WORKSHOP ON DESIGN OF SMALL SCALE WATER CONTROL STRUCTURES

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data are, in most cases found to be not effective in achieving the desired objectives.

Data needed for design of the different components of the large scale projects are usually available. Most of the required data are available from the detailed feasibility report and the established sources. But for design of the different components under the small scale project, the design engineers are to depend, in almost all cases, exclusively on the information and data supplied by the field engineers. Of the data needed for design of hydraulic structures, the hydrological data is the most important. But no long-term or short-term hydrological data for the areas under most of the small scale projects are available from the established sources. In cases where time permits, hydrological data for a period of one year should be collected by establishing temporary gauging station/stations within the project area. Such hydrological data may form a basis for hydrologic and hydraulic design of structures. In cases when the small scale projects are required to be implemented within a short time and no time is available for collection of the hydrological data even for a period of one year, the required design data may be collected from the local enquiry, by studying the existing structures like bridges, culverts, etc. on the existing channels and from the channel geometry.

It is necessary for the field engineers to be aware of the basic procedures adopted and the data required for the different steps of design, which will enable them to assess the requirements of

data for design of structures. To help promote the awareness among the field engineers as well as the design engineers, this paper outlines the basic design procedures and design data requirement for sluices, regulator, water retention structures, bridges, syphones and embankments.

This paper also provides comprehensive data checklists for the above mentioned structures. The engineers should use the said checklist as a guide only. Data requirements may change according to the site condition of the individual structure. The field engineers must thoroughly be familiar with the over all project plan and objectives and the local site conditions. They can apply their judgement in collecting and processing the design data and furnish valuable comments on them if they are thoroughly familiar with the project plan and objectives and the local site condition. The design engineers should also be familiar with the site conditions. The structures designed by the engineer not familiar with the site condition, are found to be not suitable in many cases.

The field data received in the Design offices are found to be erroneous and unreliable in many cases. And the main reason for this is that the data collected by the lower level field staff are seldom checked by the responsible field engineers. Some guidelines and points for verification of the design data before submission to the design office, are furnished in the final section of this paper.

1.1 DATA REQUIREMENTS FOR THE DESIGN OF HYDRAULIC STRUCTURES :

Data required for the design of almost all hydraulic structures are almost the same with the exception of a few.

Normally the data & maps listed below are applicable for the design of sluices, regulators, water retaining structures, syphons etc.

1. Topographic map
2. Rainfall data
3. Land use map
4. Cropping pattern
5. Long & cross sections of the channel
6. Tidal records (for Coastal structure)
7. Stage-discharge records (for upland)
8. Soil classidication
9. (a) Soil boring samples
   (b) Bore logs with SPT values, ground level
10. Ground water level & fluctuation
11. Site plan
12. Long and cross sections of embankments
13. Diversion channel cross-section
14. Live load
15. Long & cross sections of the distributaries

1.2 LABORATORY TEST RESULTS OF SOIL :

For Cohesive Soil :

1. Grain size analysis
2. Liquid limit & plastic limit
3. Specific gravity
4. In situ unit wt.
5. Natural moisture content
6. Unconfined compression strength (qu)
7. Cohesion (c)
8. Angle of internal friction (O)
9. Compression index (C)
10. Initial void ratio (e)
11. Permeability (k)

Note : For above item (1), (2), & (5) the sample shall be disturbed and for (3), (4), (6), (9) & (10) the sample shall be undisturbed.

1.3 FOR NON COHESIVE SOIL :

1. Grain size analysis
2. Specific gravity
3. In-sity unit weight
4. Natural moisture content
5. Angle of internal friction (O)
6. Permeability (k)

2. DESIGN PROCEDURES & DATA REQUIREMENTS :

2.1 SLUICES AND REGULATORS :

A drainage sluice/regulator is the structure that helps drainage of the excess runoff from the catchment bounded by the polder during the pre & post monsoon period. The other function of the structure is to prevent monsoon flood from coming inside the polder when the outside channel/river stages are higher than that of the inside one. The drainage sluice/regulator may also be used as a flushing one by designing its upstream stilling basin with due consideration of the flushing level of the river.

2.1.1 HYDROLOGIC AND HYDRAULIC DESIGN :

The runoff hydrograph is the ultimate result of the hydrologic analysis of a basin & represents the time discharge relationship of the total design storm runoff at the basin outlet. The construction of the runoff hydrograph takes advantage of the Unit Hydrograph. The runoff hydrograph is required for the routing of flow through a regulator and thereby fixing its ventage. To derive the unit hydrograph and finally for the computations of the runoff hydrograph the following maps and data are required :

(i) Topographic map

(ii) Rainfall data

(iii) Land use map

(iv) Cropping pattern

Drainage Channel.

The design of the drainage channel is one of the vital requirements for the design of a regulator. The drainage channel should be so designed that it can accommodate the flow passing through the regulating structure. Manning's formula is used to design the drainage channel and the data required are the energy slope, cross-section and roughness co-efficient.

In case of the channel having almost uniform section, the energy slopw may be assumed to be parallel to the bed slope and as such, the energy slope could be aproximated with the bed slope of the channel and may be found from the long section of the channel bed. The roughness co-efficient may be selected from the nature of the channel. Therefore, for the design of the drainage channel the following data are required.

(i) Slope of the channel

(ii) Cross-section of the channel

(iii) Roughness Co-efficient.

2.1.2 STRUCTURAL DESIGN.

2.1.2a Foundation Design.

The foundation must be judged with respect to two types of behavior :

(i) Bearing capacity consideration

(ii) Settlement consideration.

Bearing capacity is characterised by the structure breaking into the soil because the materials is incapable of supporting the load. On the other hand, settlement is characterised by the

sinking of the structure due to compression & deformation of the supporting layer underneath.

The laboratory tests required for the foundation design are given in the list of data in the previous section of this paper.

The field data in case of granular soil are the Standard Penetration Test (SPT) values, the ground levels & the water table which must be recorded during the explorating boring.

2.1.2b **Superstructure Design.**

The superstructure is the structural portion of a sluice/regulator above the floor of the structure.

The design data required are as follows :

(i) Site Plan

(ii) Embankment Cross-Section

(iii) Diversion Channel Cross-Section

2.2 WATER RETENTION STRUCTURE :

Water retention structure is a gated structure which is constructed across a channel/khal to hold water during post monsoon period for the irrigation purpose during intervening dry season. It also helps in checking ground water depletion by retaining water in the channel at the desired level by putting gates.

2.2.1 DESIGN PROCEDURE & DATA REQUIREMENTS :

2.2.1a Hydrologic & Hydraulic Design.

The design of the Water Retaining Structure calls for the maximum design depth in the channel which can be found from the direct measurement or from the maximum design flow by using Manning's formula. For a small area of about 200 acres, the rational formula may be used otherwise, empirical formula or Unit Hydrograph are the basic tools for the estimation of the design flow. This may also be directly measured by measuring average velocity & area of flow of a channel.

The design data required are as follows :

(i) Observed maximum flood level & minimum level during dry season.

(ii) Discharge (maximum & minimum)

(iii) Slope of the channel

(iv) Cross-section of the channel

(v) Roughness Coefficient.

2.2.1b Structural Design.

The stuctural design requires foundation analysis and the Superstructure design of a structure. The data required for the design are the same as that of the sluice/regulator and have been dealt earlier in this paper.

2.3 BRIDGES :

The bridge is one of the most common structures that cover a gap and generally carry a road or railway across a river, canal etc.

2.3.1 DESIGN PROCEDURES AND DATA REQUIREMENTS FOR BRIDGES :

2.3.1a Hydraulic Design.

The maximum design level or the highest high flood level ever recorded shall be used for the fixation of the level of the bridge deck. The maximum design depth may be found from the direct measurements of the observed maximum flood level or from the computed discharge using Manning's formula with the known parameters.

If the highest high flood level can not be ascertained from the record or computation the same could be approximated from the local enquiry.

The maximum design flow may be obtained from direct measurements by measuring the mean velocity and multiplying the same with known cross-section of the flow. For a small catchment the rational formula and for the bigger ones the empirical formula or Unit Hydrograph may be employed for the estimation of maximum flow.

For a given flow the minimum permissible opening of the structure and the scour depth may be found with the help of the following data.

(i) Slope of the channel

(ii) Cross-Section of the channel

(iii) Grain size analysis.

2.3.1b Structural Design.

Foundation Design.

Foundation design is done with regard to following criteria

(i) Bearing Capacity Consideration

(ii) Settlement Consideration

For the analyses on the above considerations the following design data are required :

(i) Soil boring data

(ii) Soil Classification

(iii) Water Level & Fluctuation

(iv) Channel Bed Samples

(v) Observed depth of scour

(vi) Flood Velocity measurement.

Superstructures Design.

For the design of the Superstructure the following data are required :

(i) Site Plan

(ii) Type of road

(iii) Road Embankment Cross-Section

(iv) Live Loads

(v) Vehicular Requirements

(vi) Navigationala requirements.

2.4 SYPHON :

In an Irrigation project whenever a canal crosses a natural drainage on its passage it is necessary to construct a structure termed as cross-drainage works. A syphon is one of the cross-drainage works. In an irrigation system, canals are generally

aligned on the watershed so that they meet minimum number of drains. Either the canal or the drain may be passed through a cross-drainage structure depending on the available head in the canal or drain.

1) When an irrigation canal passes over the drainage channel, an aqueduct or a syphon aqueduct may be used.

In practice an idealised condition for the selection of a syphon may not be met with and the choice in fact depends upon the following considerations.

(i) Canal alignment

(ii) Permissible loss of head in the Canal

(iii) Sediment characteristics of the drainage channel

(iv) Foundation soil

(v) Groud water table

(vi) Relative economics.

2.4.1 DESIGN PROCEDURES AND DATA REQUIREMENTS FOR SYPHON AQUEDUCT.

2.4.1a Hydrologic and Hydraulic Design.

For the estimation of design flow of the drainage channel, Irrigation or water Development maps if necessary updated ones are required for the delineation of the watershed boundary.

Rainfall Data.

At least 10 years rainfall data of a station or stations within the project area or of the nearby station may be collected. Normally, these data are available with the Hydrology Directorate of BWDB. Witht the advent of the computers in the country these data are being computerised and are available in the diskettes.

a) Land Use Maps are necessary for the determination of the culturable and non-culturable land within project area.

b) Cropping pattern within the project area is also required by the designer for the estimation of design flood.

2.4.1b **Drainage channel Discharge characteristics.**

Drainage channel slope may be established from the long section of a channel. In the natural channel the bed slope is not the true representative of the energy slope. As such the water surface slope if determined correctly will give better result in respect of computation of velocity through a channel by using Manning's formula. Caution should be taken in having water levels at a number of points upstream and downstream of the structure location including one at the structure site during high flood period on the same day and at the same time in ascertainingthe water surface slope.

To know the channel characteristics the cross-sections of the channel are very important and the long sections could be developed from the cross-sections.

2.4.1c **Roughness Co-efficient.**

Manning's formula is widely used and also gives good result for the determination of average velocity. In the Manning's formula the velocity varies inversely with the roughness of the channel. The roughness coefficient is one of major factors in this formula and as such its precise determination calls for careful attention.

2.4.1d Water levels of the Channel.

At the section of the channel where the syphon aqueduct would be constructed the maximum high water levels of the channnel for a period of 5-10 years are required. All recorded yearly maximum water levels must be given. In most of the case water levels have to be interpolated since water levels are rarely recorded at the site. In other cases short term (1 to 2 years) data at site should be correlated to long term data of stations nearest to the site.

2.4.1e **Manning's Formula.**

For the determination of the average velocity across a channel section and also to determine the friction loss through the barnel, the Manning's formula may be used. The data required are the energy slope, cross-section of the channel and roughness co-efficient.

2.4.2 STURCTURAL DESIGN.

2.4.2a Foundation Design.

Foundation Design requires the determination of the bearing capacity and this may be checked from the settlement of the structure. Structures founded on the granular soil undergoes primary settlement only and such in this type of soil having isotropic & homogeneous layer settlement criterion is not that important but in case of the cohesive soil both the primary and secondary settlement are very important and often settlement consideration governs the foundation design.

The data required for the foundation analyses are bore logs showing SPT values, gound levels and ground water table and a sample should be collected during exploratory borings for laboratory tests as mentioned in article 1.2

2.4.2b **Superstructure Design.**

For the design of the Superstructure the top level of the structure would be fixed first, the design data required are :

(i) Site Plan

(ii) Diversion channel cross-section.

(iii) Live load on the structure.

2.5 EMBANKMENT :

2.5.1 TYPES OF EMBANKMENT.

Embankment may be classified into 3 (three) major heads depending on the nature of protection required and their locations as well.

(i) Full flood protection embankment.

(ii) Submersible Embankment.

(iii) Tidal Embankment.

2.5.1a **Full flood protection embankment.**

Full flood protection embankment is that which provides full flood protection i.e, at all stages of the river the flood water will not overflow the embankment.

2.5.1b Submersible Embankment.

Submersible embankments are usually built to protect the pre-monsoon flooding but it gets submerged in water during monsoon. It is constructed of low height and requires annual repair and

maintenance because it undergoes heavy damages during overtopping by flood water.

2.5.1c **Tidal Embankment.**

Tidal Embankments are usually constructed along the coastal belt to protect the tidal surge and intrusion of of saline water into the upland. As such, special design considerations are required in respect of its safety against stability, overtopping, erosion etc.

2.5.2 DESIGN PROCEDURES AND DATA REQUIREMENTS FOR EMBANKMENT :

2.5.2a **Alignment of the embankment.**

The alignment of an embankment is governed mainly by technical, economical & morphological considerations. Economically the best alignment is that which can be built as efficient and cheap as possible, requires the least land aquisition and is constructed with the best available materials.

Primarily, for fixing the alignment the topographic map is required in order to avoid habitations depending on the set back distance.

Study of the morphological behavior of the nearly river/channel in respect of its bank movement is also required.

