE SAND		LOOSE			H.P. NON PLASTIC	1 1 2 3		
D_4		20		FINE SAND SOME SILT		MED				2 3 3 5		
D_5												
D_6		30		CLAY, TRACE FINE SAND TRACE ORGANIC		M. STIFF				Shelby		
D_7				CLAY AND FINE SAND					MP	2 3 5 6		
D_8		40		FINE SAND TRACE		MED. DENS			N. PLASTIC	4 6 8 10		
D_9										5 7 9 11		
D_{10}		50		FINE SAND SOME SILT						5 7 11 16		
D_{11}				CLAY LITTLE FINE SAND		SOFT			H.P.	2 2 2 2		
D_{12}		60		CLAY TRACE FINE SAND & ORGANIC		M. ST			H.P.	Shelby		
D_{13}				CLAY LITTLE FINE SAND,		V. STIFF			M. PL.	3 7 11 13		
D_{14}		70								3 8 10 13		
		80										

INDEX
DISTURBED SAMPLES
UNDISTURBED SAMPLES

FIG. - 10

PROJECT :-
LOCATION :-
DRILLED BY :-

HOLE NO.
GROUND LEVEL
GROUND WATER LEVEL

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG	DESCRIPTION OF METERIALS	GENERAL: PERMEABILITY	DENSITY	COLOUR	MOISTURE	PLASTICITY	BLOWS ON SPOON PER 6 INCH PENETRATION 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	REMARKS
D1				SILT, TRACE FINE SAND.		SOFT	LT. BR.	M	MED. COM.	1	2	2	3		
D2			10'			V. L.			N. P.	0	1	1	2		
D3				SILT, TRACE FINE SAND		V. SOFT	G	O	MED. COM.	0	0	1	2		
D4			20'							0	0	1	2		
D5						V. LOOSE			NON PLASTIC	0	1	2	2		
U1			30'	— DO —			R			Shel by					
D7										1	2	2	3		
D			40'	FINE SAND		MEDIUM DENSE				4	5	7	10		
D8										4	6	8	10		
D9			50'	LITTLE SILT, TRACE MICA			E			7	9	11	12		
D10										8	11	14	16		
D11			60'							8	12	16	20		
D12							Y			9	12	13	20		
D13			70'	— DO —						11	13	15	21		
D14				FINE SAND TRACE SILT & MICA						11	12	15	20		
D15			80'							12	14	16	22		