2.5.2b **Setback Distance.**

Set-back distance should carefully be fixed considering scouring of the river bank and tendency of the movement of the channel towards the planned embankment.

2.5.2c Soil Survey.

After fixing the tentative alignment of the embankment a soil survey is to be made to inventorise the available soil material. Preference should be given to soils with a fair portion of clay but the subsoil should not consist of peaty soil. For soil survey quick auger boring may be done to a depth of about 10'-0" at an interval of about 1000'-0" along the centre line of the embankment.

2.5.2d **Water levels and discharges of the river.**

For the design of the river embankment the water levels should be presented as daily maximum for the months from May to October for a period of 5-10 years. Discharges may be given in monthly maximum. The frequency of ocurrence of flood may be taken as 20 years where only the agricultural damage is predominant and 100 years where the loss of human life is involved.

2.5.2e Long and Cross-Sections along the alignment.

Once the alignment is fixed the long and Cross-Sections are to be taken. The cross-sections should, preferably, be taken at an interval of 500'-0" if the ground profile is almost level and in other cases of depressions or high land at least one section at these points must be taken.

2.5.2f **Cross-Sections of the river.**

To compute the Water surface profile of the river, Cross-Sections, and Water level at control section are required and should be at an interval of 1000 ft.

2.5.2g Free Board.

The determination of free board of an embankment is governed by the embankment size, location, water level fluctuation in the river, wind action, service road requirement, soil characteristics etc.

The data required are wind velocity, fetch, average water depth.

2.5.2h Stability and Settlement Analysis.

For the analysis of stability and settlement of the embankment, subsoil investigation is required. Borelogs along the centreline of the embankment upto a depth of 30'-40' depending upon the width of the embankment showing the SPT values and in case of cohesive soil, samples should be collected for laboratory tests.

The laboratory test data include the following :

(i) Unit ult of soil :
Dry, compacted, moist and saturated.

(ii) Grain size analysis.

(iii) Void ratio for grantar soil (natural, loosest and densest state)

(iv) Liquid limit test

(v) Shear test, Triaxial test, Compressibility test, Unconfined Compression test, Permeability test.

DESIGN DATA CKECKLIST

Design data are normally collected from the following sources :

1. Published maps, photographs and reports.

2. Published hydrological records.

   - rainfall
   - stage
   - discharge
   - tidal fluctuations.

3. Engineering field reconnaissance, and surveys.

   - cross-section and profile surveys
   - hydrologic and hydraulic measurements.

4. Soil Investigations.

5. Agricultural, Social, and Environmental Information from Government Department and other organizations and local farmers.

6. Hydrologic observations by local farmers.

   - frequency, extent, duration and time of flood
   - maximum flood stage, minimum discharge

The actual design data requirements can be divided into five categories.

1. Geographic Data

   - topographic conditions.
   - land use
   - cropping pattern

2. Hydrologic Data

   - rainfall information
   - channel/riverstage
   - discharge patterns
   - tide records

3. Hydraulic Data

   - channel hydraulic properties.
   - velocity measurements.

4. Geogtechnical Data

- soil classification
- soil boring samples
- water table level and fluctuation
- channel bed samples

5. Structural Data

- site conditions
- live loads
- vehicular requirements
- navigational requirements.

2.1 Data Checklist for Sluices and Regulators.

1. Geographic Data.

1.1 Sub-Project Description
- name and location
- background to sub-project
- function or type
- environmental impact
- problem, solution, and expected result

1.2 Describe Existing Hydraulic Structures
- operation
- maintenance
- scour
- sedimentation

1.3 Drainage Basin (Index) Map Scale : 1" = 1 mile
- channels
- embankments, roads, cross-dams
- water movement
- basin boundaries
- benefitted area
- existing and proposed strutures

1.4 Topographic Map (Scale : 4" = 1 mile;
Contours : 1' to 5' intervals)

1.5 Site Plan (Scale : 1" = 50')
- spot levels at 25' intervals
- minimum area of coverage 500'x500'; may be large depending on the size of the structure.
- proposed structure location
- embankment, diversion channel locations.

1.6 Cross-section of Existing or proposed Embankment and Diversion Channel.

1.7 Land Use Map (Scale : 1" = 1 mile)
- village boundaries
- cultivated land

1.8 Description of Cropping Pattern

1.9 Channel Stability
- extent of scour or sedimentation in main channels

1.10 Aerial Photographs, and Ground Level photos (if available)

2. Hydrologic Data.

2.1 Rainfall Records or name and location of nearest Rainfall Station.

2.2 Tide Records (only required in Coastal Areas)
- hourly, daytime records for at least 1 year, when other records are not available.

2.3 Stage, Discharge Records of Outfall Channel (only required in Upland Areas)
- daily stage records for at least 1 year, when other records are not availbale.

2.4 Minimum Discharge Measurement (Only required if structure used for dry season irrigation)

2.5 Recorded, or Measured Maximum Flood Stage in Outfall Channel and Drainage Channel

3. Hydraulic Data

3.1 Cross-Sections and Profile Measurements in Outfall Channel

- Cross-section measurements to be taken at 500' intervals and extend for a mile upstream and downstream of the outfall.

- longitudinal profile measurements at 500' intervals for at least 1 miles upstream and downstream of the outfall.

- the number and interval of cross-sections and profile measurements depends on the size and uniformity of the of the channel. In cases where the channel is straight and there are no break in topography, existing structures or side channels, the length between cross-sections and profile measurements may be increased to 1000'.

- cross-sections should be extended far enough on both sides of the center-line to show the field elevation and changes in topography that may affect design (embnakment and berm details).

- profile measurements include the bed level, right and left bank level (ground or field level) and water profile. When embankments are present, the right and left top level should also be incuded.

- Water profile should be taken at as high a stage as possible.

- the location of embankments, roads, other channels and existing temporary or permanent structures should be shown on the profile drawing.

3.2 Cross-sections and Profile Measurements in Drainage Channel.

- Cross-section measurements to be taken at 500' intervals and extend from outfall channel to a point 1 mile upstream of proposed site.

- longitudinal profile measurements at 1000' intervals and extend from outfall channel to a point at least 1 mile upstream of proposed site.

4. **Geotechnical Data.**

4.1 Soil Survey, Soil Classification in Basin

4.2 Soil Classification at Site.

4.3 Soil Boring Samples
- to determine settlement, bearing capacity and permeability.

4.4 Water Table Elevation and Fluctuation.

5. **Structural Data**

5.1 Live loads over the structure.

2.2 **Data Checklist for water Retention Structures.**

1.1 Sub-Project Description
- name and location
- background to sub-project
- function or type
- environmental impact
- problem, solution, and expected result

1.2 Describe Existing Hydraulic Structures
- operation
- maintenance
- scour
- sedimentation
- debris

1.3 Drainage Basin (Index) Map Scale : 1" = 1 mile
- distribution canals
- embankments, roads, cross-dams
- water movement
- basin boundaries
- benefitted area
- existing and proposed strutures

1.4 Topographic Map (Scale : 4" = 1 mile;
Contours : 1' to 5' intervals)

1.5 Site Plan (Scale : 1" = 50')
- spot levels at 25' intervals
- minimum area of coverage 500'x500'
- proposed structure location
- embankment, diversion channel locations.

1.6 Cross-section of Existing or proposed Embankment and Diversion Channel.

1.7 Land Use Map (Scale : 1" = 1 mile)
- village boundaries
- culltivated land

1.8 Description of Cropping Pattern

1.9 Channel Stability
- extent of scour or sedimentation in main channels

2. Hydrologic Data.

2.1 Rainfall Records or name and location of nearest Rainfall Station.

2.2 Tide Records (only required in Coastal Areas)
- hourly, daytime records for at least 1 year, when other records are not available.

2.3 Stage, Discharge Records
- daily stage records for at least 1 year, when other records are not availbale.

2.4 Minimum Discharge Measurement

2.5 Recorded, or Measured Maximum Flood Stage

পলি দাশ
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3. Hydraulic Data

3.1 Cross-Sections and Profile Measurements

- as a minimum requirement, cross-section measurements to be taken at 2 cross-section intervals and extend for 2 mile upstream and downstream of the proposed site.

- longitudinal profile measurements at 1000' intervals to be taken for at least 1 mile upstream and downstream of the proposed ate.

- for comprehensive sub-project planning, cross-sections and profile measurements should be taken from the outfall channel to a pointupstream of the proposed structure where the structure no longer influences channel flow (channel bed level equals retention level). Cross-section interval may be expanded to every 1000'.

- cross-sections should be extended far enough on both sides of the center-line to show the field elevation ahd changes in topography that may affect design (embankment and berm details).

- profile measurements include the bed level, right and left bank level (ground or field level) and water profile. When embankments are present, the right and left top level should also be included.

- Water profile should be taken at as large a stage as possible.

- the location of embankments, roads, other channels and existing temporary or permanent structures should be shown on the profile drawing.

3.2 Velocity Measurement.

- to be taken during flood.

4. Geotechnical Data.

4.1 Soil Survey, Soil Classification in Basin

4.2 Soil Classification at Site.

4.3 Soil Boring Samples

- to determine settlement, bearing capacity and permeability.

4.4 Water Table Elevation and Fluctuation.

5. **Structural Data.**

5.1 Live loads over the structure.

3.3 Data Checklist for Bridges.

1. **Geographic Data.**

1.1 Sub-Project Description
- name and location
- background to sub-project
- function or type
- environmental impact
- problem, solution, and expected result

1.2 Describe Existing Hydraulic Structures
- maintenance
- scour
- sedimentation
- debris

1.3 Drainage Basin (Index) Map Scale : 1" = 1 mile
- channels
- embankments, roads, cross-dams
- basin boundaries
- existing and proposed strutures

1.4 Topographic Map (Scale : 4" = 1 mile;
Contours : 1' to 5' intervals)

1.5 Site Plan (Scale : 1" = 50')
- spot levels at 25' intervals
- minimum area of coverage (depending on the length length of the bridge).
- proposed structure location
- road embankment location.

1.6 Cross-section of Existing or proposed Road

1.7 Longitudinal profile of Existing or Proposed Road for at least 1000' on either side of structure.

1.8 Channel Stability
- extent of scour or sedimentation in channels
- maximum depth of scour in channel
- map showing the movement of the river in recent years (if available)

1.9 Aerial Photographs, and Ground Level Photos (if available)

2. Hydrologic Data.

2.1 Tide Records (only required in Coastal Areas)
- hourly, daytime records for at least 1 year, when other records are not available.

2.2 Stage, Discharge Records
- daily stage records for at least 1 year, when other records are not availbale.

2.3 Recorded, or measured Maximum Flood Stage.

3. Hydraulic Data

3.1 Cross-Sections and Profile Measurements

- cross-section measurements to be taken at 500' intervals and extend for 1 mile upstream and downstream of the proposed site.

- longitudinal profile measurements at 1000' intervals for at least 2 miles upstream and downstream of the proposed site.

- the number and interval of cross-sections and profile measurement depends on the size and uniformity of the channel. In cases where the channels is straight and there are no breaks in topography, existing structures or side channels, the length between ceoss-sections and profile measurements may be increased to 1500'

- cross-sections should be extended far enough on both sides of the center-line to show the field elevation and changes in topography that may affect design (embankment and berm details).

- profile measurements include the bed level, right and left bank level (ground or field level) and water profile. When embankments are present, the right and left top level should also be included.

- Water profile should be taken at as large a stage as possible.

- the location of embankments, roads, other channels and existing temporary or permanent structures should be shown on the profile drawing.

3.2 Velocity Measurement.
- to be taken during flood.

4. **Geotechnical Data.**

4.1 Soil Survey, Soil Classification in Basin

4.2 Soil Classification at Site.

4.3 Sampling Bed Material to Determine Scour Potential

4.4 Soil Boring Samples
- to determine settlement, bearing capacity and permeability.

4.5 Water Table Elevation and Fluctuation.

5. **Structural Data.**

5.1 Road Classification (highway, feeder road, local road)

5.2 Traffic Volume along Road.

5.3 Live Road

5.4 Proposed Dimensions (height, width)
- vehicular requirements (no. of lanes, width of lanes)
- navigational requirements (minimum head clearance and width)

3.4 Data Checklist for Syphon.

1. **Geographic Data.**

1.1 Sub-Project Description
- name and location
- background to sub-project
- environmental impact

1.2 Describe Existing Hydraulic Structures
- operation
- maintenance
- scour
- sedimentation

1.3 Drainage Basin (Index) Map Scale : 1" = 1 mile
- channels
- embankments, roads, cross-dams
- basin boundaries
- benefitted area
- existing and proposed structures

1.4 Topographic Map (Scale : 4" = 1 mile;
Contours : 1' to 5' intervals)

1.5 Site Plan (Scale : 1" = 50')
- spot levels at 25' intervals
- minimum area of coverage 500'x 500' (depending on the size of the structure).
- proposed structure location
- embankment, diversion channel locations.

1.6 Cross-section of Existing or Proposed Embankment and Diversion Channel.

1.7 Land Use map (scale 1" = 1 mile)
- Village boundaries
- Cultivated land

1.8 Description of Cropping Pattern

1.9 Channel Stability
- extent of scour or sedimentation in channels

2. Hydrologic Data.

2.1 Rainfall records or name & location of nearest rainfall station.

2.2 Tide Records (only required in Coastal Areas)
- hourly, daytime records for at least 1 year, when other records are not available.

2.3 Stage, Discharge Records at the location of the structure.
- daily stage records for at least 1 year, when other records are not availbale.

2.4 Minimum Discharge Measurements

2.5 Recorded, or Measured Maximum Flood Stage.

Hydraulic Data

Cross-Sections and profile measurements of the Drainage channel and Irrigation Canal.

- cross-section measurements to be taken at 500' intervals for 1/2 mile upstream and downstream of the structure point.

- longitudinal profile measurements at 1000' intervals for at least 1 mile upstream and downstream of the structure point.

- Cross-section of the irrigation canal at structure point showing Full Supply Level.