INDEX
DISTURBED SAMPLES
UNDISTURBED SAMPLES

Figure II

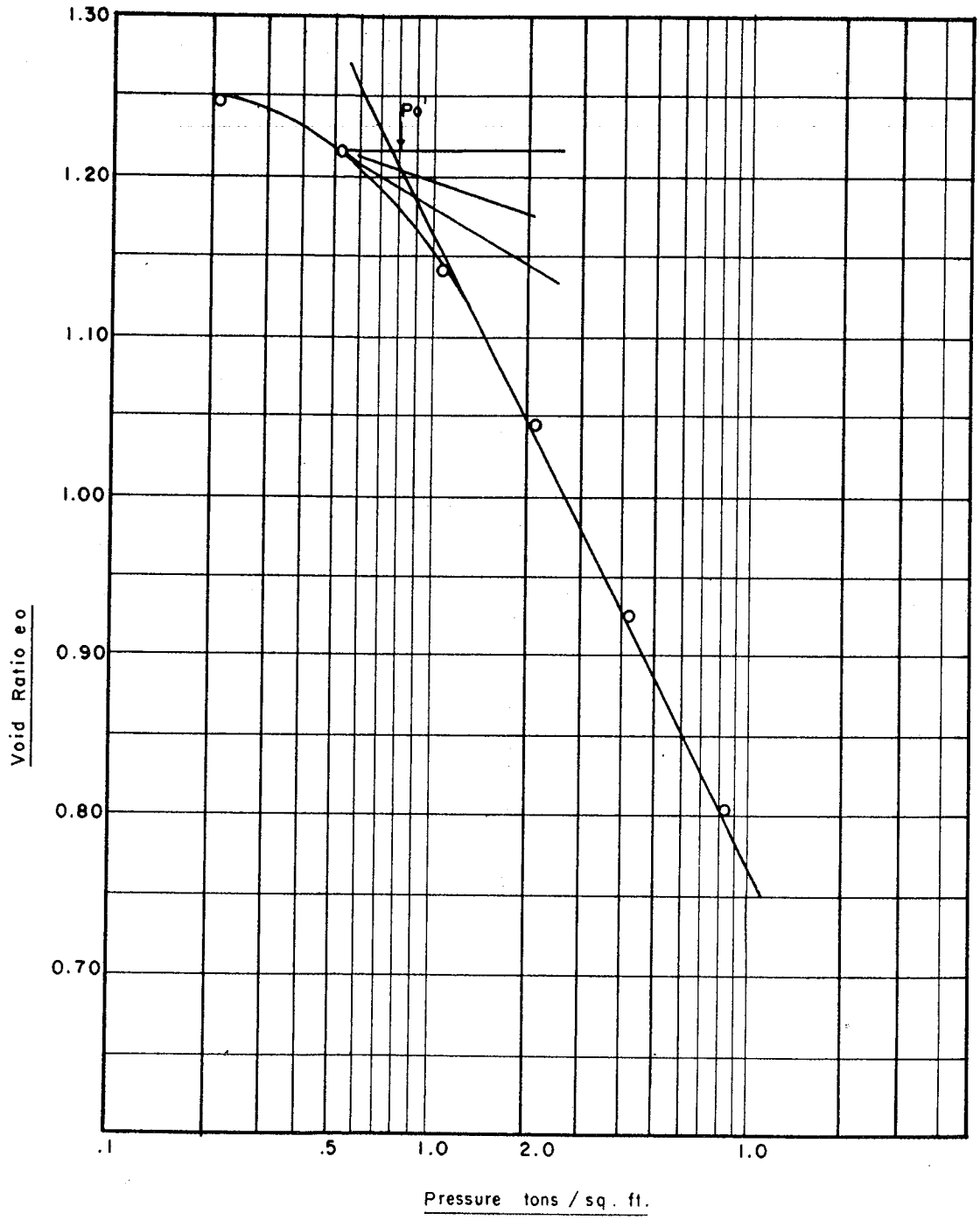
PROJECT :-
LOCATION :-
DRILLED BY :-

HOLE NO.
GROUND LEVEL
GROUND WATER LEVEL

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG	DESCRIPTION OF MATERIALS	GENERAL: PERMEABILITY	GENERAL: DENSITY	GENERAL: COLOUR	GENERAL: MOISTURE	GENERAL: PLASTIC	BLOWS ON SPOON PER 6 INCH PENETRATION 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	REMARKS
D1				SILT, Trace fine Sand		T F O S	Y E R G	M O I S T	MED. COMPRESS	1	2	2	2		
D2		10													
D3										0	0	2	2		
		20								0	1	1	2		
U1				CLAY, Trace fine Sand,					M. PL.	Shelby					
D4				SILT, little fine Sand,											
		30								0	1	2	2		
D5				SILT, trace fine Sand,					MED. COMPRESS						
D6						MED. STIFF				1	1	2	3		
		40								1	2	3	3		
D7										1	2	3	4		
D8															
		50								2	2	3	3		
D9										2	2	4	6		
D10															
		60								3	3	4	7		
D11				SILT, little fine Sand,		STIFF		T		3	4	5	7		
D12															
		70								4	5	6	6		
D13				CLAY, Trace fine Sand					MED. PLASTIC	4	5	6	6		
D14															
		80								5	6	8	9		
D15										6	7	7	10		

INDEX
DISTURBED SAMPLES
UNDISTURBED SAMPLES

FIG.-12

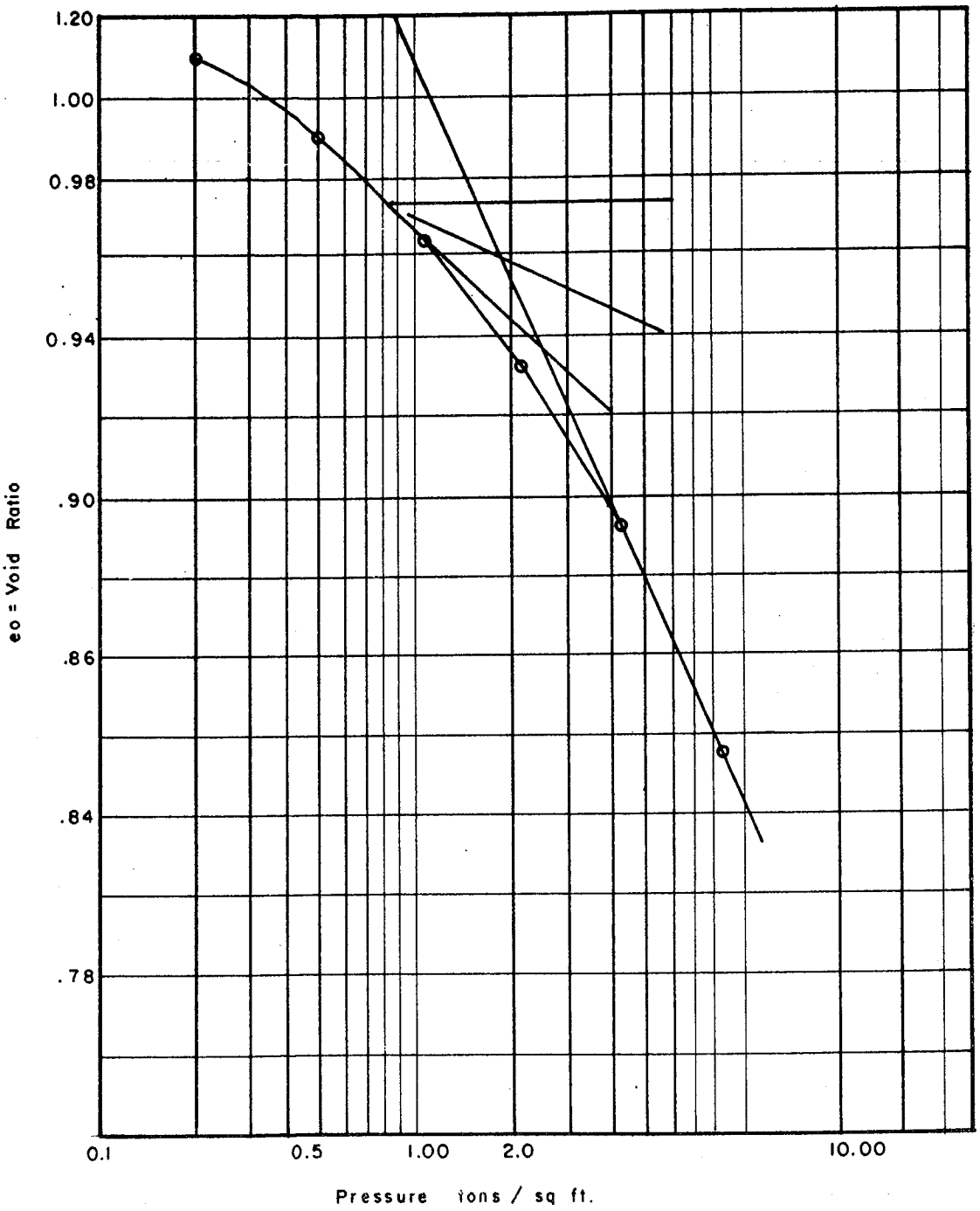


Po = 0.80 ton / sft

Cc = 0.32

Co = 1.28

FIG. 13



Po = 1.80 ton. / sft

Cc = 0.188

eo = 1.05

FIG. 14

SOILTECH
INTERNATIONAL LIMITED
D-H-A-K-A

Client:- W D B O & M DIV. KAPTAI

Site:- KHANDAKIA (RIVER BED) HATHAZARI UPA-ZILLA, CHITTAGONG

Bore chart of Boring No. 1

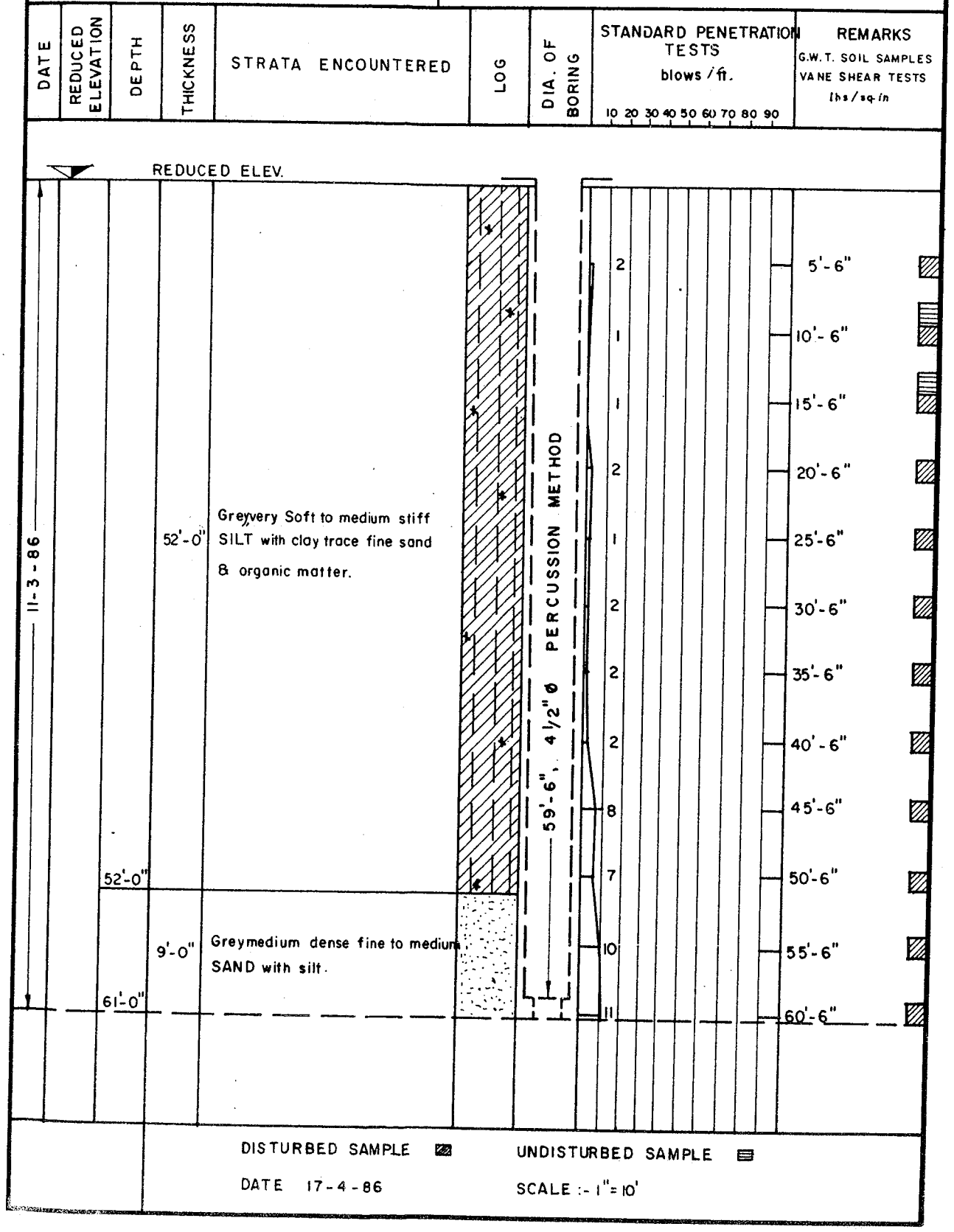


Fig. - 15

BANGLADESH WATER DEVELOPMENT BOARD
GROUND WATER DIVISION – I
GROUND WATER CIRCLE
FIELD BORE LOG

PROJECT : Bridge Over Madari Khal
LOCATION : Madari Bridge
CLIENT : XEN, Kaptai O&M Div. WDB. Kaptai
DATE STARTED : 24.12.85
DATE COMPLETED : 24.12.85
DRILLED BY : GWD-I, UNIT— III

SITE : Madari
HOLE NO. 3-GROUND LEVEL N/S
GROUND WATER LEVEL : River Boring AT ..... HRS. OF DATE .....
TOTAL DEPTH : 82'