- Design bed slope of the canal

- cross-section of the drainage should be extended far enough on both sides of the centerline to show the field elevation and changes in topography that may affect the design (embankment and berm details).

- profile measurements include the bed level, right and left bank level (ground or field level) and water profile when embankments are present, the right and left top level should also be included.

- Water profile should be taken at as large a stage as possible.

- the location of embankments, roads, other channels and existing temporary or permanent structures should be shown on the profile drawing.

3. Flow measurements at the structure point during high flood with the record of the highest flood level.

4. **Geotechnical Data.**

4.1 Soil Survey, Soil Classification in the Basin

4.2 Soil Classification at Site.

4.3 Soil Boring Samples
- to determine settlement, bearing capacity and permeability.

4.4 Water Table Elevation and Fluctuation.

5. **Structural Data**

Live Loads over the structure.

3.5 Data Checklist for Embankment.

1. **Geographic Data.**

1.1 Sub-Project Description
- name and location
- background to sub-project
- type
- environmental impact

1.2 Channel Stability.
- bank movement
- scouring
- bend of the river
- sedimentation

1.3 Topographic Map (Scale : 4" = 1 mile;
Contours : 1' to 5' intervals)

1.4 Cross-section of the proposed embankment.

1.5 Land Use map (scale 1" = 1 mile)
- Village boundaries
- Cultivated land

1.6 Description of Cropping Pattern

1.7 Aerial photographs and ground level photos (if available)

2. Hydrologic Data.

2.1 Rainfall records or name & location of nearest rainfall station.

2.2 Tide Records (only required in Coastal Areas)
- hourly, daytime records for at least 1 year, when other records are not available.

2.3 Stage, Discharge Records (only required in upland areas.
- daily stage records for at least 5 years, (May through October) and high flood level records for at least 10 years.

2.5 Recorded, or measured maximum flood Stage in the channel

3. Hydraulic Data

3.1 Cross-Sections of the channel at suitable intervals when the both banks are to be embanked.

- long and cross-sections along the profile of the proposed embankment at an interval of 1000' and 500'-0' respectively.

- cross-sections should be extended a few feet beyond the right of way both sides from the center line of the proposed embankment.

- profile measurements include the bed level, right and left bank level and water profile. When embankments are present, the right and left top level should also be included.

- water profile should be taken at as large a stage as posible.

- the location of roads, other channels and existing temporary or permanent structures should be shown on the profile drawing.

4. Geotechnical Data.

4.1 Soil Survey, Soil Classification in the Basin

4.2 Soil Classification along the embankment.

4.3 Soil Boring Samples
- to determine settlement

4.4 Water table elevation and fluctuation.

VERIFICATION OF DATA.

Whatever may be the sources of design data, it is important to verify these data package by the field engineers. In that case, the works of the design engineer will be easier and faster. Following are the points that should be considered and given bellow :

1. **Geographic Data.**

   1.1 All maps must include the following details :

   - North arrow
   - latitude to Longitude Co-ordinals
   - Scale
   - Data drawn
   - Signature of signing authorities

   1.2 All maps must be checked that they contain all updated information.

   1.3 The size of the existing structure over the channel must be noted & verified.

2. **Hydrological Data.**

   2.1 Field engineers must consider the post Construction condition when estimating the flow discharge. The flow in a channel often changes after sub-project implementation due to :

   - Closure on other branches of the channel
   - flood embankments that concentrate the entire discharge into the channel cross-section.
   - polder embankments which exclude entry of flow water during high tide level or high river stage.
   - re-excavation of the channel that improves channel conveyance and increases the flow discharge.

These change to the flood discharge are of the very difficult to quantity, but at least they must be properly documented.

2.2 Check whether or not the location and method used to measure (calculate) velocity of discharge is correct and conforms to the post construction condition.

2.3 Normally, the designers are interested in the maximum and the minimum water levels that are likely to occur during the critical period of flow.

2.4 If published hydrologic records are submitted the name & location of the station must also be given.

2.5 Field engineers should try to interview as many farmess as possible.

3. Hydraulic Data.

3.1 Cross-section and longitudinal survey drawings must show the following details

- Scale
- all pertineal dimension
- location of embankments, roads, other channels and existing temporary permanent structure.
- date drawn
- signature of the signing authorities

3.2 The bench mark used for the engineering surveys must be correct and permanent with respect to the datum.

4. Geotechnical Data.

4.1 Field engineers must ensure that all required soil information is collected and that the locations & grown elevation of the bore holes are correctly shown on the site plan.

CHAPTER II

HYDROLOGIC DESIGN

A.N.M. WAHEDUL HUQ

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1.0 INTRODUCTION :

Design of Hydraulic Structures consists of Hydrological part, Hydraulic part and then Structural and Foundation part. For correct design, related data are needed. Even in the case of culvert and bridges hydrological information are no less important than foundation and other data. In this part of the paper only the hydrological analysis for a regulating structure will be discussed.

Design of hydraulic structure will be poor if the data provided to the design office is inadequate and poor. The performance of the structure to adjust to flow system as well as proper gate operation will depend upon proper dimension and elevation of invert as well as other structural elements. A bridge with inadequate information of catchment may need less opening but may endanger safety of the stucture. So correct data is a must for studying hydrological as well as other part of design.

2.0 ESTIMATE OF DISCHARGE BY SIMPLE METHOD :

2.1 INTRODUCTION

One of the first steps in the design procedure is the estimation of the peak flows to be handled by the structures. Many methods are available for flood flow analysis but three methods have been developed especially for this region. The first has been developed for India and the second for Bangladesh. Both are simple to apply and they provide comparable results.

These methods 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). Second by LDL for Coastal Embankment Project and the third is a method proposed by Desai and Thirmalai two Indian Engineers. The methods are described below :

2.2 IECO METHOD (From graph and chart)

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.

2.3 LDL(IECO) METHOD FROM DRAINAGE MODULUS :

This is very simple method. The whole country has been divided by different isohyets of runoff intensity in inches/day. From this intensity if catchment area can be correctly delineated, the volume of runoff day and corresponding peak discharge can be found out. Opening can be fixed depending upon flow condition either in tidal or non-tidal zone. (Fig. 4)

2.4 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.Thermalai 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 drainge basin area.

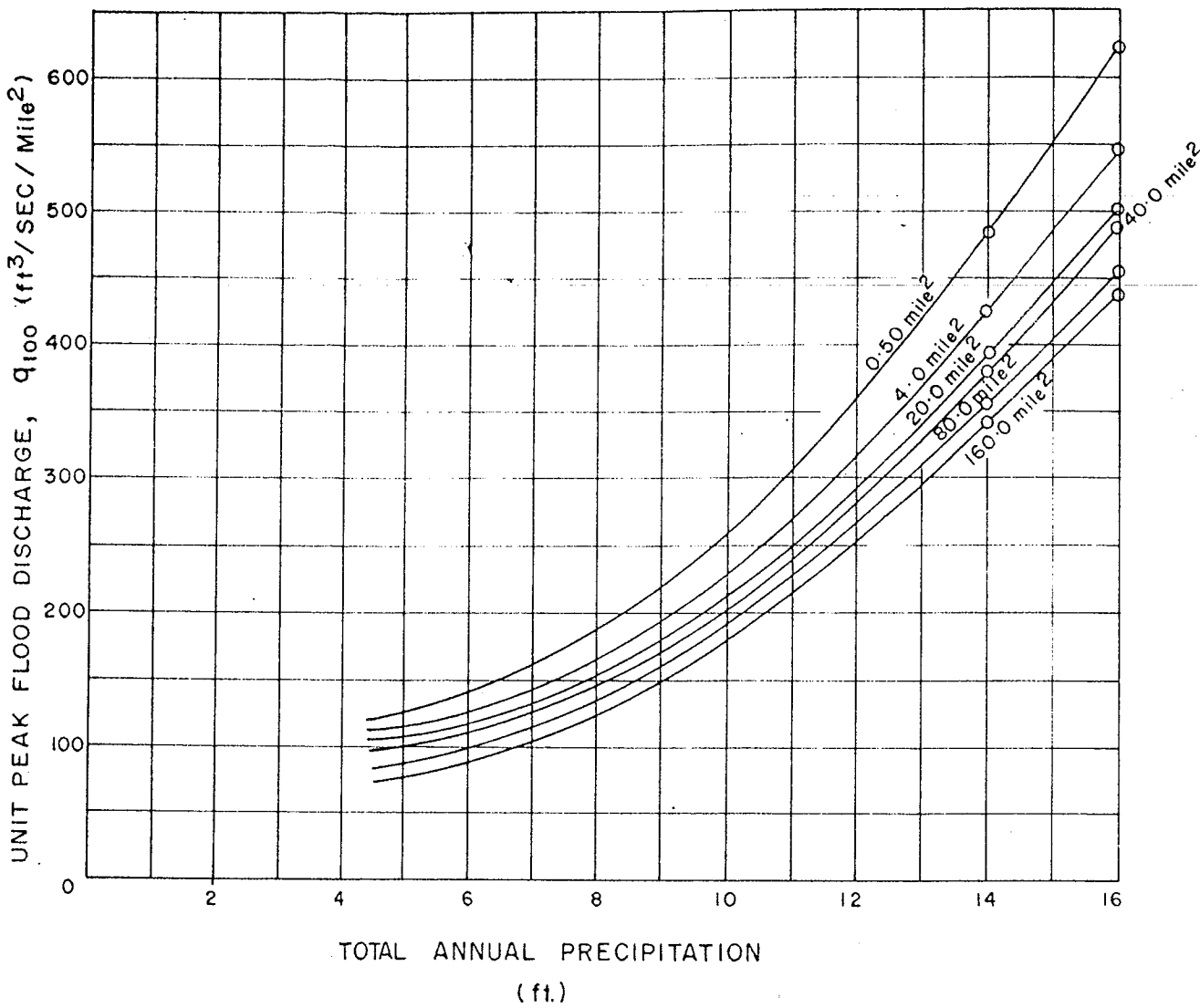
2. Estimate the 2 year 1-hour rainfall for the basin from Figure 5.

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 5).

5. Enter Table 1 (taken for Desai and Thermalai) and obtain an estimate for K (where K = 1"X/10 where X is the discharge for basin area of 2.6 sq.km. or 1 sq.mile).

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



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)

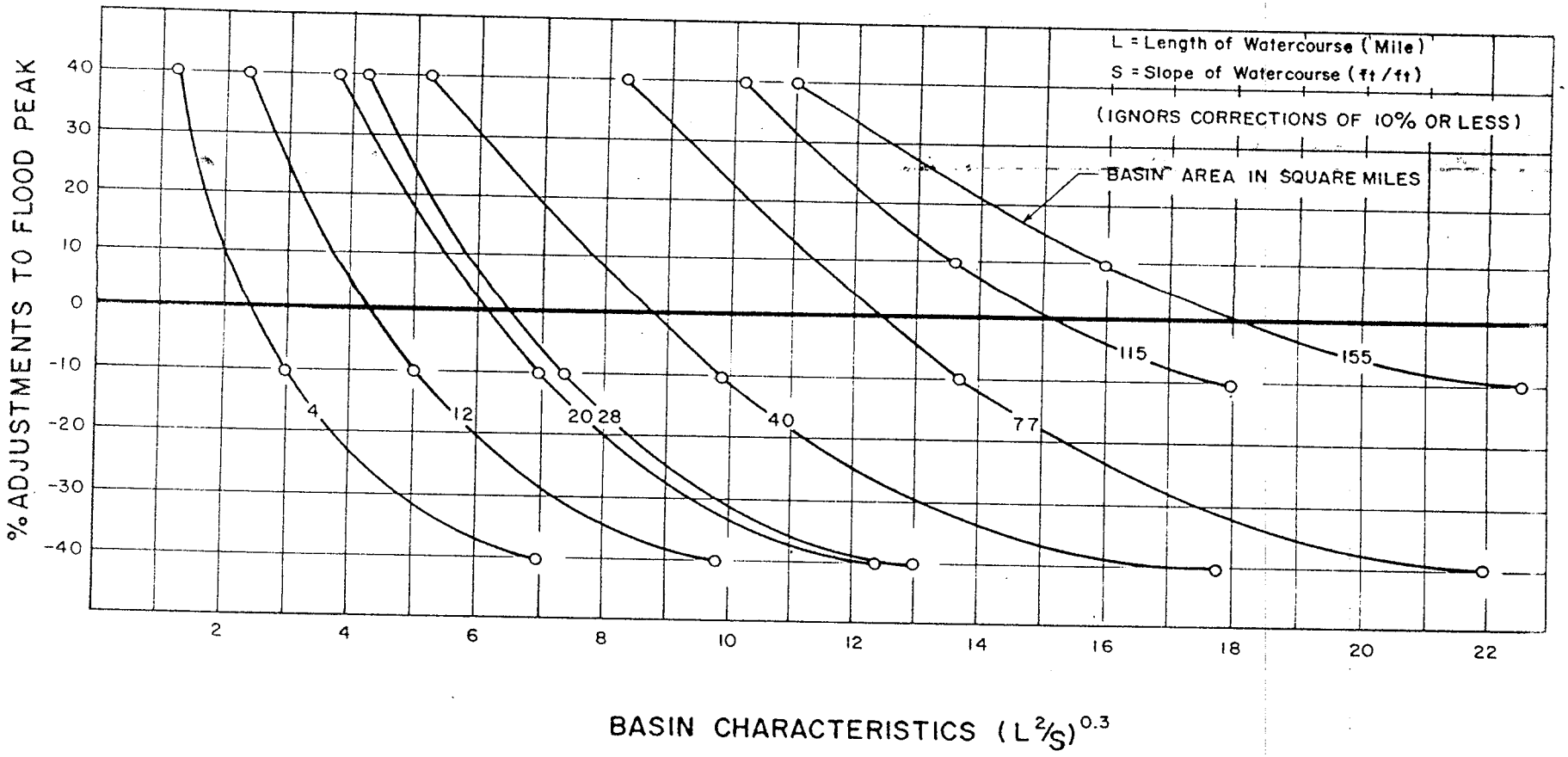


FIGURE 2. ESTIMATED PEAK ADJUSTMENTS FOR BASIN SLOPE AND SHAPE CHARACTERISTICS.