NO. OF SAMPLE	TYPE OF SAMP.	SCALE	DESCRIPTION OF METERIALS	GENERAL: PERMEABILITY	GENERAL: DENSITY	GENERAL: COLOUR	GENERAL: MOISTURE	GENERAL: DILATANCY	DISTURBED SAMPLES: BLOWS ON SPOONS 0/0	6/12	12/18	16/24	BLOWS ON CASING 0/12	12/24	PENETRATION TEST NO. OF BLOWS PER 1ST PENT.	REMARKS
D1			CLAY LITTLE SILT		V. SOFT	GREY	MOIST	PLASTIC	0	0	1	1				
			8'													
D2			FINE SAND		LOOSE			NON PLASTIC	2	3	4	5				
D3									2	3	4	5				
D4									2	4	4	6				
D5									3	4	4	7				Bottom of caisson
D6			TRACES OF SILT		MOENSE				3	4	5	7				
D7									6	8	10	17				
D8									7	9	12	16				
			43'													
D9			CLAY LITTLE SILT		V. SOFT			PLASTIC	1	1	1	2				
D10									1	1	1	2				
D11					SOFT				1	2	2	3				
D12									2	2	2	4				
D13									2	3	3	5				
D14			CLAY LITTLE SILT		MSTIFF				2	3	3	6				
D15									3	3	3	8				
D16									3	3	4	10				
			82'													

INDEX: DISTURBD SAMPLES; UNDISTURBED SAMPLES

Figure 16

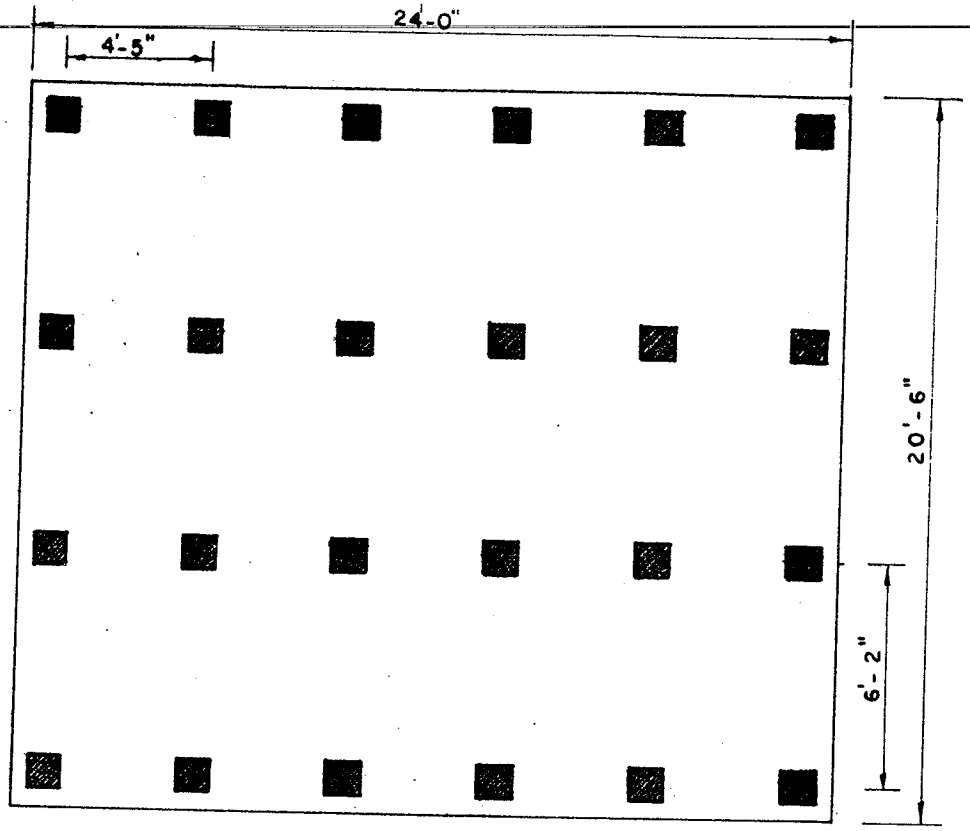
RIVER RESEARCH INSTITUTE, BWDB. BORING LOG

PROJECT : Bridge over Changkhali Khal	HOLE NO. : H5
LOCATION : Changkhali	GROUND LEVEL : N/S
DRILLED BY : G W D – I	GROUND WATER LEVEL : River Boring