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

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

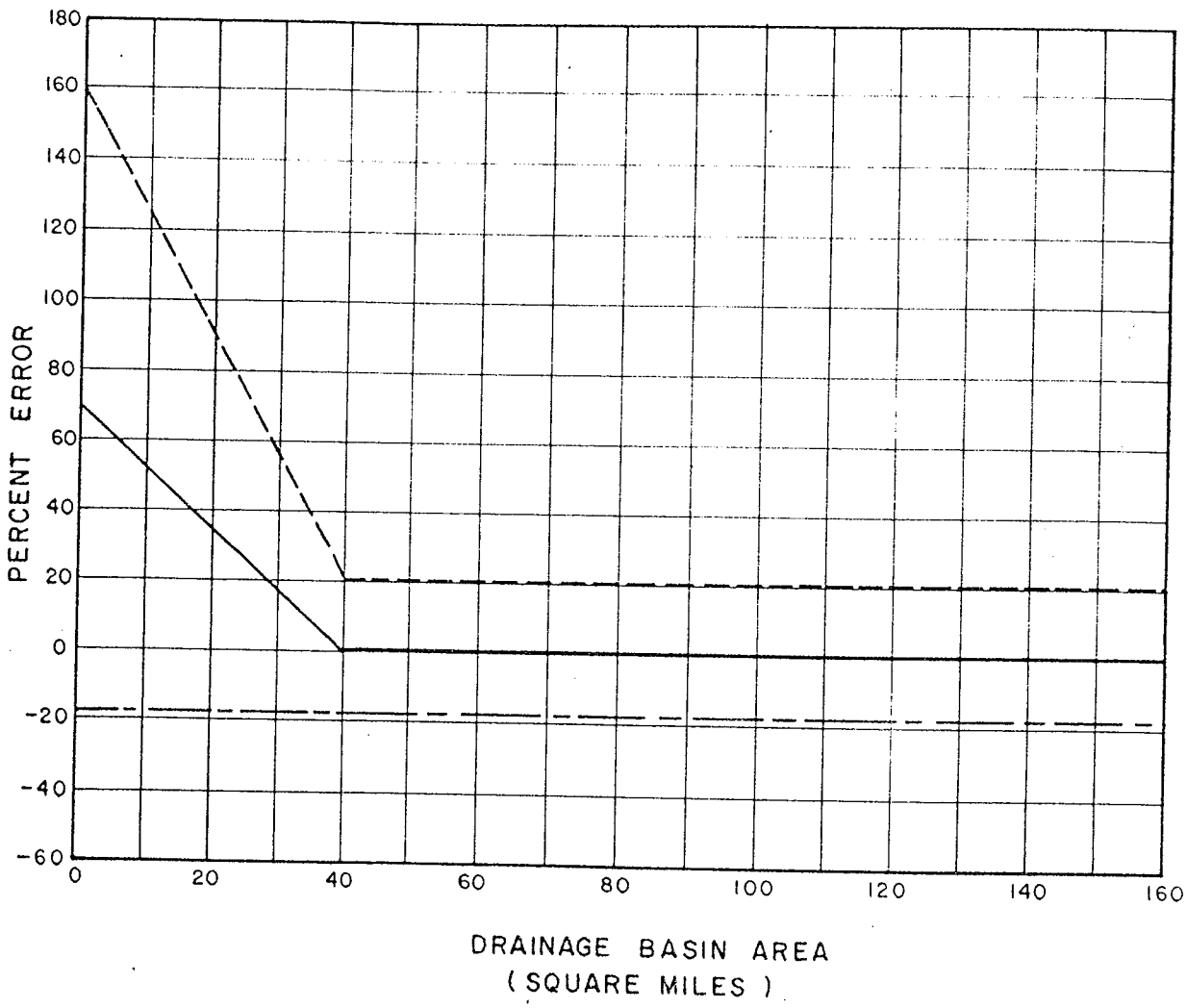
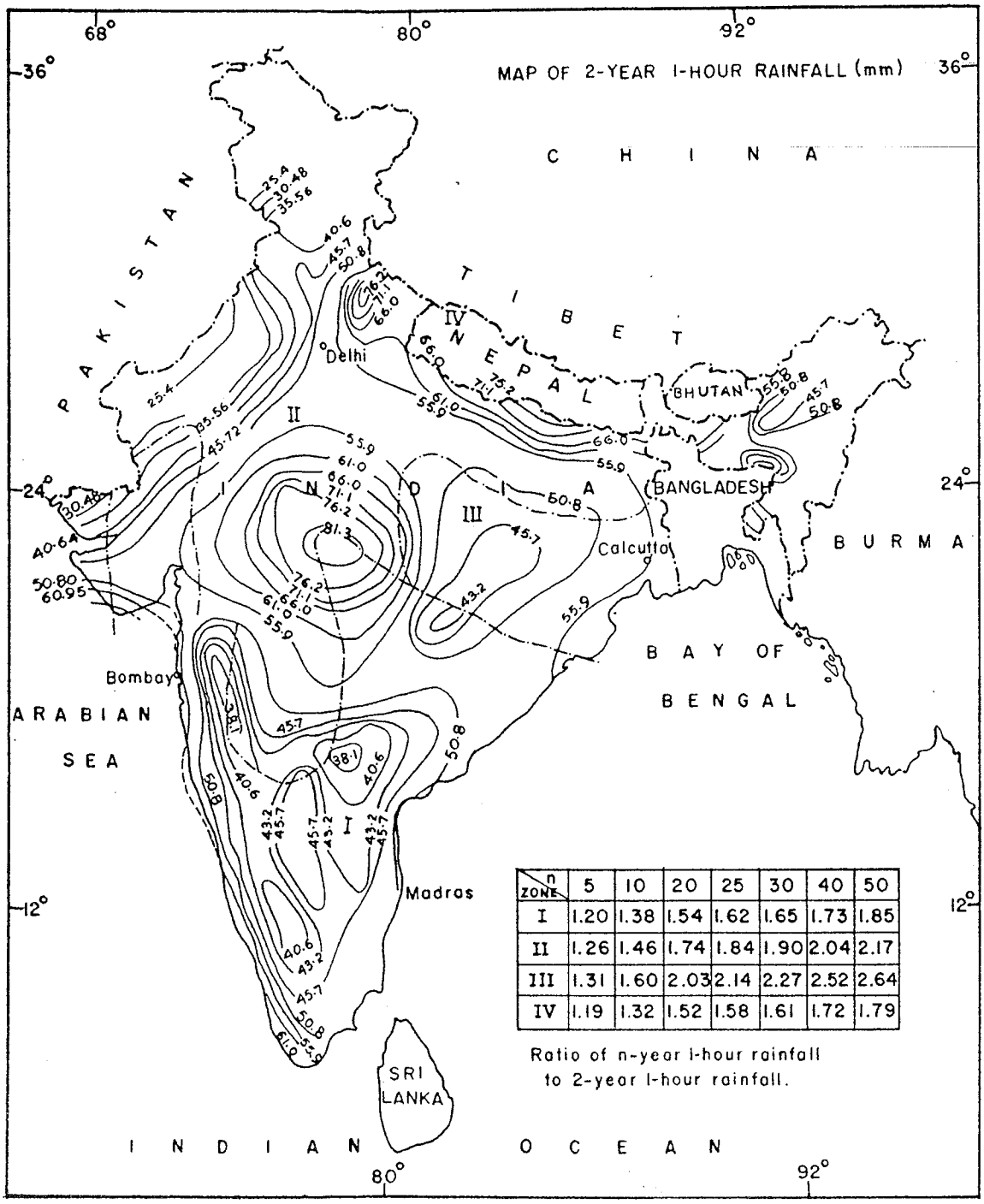


FIGURE 3. ERROR AS A FUNCTION OF DRAINAGE AREA

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



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

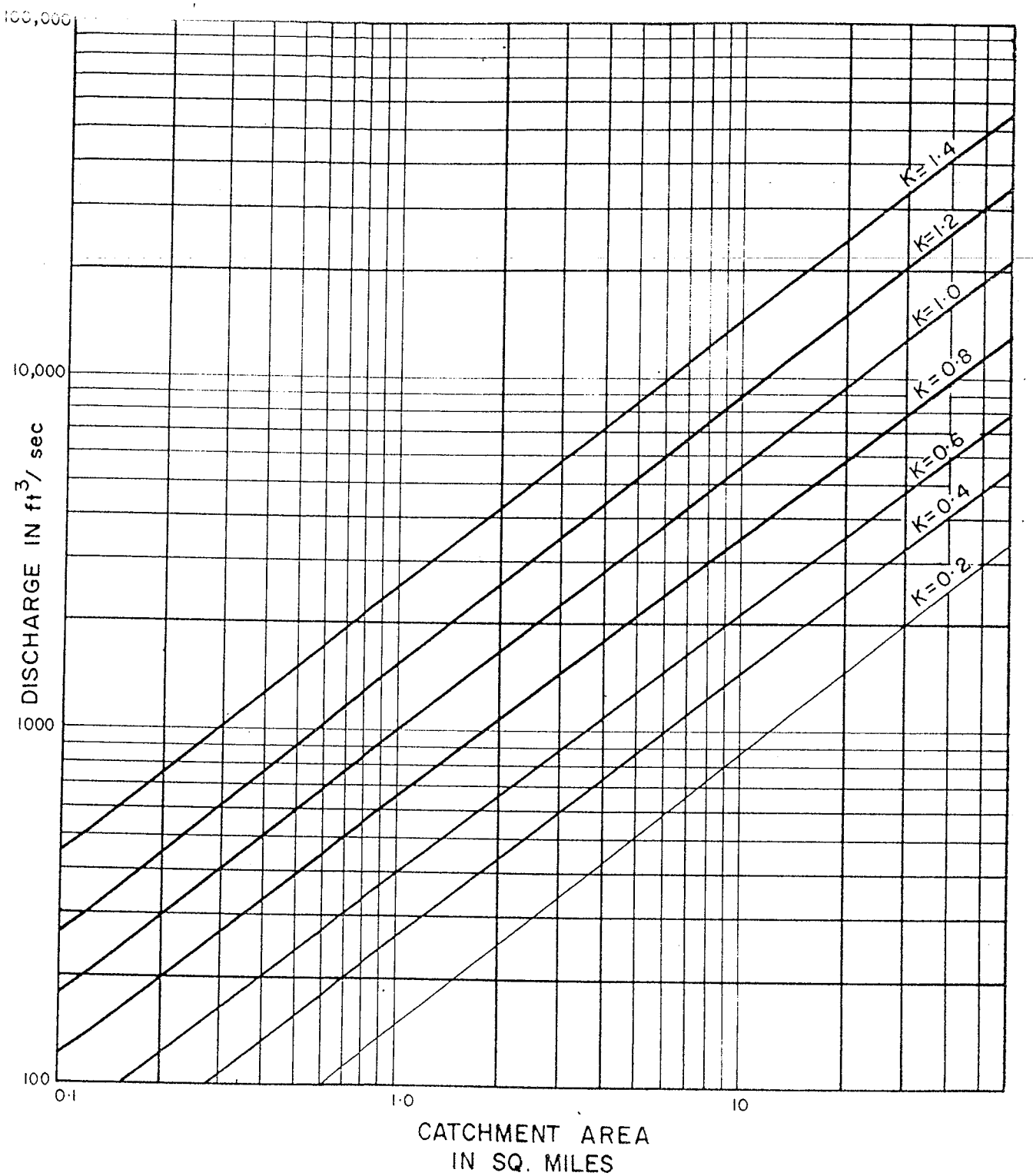
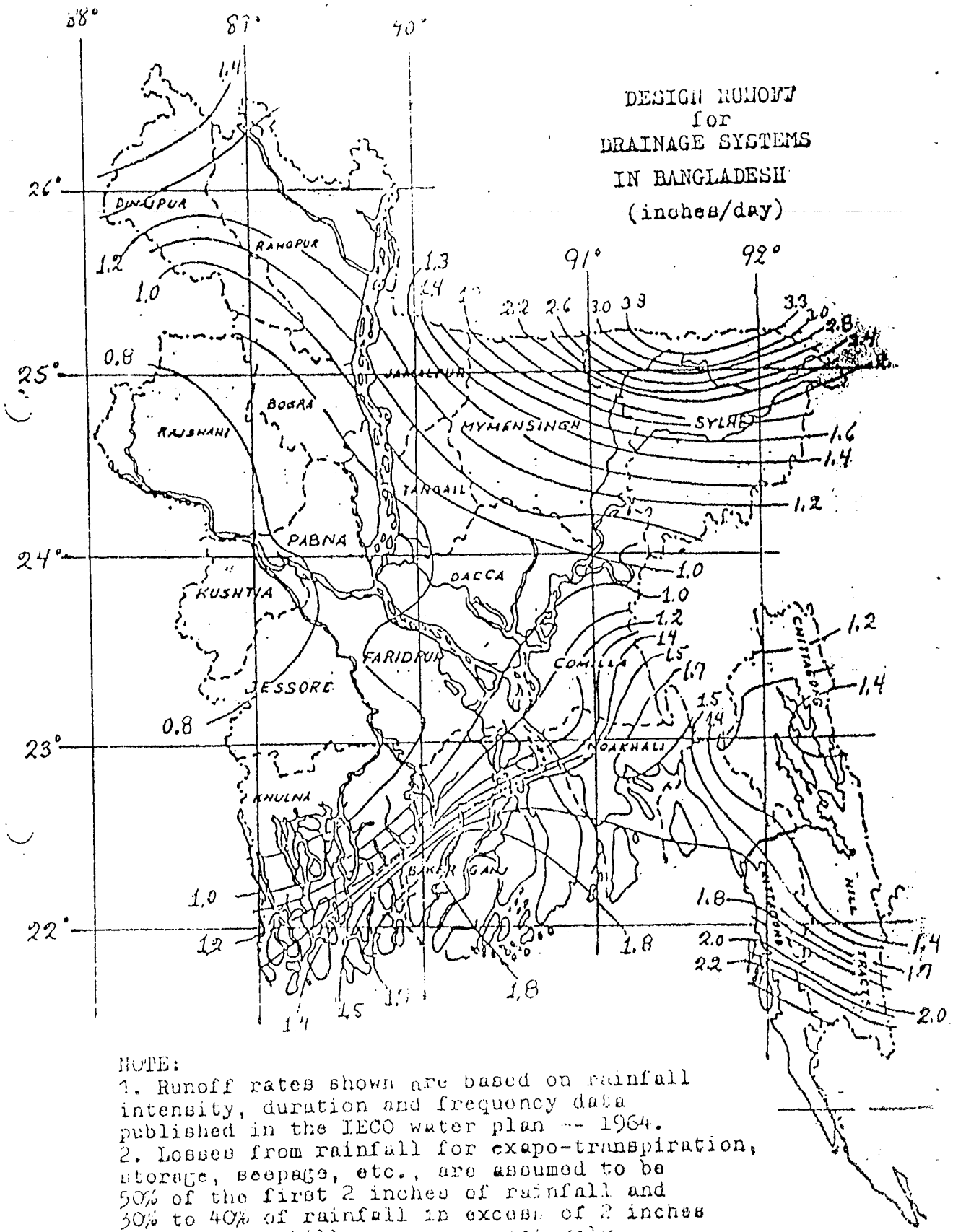


FIGURE 5. DISCHARGES FOR VARIOUS 'K' VALUES.

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

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



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.
(LDL 1968)

TABLE I

"K" VALUES FOR VARIOUS TERRAIN

50-year 1-hour Rain fall		Statistical		Slope			Ahmadnagar-Vikrabad plateau and Ajanta range and for similar areas		
(mm)	(inches)	Hilly 10% and above	Rough hilly 5%–10%	Rolling 2%–5%	Fall to rolling 1–2%	0–1%	5%–10%	2%–5%	0–1%
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	6.6	1.34	—	—	—	—	—	—	—

*Values suggested.

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

Check these runoff estimates against other estimates and select what is, in the Engineers opinion, the most suitable dischare. The estimated discharge at times vary in different calculations. This should also be checked by examining x-sections of channel at various places.

3.0 OTHER METHODS FOR CALCULATION OF VENTAGE OF REGULTOR :

1) By using Nomograph.

2) By Flood Routing.

4.0 CALCULATION OF VENTAGE BY NOMOGRAPH :

Nomograph is used both in tidal and non tidal zone. In tidal zone ventage is established by finding out discharge from tidal lockage consideration. Lockage period generally varies from 8 to 12 hours depending upon the river levels and structure location.

The purpose of regulator in non-tidal is to exclude inflow into cultivated areas from high river stage during monsoon season, and to drain this area when the river is in receeding stage. Now a days these structures are also used to retain water during post monsoon period. It may also be used for flushing when the river level permits. The Nomograph are prepared to determine the capacity of drainage regulator. The procedure in short is described below :

a) To prepare area-elevation curve

b) To prepare capacity-elevation curve

c) To collect rainfall records

d) To collect evaporation record

e) To determine the volume of water accumulated in the poldered area.

f) To determine the elevation to which the store water will rise .

g) From the level and from curve the net area inundated can be found out. From the capacity elevation curve and volume of area stored in each month the elevation can be found out. But unless the rive stages fall below that level, the drainage can not start. The time in which drainage can occur decide on how fast the river falls.

The rate of fall can be determine by the gage hydrograph. This hydrograph is constructed by considering average of a number of years of record. Generally water remains at high elevation until after 1st week of October when rain ceases and rivers begin to fall. It generally takes several weeks to drain the land upto desired level and in this way the control passes from tail water to head water level.

4.1 TIDAL REGULATOR :

Drainage requirements are determine by catchment area, assumed rainfall and tide lockage. Tide lockage is determined by the range of tide and the elevation to which it is proposed to drain the country side of the embankment. This level is generally taken 1 ft. above ground level. Tide lockage occurs from this elevation upto the elevation of high tide as water outside the dike is higher than the water inside.

4.2 DISCHARGE THROUGH REGULATOR AND TIDE CURVE :

When the tidal water elevation is lower than one half the sum of the critical depth plus the height of the vent. The discharge is constant and independent of tail water fluctuations until the tail water rises above the elevation $\frac{D + D_c}{2}$. The amount of drainage possible for each culvert has been determined through the use of curves based on experiments and published by the U.S. Bureau of Public Road.

EXAMPLE :

Average G.L. = + 5.70

L.W.L (monsoon) = 0.00

H.W.L monsoon = +10.00

Size of Box = 5' x 6'

Design W.L C/S = + 6.70

Invert level = 0.00

$$\frac{HW}{D} = \frac{6.70}{6} = 1.12, \quad \frac{Q}{B} = 50, \quad Q = 250 \text{ cfs.}$$

[Ref curve 1058 (Fig.7)]

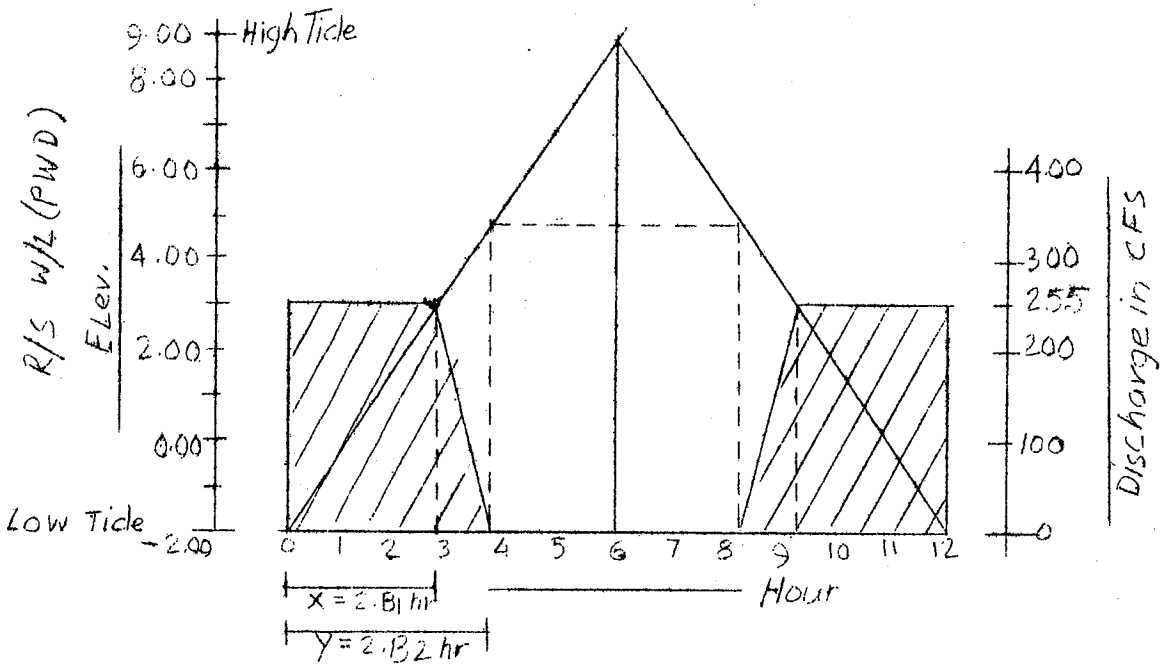
$$D_c = \left(\frac{q^2}{g}\right)^{1/3} = 4.25 \text{ ft.}$$

$$\frac{D_c + D}{2} = 5.10 \qquad \text{T.W. El.} = 5.10$$

The control remains at the entrance until tail water reaches to 5.10. Discharge in all stages of tail water from 0 to 5.1 remains constant at 250 cfs.

The sluice is flowing with tail water control from + 5.10 to tide lockage at + 6.70 the discharge decreased in a straight line from 250 cfs to zero on the basis of straight line tide curve assumed as average.

The total volume under the discharge curve for one tide cycle and the average discharge for a 24 hour period is computed from the curve as follows :



Tide and discharge curve.

Fig. 8.

The drainage is constant, by a similar triangle, (Fig. 8)

$$\frac{X}{5.1} = \frac{6}{10} \qquad X = 0.6\ (5.1) = 3.06 \text{ hours.}$$

Time drainage reaches zero from change of tide to tide lockage

$$\frac{Y}{6.7} = \frac{6}{10} \qquad Y = 0.6\ (6.7) = 4.00 \text{ hours.}$$

Total volume under drainage curve for one tide cycle

$$V = [2\ (3.06 \times 250) + \frac{2\ (4.0 - 3.06)\ 250}{2}]\ 3600$$

= 6354000cft/12 hours = 147 cfs/vent.

From area and drainage modulus as given by IECO drainage requirements may be calculated and vent no. selected. But during hydraulic design peak discharge of 250 cfs/vent should be taken as it will give worst condition and structure will experience such a state of affairs.

If the design water level is less than discussed in above example with the same tide range, lockage may be increased and discharge will reduce. In this case no. of vents will increase. So selection of design water level, invert level has considerable effect on the discharge capacity of sluice.

5.0 CALCULATION OF VENTAGE BY FLOOD ROUTING :

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 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% 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 more than a period of 72 hours may be considered upto 5% of the total project area.

A post-monsoon 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. 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.

Pre-monsoon flood routing procedure have been discussed with basin runoff hydrographs generated both by Unit Hydrograph Method and Richard's Method. The computed results are discussed in Art. 8.0.

6.0 PRE-MONSOON FLOOD ROUTING :

DESIGN PROCEDURE

6.1 DEVELOP A DESIGN STORM :

6.1.1. Point Rainfall

a) Locate basin on Fig. 9 and find the 4-month rainfall index. (say 50")

b) Determine from Fig. 10 the ratio to apply to the four-month index to compute the index rainfall for the selected time interval.

c) Determine from Fig. 11 the ratio to apply to the index rainfall for the selected time period to determine the rainfall for the selected recurrence interval.

d) Multiply the index rainfall for the selected time period determined by step 1 by the ratio determined by step 1.c. The calculation is in Table 2.

TABLE - 2

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

e) Multiply the 4-month index by the daily combined indices in Table 2. This gives the accumulated point rainfall volume in inches for storms of 1,2,3,4 & 5 days duration.

6.1.2. Equivalent Uniform depth of Rainfall.

a) Compute the reduction factor for rainfall variability. (Table 3).

TABLE - 3

RAINFALL VARIABILITY

DISTANCE FROM STORM CENTRE	5 DAY STORM PERCENTAGE OF POINT RAINFALL
1/4	100
1	88
2	81.7
3	77.5
4	74.2
5	72.0
6	69.8
7	67.8
8	66.0
9	64.5
10	63.1

b) Multiply the point rainfall volumes by the rainfall reduction factor. Separate accumulative totals into daily increments.

6.1.3. Rainfall Time Distribution

a) Arrange the daily increments in an order giving the worst flood condition. For a 3-day storm use the sequence 3,2,1. For a 5-day storm use 3,2,1,4,5. For a 6-day storm use 6,3,2,1,4,5 and so on.

3. For the 3rd period of the 2nd day we will assign the remainder of the total 2nd day rainfall or 0.87 inches.

4. The 1st day is similarly handled with the remainder of 0.00 inches in the 3rd period.

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

TABLE - 5

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

6.2 DETERMINATION PERCENTAGE OF PADDY AND NON-PADDY LAND :

Determine land classification as percentage of area in paddy land and non-paddy land. Whenever possible these percentage 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 percentanges on Table 6 can be used as an average.

TABLE - 6

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

6.3 DETERMINE RAINFALL EXCESS (RUNOFF DISTRIBUTION) :

6.3.1 Paddy Land.

Separate the rainfall losses from the rainfall time distribution determined in Table 5 by assuming the entire basin area consists of paddy land.

6.3.2 Non-Paddy Land.

Separate the rainfall losses from the rainfall time distribution determined in Table 5 by assuming the entire basin area consists of non-paddy land.

6.3.3 Weighted Basin Runoff.

Compute the weighted basin runoff distribution by multiplying the net runoff obtained in steps 6.3.1 and 6.3.2 by the respective land classification percentage as in Table 6.

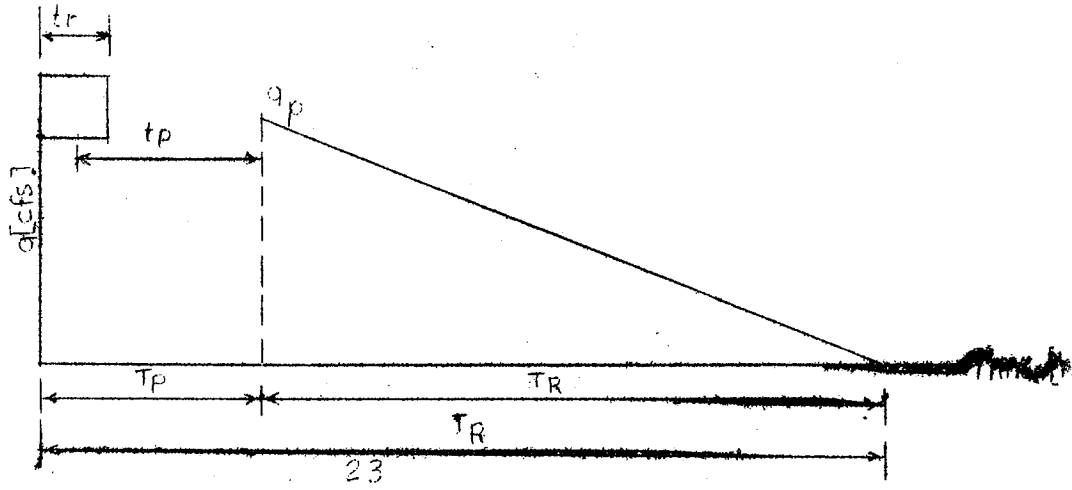
6.4 UNIT HYDROGRAPH :

6.4.1 Introduction.

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 uniformaly over the entire basin area during a unit time interval of 6 hours.