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG	DESCRIPTION OF METARIALS	GENERAL: PERMEABILITY	DENSITY	COLOUR	MOISTURE	PLASTICITY	BLOWS ON SPOON PER 6 INCH PENETRATION 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	REMARKS
				SILT, TRACE FINE SAND		V. SOFT	GREY	MOIST	M. COMPRESS	0	0	0	1		
D1															
D2			10'							0	0	0	1		
				CLAY, LITTLE FINE SAND					M. PL.						
U1										Shelby tube 3"x24" Recovery is 19"					
			20'	CLAY & FIND SAND		SOFT			L. PLASTIC	1	1	2	3		
D3															
D4										1	1	3	4		
			30'			MST.									
D5										2	3	3	6		
				SILT & FINE SAND		LOOSE			N. P.						
D6										3	3	4	7		
			40'	SILT, LITTLE FINE SAND		MST.			M. C.						
D7										3	3	5	8		Bottom level of foundation.
				CLAY, LITTLE FINE SAND		SOFT			M. PLASTIC						
D8										1	1	1	2		
			50'												
D9										1	1	1	2		
D10										1	1	1	3		
			60'	FINE SAND, TRACE SILT TRACE MICA		M. D.			M. P.						
D11										5	6	6	8		
				CLAY, LITTLE FINE SAND		SOFT			HED PLASTIC						
D12										2	2	2	3		
			70'												
D13										1	2	2	3		
D14										2	2	2	3		
			80'												
D15										2	2	2	5		

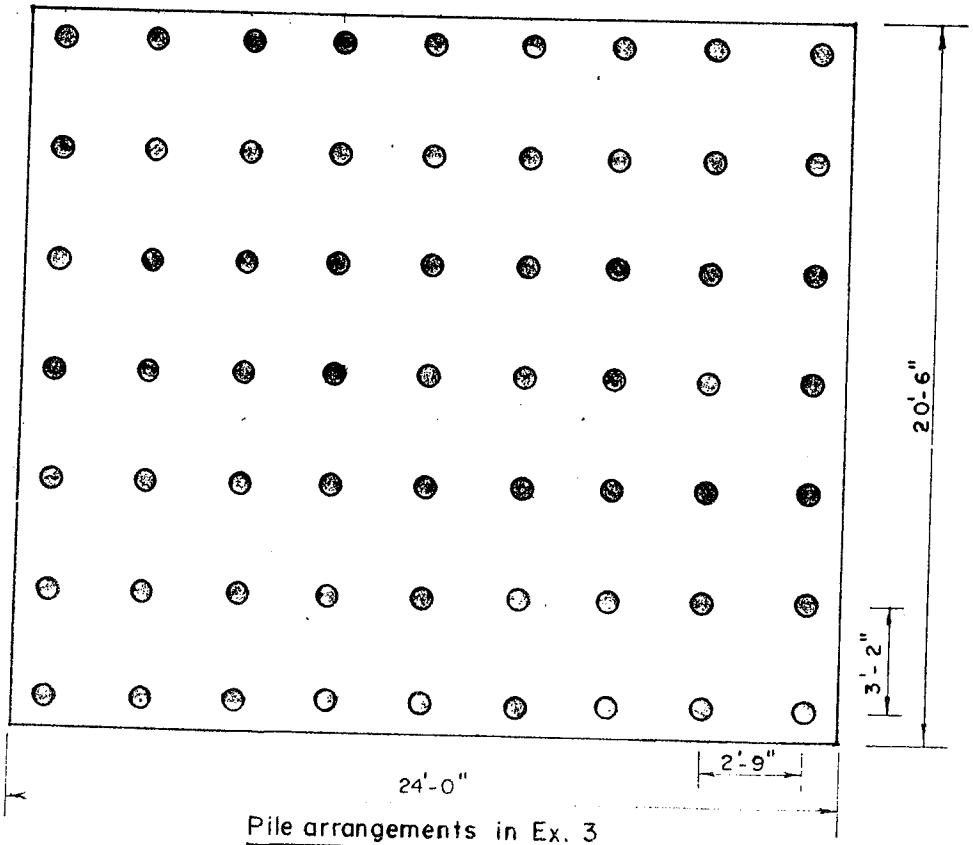
INDEX: DISTURBED SAMPLES; UNDISTURBED SAMPLES

Figure 17



Pile arrangements in Ex. 5

Scale:- 3/16" = 1'-0"



Pile arrangements in Ex. 3

Scale:- 3/16" = 1'-0"

FIG. -18