6.4.2 The Unit Hydrograph Shape.

Most of the basins we will encounter will be ungauged with scanty hydrologic available. Therefore, rather than using the more complicated and classical curvilinear hydrograph shape we will use the simpler traingular 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 hydrograph 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 or rainfall excess to peak.

6.4.3 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 : $\bar{L}$ = 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

6.4.4 The Elements of the Unit Hydrograph.

1. Time of concentration $T_c = 31 \left(\frac{L^2 n^2}{S^5}\right)^{0.3}$

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

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

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

4. Coefficient C_t : $C_t = \frac{t_p}{(L \times \bar{L})^{0.3}} = \frac{13.5}{(9 \times 3.6)^{0.3}}$

$= 4.76$

5. Coefficient C_p : 0.43, obtained from Fig. 12.

6. Unit-graph Base Length :

$$T_b = \frac{1.2\ T_c}{C_p} = \frac{1.2 \times 22.5}{0.43} = 63 \text{ hrs.}$$

7. Peak Discharge : $$q = \frac{2A}{T_b} = \frac{2 \times 10707.2}{63} = 340 \text{ cfs.}$$

The completed hydrograph is shown in Fig. 13.

6.4.5 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 7 while Fig. 14 represents the shape of the runoff hydrograph.

6.5 THE RUNOFF HYDROGRAPH RICHARD'S METHOD.

6.5.1 RICHARD'S EQUATION.

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

(ii) $$\frac{t^3}{t+1} = \frac{N.C.L^2}{K.S.R.f(a)}$$

(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) " 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 $t = t^3 \left(\frac{r}{L}\right)^2$

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 are with radius r.

6.5.2 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. 15 for the area of the catchment

d) Compute R using Equation (i)

e) Select K for the catchment using Table 8 and compute K.R.

f) Determine C from Fig. 16 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 1,2,3,4,5 ..... 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 - 8

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

TABLE - 9

RUNOFF HYDROGRAPH COMPUTATIONS (RICHARDS METHOD)

Distance from outlet in miles	$\frac{a_1}{a}$	$\frac{r^2}{L}$	Rising Flood		Falling Flood	
			Q_R (in cfs)	t_R (in hour)	Q_F (in cfs)	t_F (in hour)
(1)	(2)	(3)	(4) (1426xcol.2)	(5) (36xcol.3)	(6) (1426-col.4)	(7) (36+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

6.6 PRE-MONSOON FLOOD ROUTING :

6.6.1 Introduction.

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

6.6.2 Flood Routing Equation.

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

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

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

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

Substituting the terms.

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

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

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

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 + O curve.

After completing the routing computations the outflow hydrograph i.e, discharge vs time is plotted and presented in Fig. 18. To assist in determining the extent of crop damage, a basin water surface elevation vs time is also plotted and presented in Fig. 19. The result of the computations are discussed in Article 8.0.

7.0 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.19). During the period the gates are closed 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 10, Table 11 and Table 12 respectively. The basin water surface elevation after each month is computed in Table 13 and presented in Fig. 20. It may be observed that the basin water level always remains below the level in the peripherial rivers. The difference is the benefit accrued

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

RAINFALL RUNOFF VOLUME (NON PADDY LAND)

TABLE 10

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

RAINFALL RUNOFF VOLUME (PADDY LAND)

TABLE 11

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

RAINFALL RUNOFF VOLUME (WEIGHTED BASIN AVERAGE)

TABLE-12

Date	River W/L (in ft.PWD)	Run off volume (Acre ft.)	Gate position	Inside	
				Storage Acre ft.	Work 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

COMPUTATION OF INSIDE STORAGE AND WATER LEVEL

TABLE- 13

Date	C/s water level W_1	R/s water level W_2	Δh $(W_1 - W_2)$	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	23.56	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			

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

TABLE 14

out of the project by reclaiming land which were subjected to annual flooding during pre-project condition. It may also be observed from Fig. 20 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 14.

8.0 DISCUSSION AND CONCLUSION :

The result of the permonsoon routing for 3 vents of size 5'x6' are tabulated below in Table 15.

TABLE - 15.

PRE-MONSOON FLOOD ROUTING RESULTS (3 VENTS 5'X6')

Item	Unit hydrograph Method	Richard's Method
1) Runoff hydrograph peak discharge	1283 cfs	1426
2) Time to runoff hydrograph peak	54 hours	36 hours
3) Time base of runoff hydrograph	120 hours	72 hours
4) Outflow hydrograph peak	565 cfs	615 cfs
5) Basin storage level correspon-ding to peak discharge	+19.80 (PWD)	+20.15 (PWD)
6) Percentage of area submerged corresponding to peak discharge	8.97 %	9.63 %
7) Basin submergence level for more than 72 hours	+18.10 (PWD)	+18.25 (PWD)
8) Percentage of area submerged for more than 72 hours	4.96 %	5.30 %

From Table 15 it may be observed that due to greater peak discharge of runoff hydrograph and also due to the early peak by Richard's method the basin submergence level is little more than that by unit hydrograph method and exceeds tolerable submergence limit to some extent. In this case the designer may have a choice to go for next higher size of the structure.

Considering the simplicity of computation by Richard's method and when there is time constraint for detail design computation, the Richard's 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. 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 vent regualtor of size 5'x6'. It may therefore be concluded that a 3 vents drainage regulator of size 5'x6' placed at the outfall is adequate for the basin.

9.0 ESTIMATE OF DISCHARGE FOR WRS AT UNGAGED LOCATION :

Problems arises during design of water retention structure due to non availability of Hydrological Data. Most of the

structure demand comes from hill areas where catchment boundary and discharge are not available. Correct identification of boundary is also difficult. Moreover, gauge level are not also available for long period. In many cases data is supplied by local information which is not reliable. If it is very much urgent to construct structure in such cases, discharge is to be estimated from X-section. Long section should be made available to design office. If debris come from upstream, from visit of the locality velocity may be estimated and should be compared with computed velocity. Velocity should be estimated at several sections and average is to be taken. In flashy channel the bed slope does not represent water slope during flash flood. Post flood estimate seems to be in lower side. Calculated velocity should be increased by 10-20 percent. It is advisable to defer design of the structure for at least one year for data collection in such cases.

10.0 CONCLUSION :

Calculation of peak discharge and fixation of opening is the 1st phase of design of sluice, regualtor or water retention structures. This will be followed by Hydraulic and Structural design. If the discharge can correctly be estimated the structure will not only be economical but also will not create problem during operataion. An inadequate estimate of discharge may induce scouring in downstream and may need constant maintenance whereas an over-estiamte of discharge will increase the size of structure.

CHAPTER III

HYDRAULIC DESIGN

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CHAPTER III

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1.00 Introduction

Structures 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 structures. 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 structures like weirs, regulators, water retention structure and sluices.

2.00 Types of flow

2.01 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 :

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

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

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

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

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 .... ($sec/ft^{1/3}$)

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

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

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".

2.02 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/ft^3

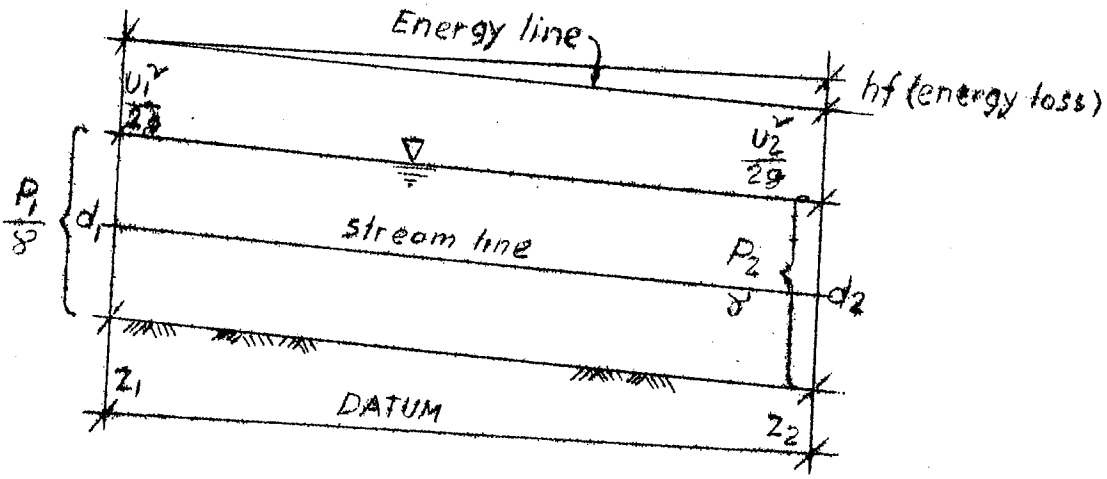

Fig. 2.1 : Definition Sketch for Bernoulli's Equation.

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.

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

Where hf = energy loss

g = acceleration due to gravity

z = height of bed above datum (ft)

d = depth of flow (ft)

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

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

(various terms are shown in Figure 2.2), allows for solution to gradually varied flow problems.

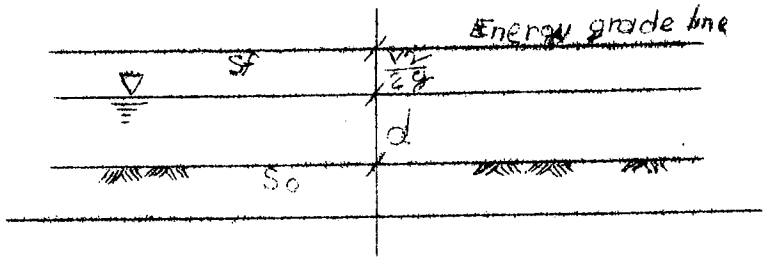

Fig. 2.2 : Definition Sketch for gradually varied flow.

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

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

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

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

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

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

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

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

or $$\frac{dd}{dx} = \frac{S_O - S_f}{1 - F_r^2}$$ where F_r 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 methods 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.

2.03 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.

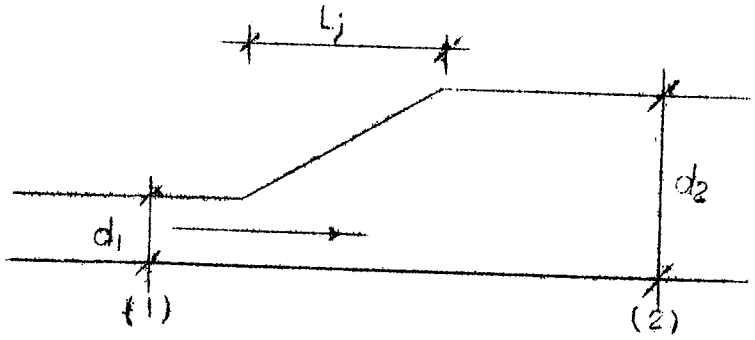

Fig. 2.3 : Definition Sketch for the Hydraulic Jump.

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 lenght of the jump and the energy loss at the jump.

Equations required are provided below :

Starting with this equation:

$$V_1^2 = gd_1 \left(1 + \frac{d_2 - d_1}{d_1}\right)\left(1 + \frac{d_2 - d_1}{2d_1}\right) \quad .... \quad (10)$$

and simplification leads to:

$$V_1^2/g = d_2 (d_1 + d_2)/(2d_1) \quad .... \quad .... \quad (11)$$

Using the expression for the Froude number.

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

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

$$d_2 = \frac{d_1}{2} \left[(1 + 8F_1^2)^{1/2} - 1\right] \quad .... \quad .... \quad (13)$$

Figure 1 in the Appendix can be used to determine the ratio d_2/d_1 for various Froude numbers, for stilling basins having vertical and parallel walls and a horizontal floor. The length of the jump varies with the Froude Number of the incoming flow and the tailwater depth (Figure 4, Appendix). In Figure 4, it can be seen that the relationship between the length of the jump, L_j, and the downstream depth, in most cases, can be taken to be

$$L_j = 6\, d_2 \quad .... \quad .... \quad (14)$$

The energy loss in the jet is obtained, using the equation

$$E = \frac{(d_2 - d_1)^3}{4\, d_2\, d_1} \quad .... \quad .... \quad (15)$$

and it is shown graphically in Figure 4 in the Appendix.

3.00 Hydraulic Structures

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

3.01 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 relationships 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 3.1

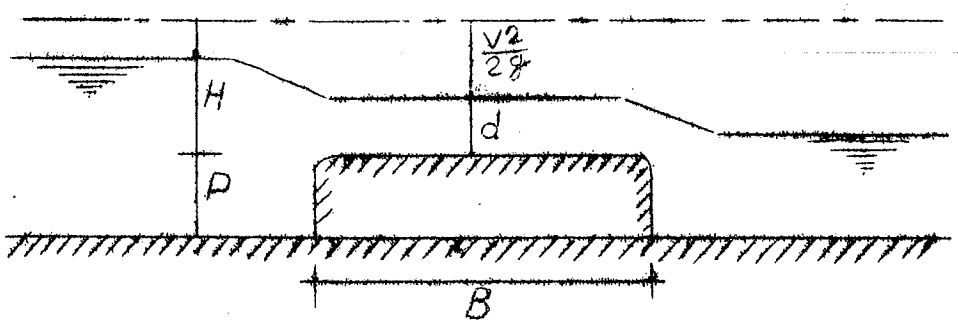

Fig. 3.1 : Definition Sketch for a Broad Crested Weir.

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

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

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

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:

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

or

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

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

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

figures 3 and 3.1 in Appendix 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 3.1 in the Appendix.

3.02 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 regulator 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 3.2. As the contraction becomes more and more severe for a given discharge, the water surface

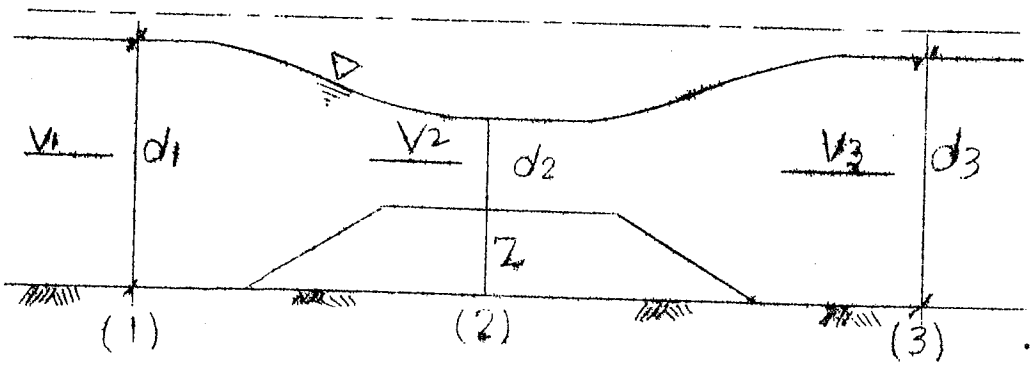

Fig. 3.2 : Definition Sketch for flow in contraction.

in 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 4.2 in the Appendix. The equation relating to depth of flow and velocity is as follows:

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

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

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

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

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 :

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

where the terms are given in Figure 3.3 and

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

and

H_{f2} = energy loss due to friction

H_{le} = energy loss due to expansion

C_e = expansion coefficient

$$Z_2 + d_2 + \frac{V_2^2}{2g} + H_{1c} + H_{f1} = Z_1 + d_1 + \frac{V_1^2}{2g} \quad \ldots\ldots \quad (25)$$

and $H_{1c} = C_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right)$ energy loss due to contraction

and H_{f1} = energy loss due to friction

C_c = contraction coefficient

The terms are illustrated in Figure 3.3 for a constriction in width and Table 3.1 gives the coefficient for various transition types.

Table 3.1

Transition Loss Coefficients

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

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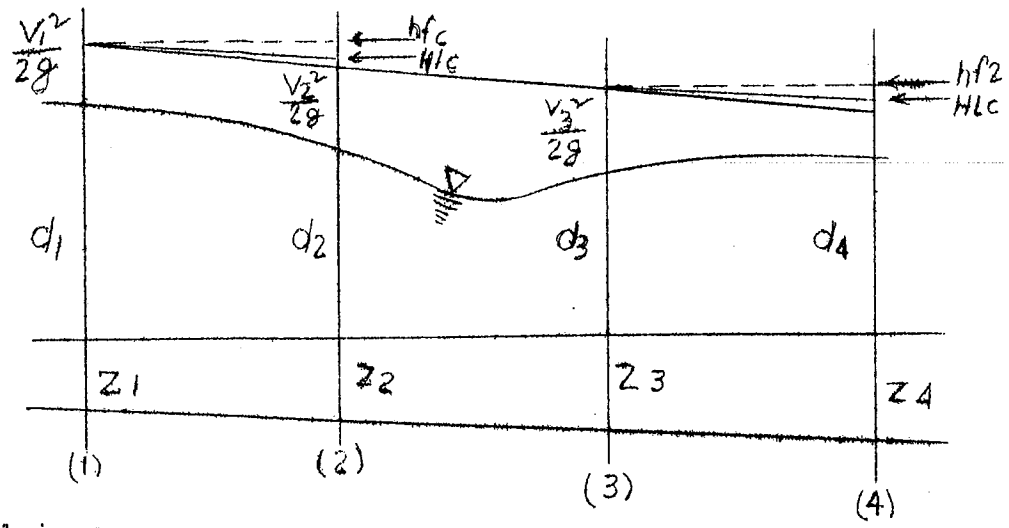
নকশা সার্কেল-২
বাপাউবো, ঢাকা-১২১৫

Fig. 3.3 : Flow depths at a contraction.



3.03 **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 slucies.

3.04 **Water Retention structure**

Water retention structures are designed to retain water for dry period irrigation and to check over drainage of beel areas. They are usually submersible 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. 3.4). against any possible erosion due to overflanking by flash floods in hill streams.

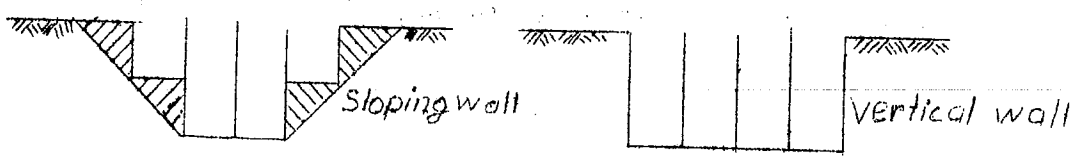


Fig. 3.4 : Water retention Structure.

4.00 Design Procedures

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

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

  Loose appron - Inverted filter.

4.01 Depth of scour and cut off wall

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

$$R = 0.90 \left(\frac{q^2}{f}\right)^{1/3} \quad \ldots \quad \ldots \quad (26)$$

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

f = Lacey's silt factor

$$= 1.76 \, (dmm)^{1/2} \quad \ldots \quad \ldots \quad (27)$$

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

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 4.1 below.

TABLE 4.1

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	Right angled bend	2.00	2.00R-Y

A value of 1.25 R 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.

4.02 **Estimation of design head**

Estimation of design head is very important in calculating the uplift pressure and floor thicknes 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 rou
- R/S water level (from river hydrograph)

Water Retention Structure.

- Upstream retention level and downstream minimum water level.

Sluices (coastal)

- Riverside high tide level and country side minimum water level
- Country side retention level and riverside low tide level.

4.03 **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 4.3), the exit gradient at its downstream end is given by

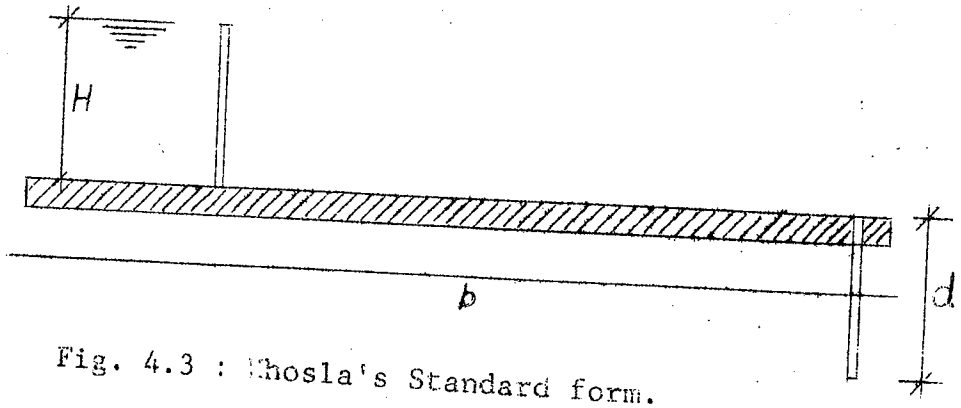


Fig. 4.3 : Khosla's Standard form.

$$GE = \frac{H}{d} \frac{1}{\alpha\sqrt{L}} \quad \cdots\cdots \quad \cdots\cdots \quad (28)$$

$$\text{where} = \frac{1 + \sqrt{1 + a^2}}{2} \quad \cdots\cdots \quad \cdots\cdots \quad (29)$$

$$\text{and } a = \frac{b}{d} \quad \cdots\cdots \quad \cdots\cdots \quad (30)$$

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 gradient should be ensured. Value of Khosla's safe exit gradient for different type of material is shown in table 4.2 below:

Table 4.2

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 and 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.4.4 below where H_L

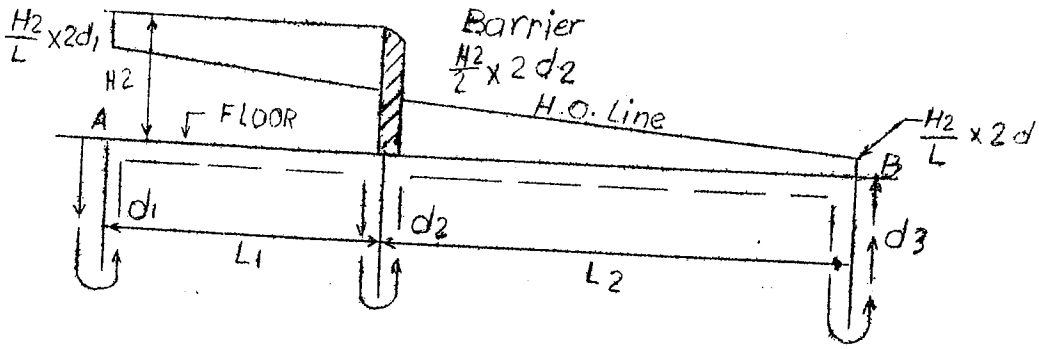


Fig. 4.4 : Bligh's creep.

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

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

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

$$= (L_1 + L_2) + 2(d_1 + d_2 + d_3) \qquad \ldots \qquad \ldots \qquad (31)$$

Hydraulic gradient

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

Head Losses equal to

$$\frac{H_L}{L} \times 2d_1, \quad \frac{H_L}{L} \times 2d_2 \quad \text{and} \quad \frac{H_L}{L} \times 2d_3 \quad \ldots \quad (32)$$

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 4.3 below

TABLE - 4.3

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 4.4 the total Lane's creep length.

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

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

$$= 1/3\,b + 2\,(d_1 + d_2 + d_3) \qquad \ldots\ldots \qquad \ldots\ldots \qquad (33)$$

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-4.4

TABLE-4.4

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

4.04 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.4.3 in Appendix) which can be used for determining percentage pressure at various key points under the structure. The simple profiles are shown in Fig 4.5 below :

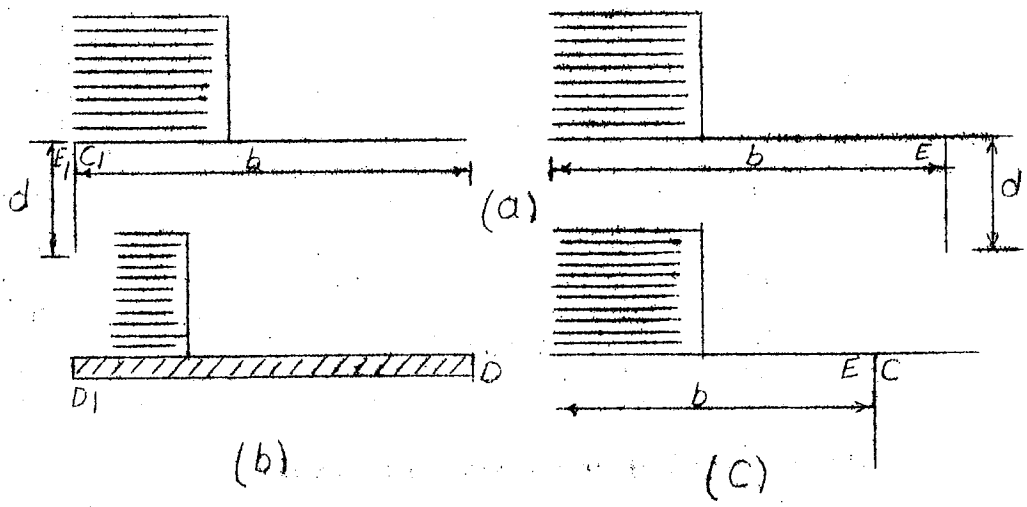


Fig. 4.5 : Khosla's simple profiles.

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) **Correction for mutual interference of piles**

$$C = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d+D}{b}\right]^{1/2} \quad \ldots \quad (34)$$

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) Correction for floor thickness

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. 4.6). Since the corrected pressure at E_1 should be less than the

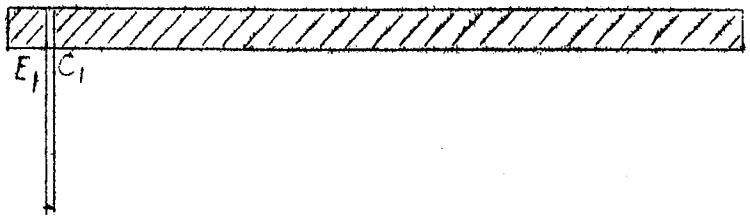


Fig. 4.6 : Definitation sketch for correction to floor thickness.

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) **Correction for slope of floor:**

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 4.5.

TABLE - 4.5

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.

4.05 Energy Dissipation and stilling basin design

The hydraulic jump was discussed in section 2.03 and its equations were derived. It was shown that :

$$\frac{d_2}{d_1} = \frac{1}{2}\left[\left(1 + 8F_1^2\right)^{1/2} - 1\right] \qquad \ldots\ldots \qquad \ldots\ldots \qquad (35)$$

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

$$L_j = 6\,d_2 \qquad \ldots\ldots \qquad \ldots\ldots \qquad (36)$$

$$E = (d_2 - d_1)^3/4d_2d_1 \qquad \ldots\ldots \qquad \ldots\ldots \qquad (37)$$

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 5, 6 and 7 in the appendix).

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 4.6

TABLE - 4.6

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

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 $F = \frac{V_1}{\sqrt{gD_1}}$. The details of SAF basin are illustrated in figure 7 in Appendix A.

- Low froude number stilling basin.

The development of a low froude basin design started with the basin III design and modified according to the results of model tests. Data obtained during 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 4.4, 4.5 and 4.7 in Appendix A). The recommended design is a relatively short stilling basin (L_s equals approximately $3d_2$) with chute blocks, baffle piers and dentated end sill (figure 4.6 in Appendix A).

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) :

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

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 0.375 W nor greater than 0.50 W. Usually, no baffle piers will be placed within 1.375 W 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 approximately 0.50 blockage. Any configuration with less than four baffle piers will need 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 baffle piers were tested, no difference in performance was noted when partial side

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 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 4.0 (Appendix A) 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 troublesome 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 6 (Appendix A), 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 5 (Appendix A) 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

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 4.6 (Appendix A). The distance from the chute blocks to the baffle piers (X in figure 4.6 Appendix A) 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 4.7 Appendix A. L_1 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_1 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 4.4 Appendix A) 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 than D_2 (five percent or less). The additional depth increases the factor of safety against sweepout and decreases the flow velocity.

4.06 **Loose protective work - inverted filter**

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) Downstream Loose Protection

(i) Inverted filter

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. 4.3)

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.

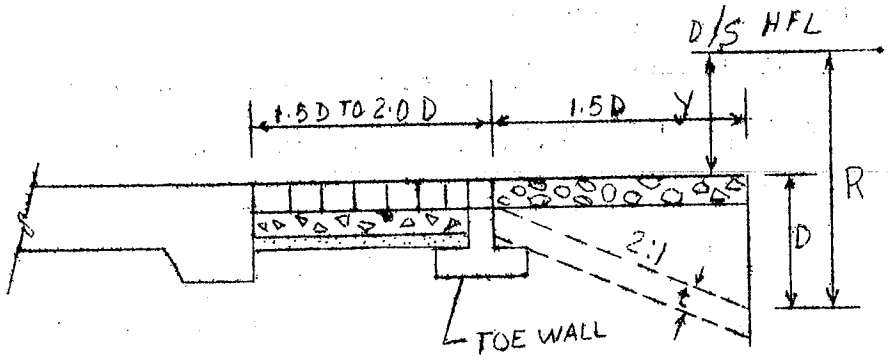


Fig. 4.3 : Details of loose protection.

(b)**Upstream Loose Protections**

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'.

5.00 **Design Example.**

5.01 **Hydraulic Computaions for a Drainage Regulator** (For design data ref. hydrologic design)

= **Stilling basin Design**

Q = 565 cfs

No. of vents = 3

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

Using pier width 1'-3"

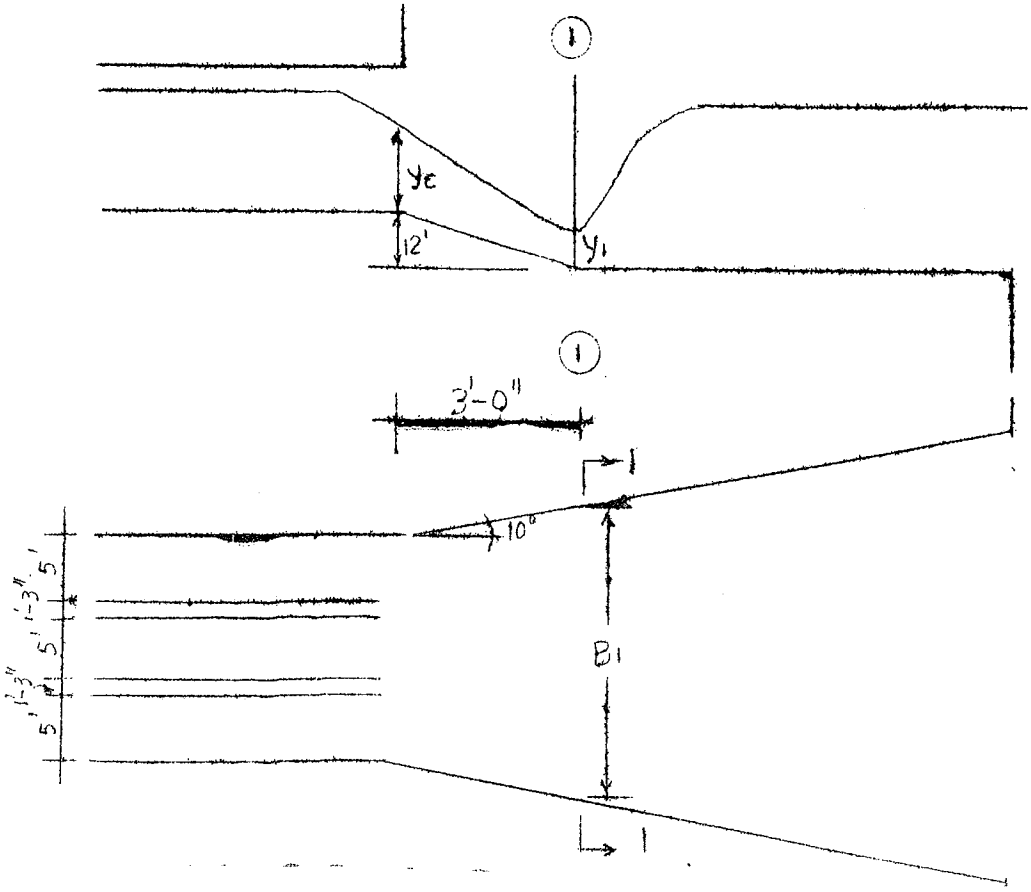
Flow width = 3 x 5 + 2 x 1.75 = 17.5'

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

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

let the apron floor be depressed by 12"

Slope of glacis = 3:1



Flaring angle $= 10^{o}$

$B_1 = 17.5 + 2(3 \tan 10^{o}) = 18.56$

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

$V_1 = q_1/Y_1 = 30.44/Y_1$

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

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

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

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

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

By trial & error

$Y_1 = 1.93$

$V_1 = 30.44/1.93 = 15.78$ fps.

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

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

From curve $\frac{L}{Y_2} = 4.35$ (see fig. 4 Appendix A)

$L = 4.58 \times 4.35 = 19.92$

With factor of safety 1.5

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

- **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

$= \frac{565}{19.14} = 19.39$ 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)

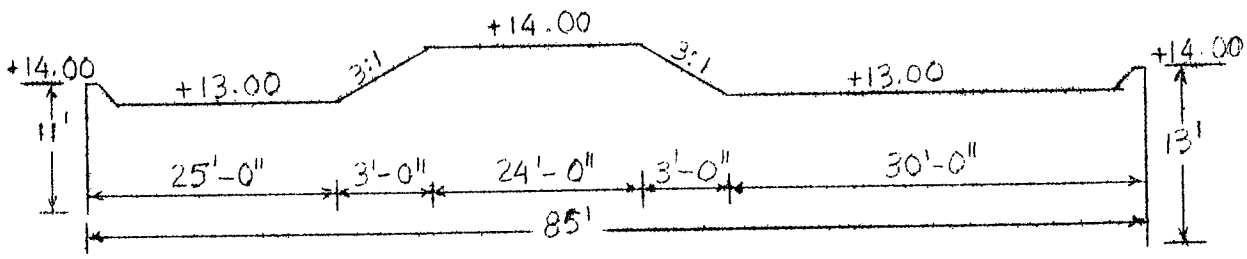
Design Head for designing U/S floor = 7.03' say = 7'-0".

- Computation for exit Gradient.

Maxm seepage head, H = 7'-0"

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

Total floor length = 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.

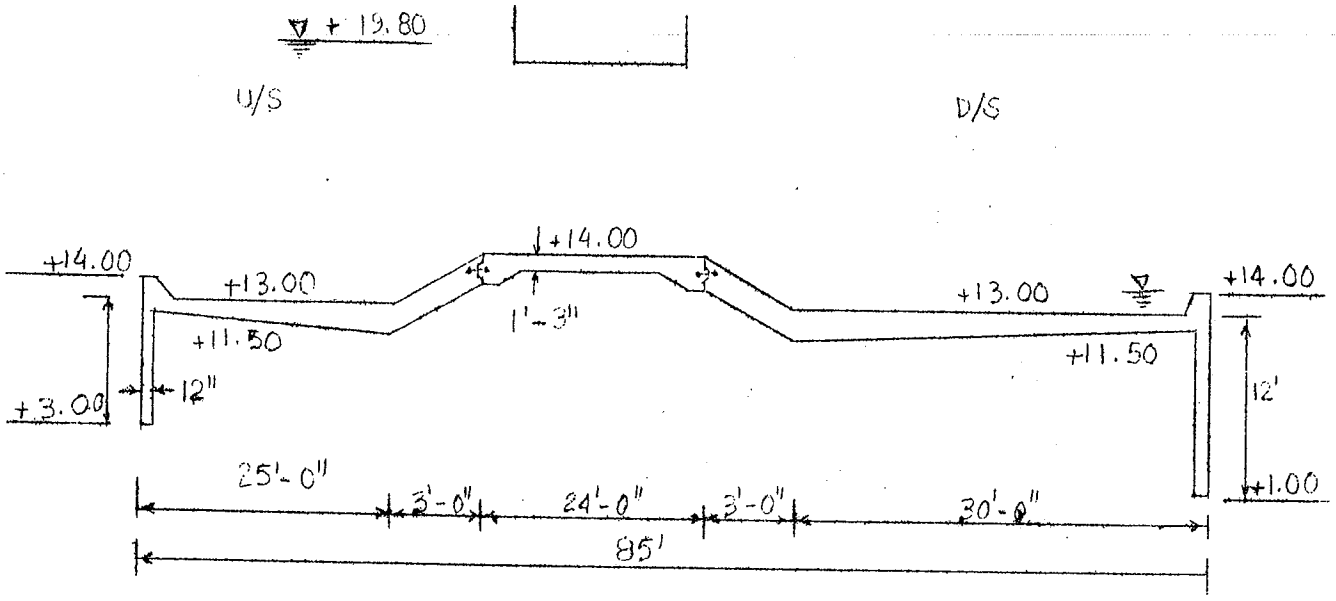
$$(GE) = \frac{H}{d} \times \frac{1}{\pi\sqrt{L}}$$

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

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 4.3 Appendix A)

$PE_1 = 100\%$

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

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

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

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

$D = 11.5 - 3 = 8.5'$

$d = 14.0 - 1.0 = 13.0$

$b' = 83'$

$b = 85'$

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

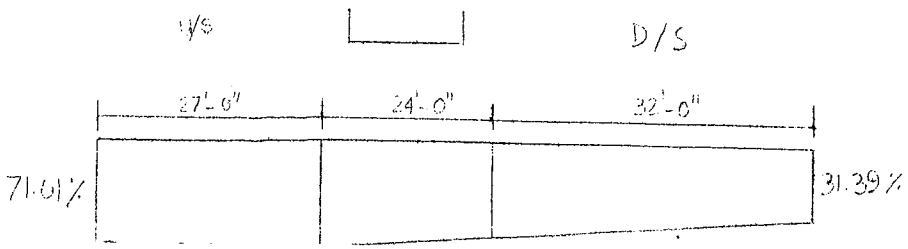
$= 1.36\ \%\ (-ve)$

ii) Correction due to thickness of floor

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

iii) Correction due to slope = nil

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

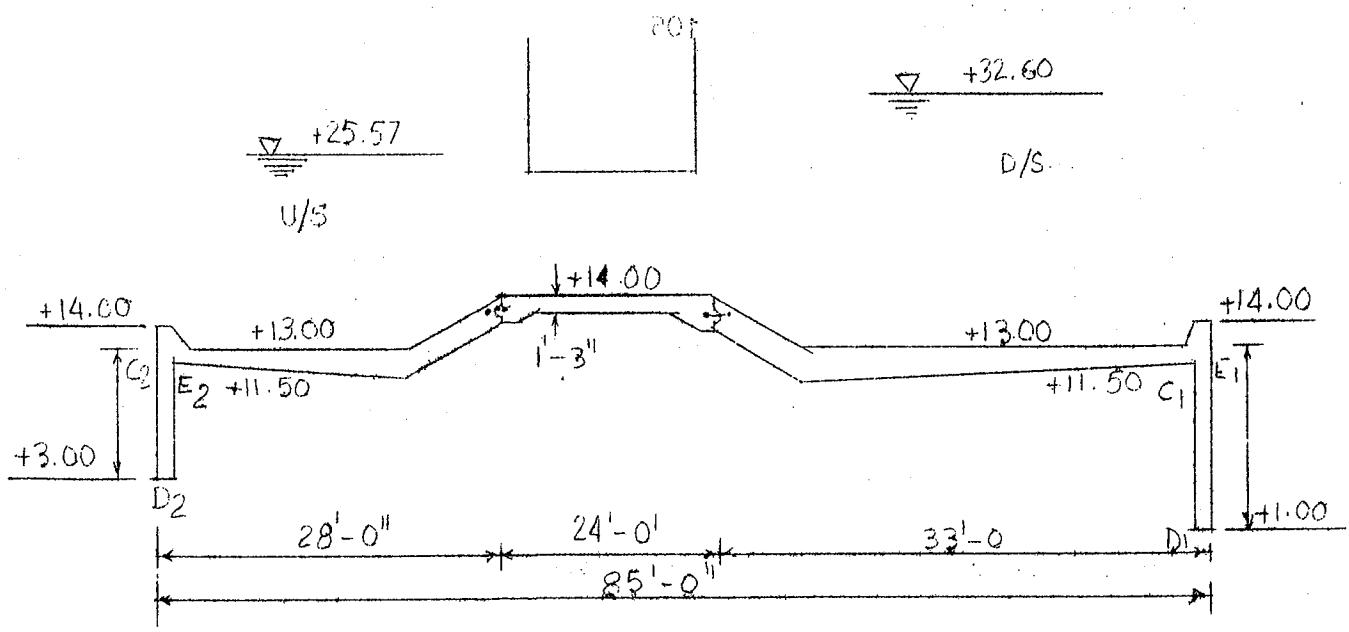
Maxm uplift pressure at sec.2 = 31.39 % of 6'-0" = 1.88' of water

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

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

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

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 4.3 Appendix A)

$PE_1 = 100\ \%$

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

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

Correction for PC_1

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

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

$D = 11.50 - 3 = 8.5'$

$d = 11.5 - 1.0 = 10.5'$

$b' = 83'$

$b = 85'$

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

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

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

Corrected $PC_1 = 66\ \% + 1.36\ \% + 1.5\ \% = 68.86\ \%$

Downstream cutoff wall No. 2.

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

$PC_2 = 0\ \%$

$PE_2 = PE = 32\ \%$

$PD_2 = PD = 22\ \%$

Correction for PE_2

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

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

$D = 11.5 - 1.0 = 10.5'$

$d = 11.5 - 3.0 = 8.5$

$b' = 83'$

$b = 85'$

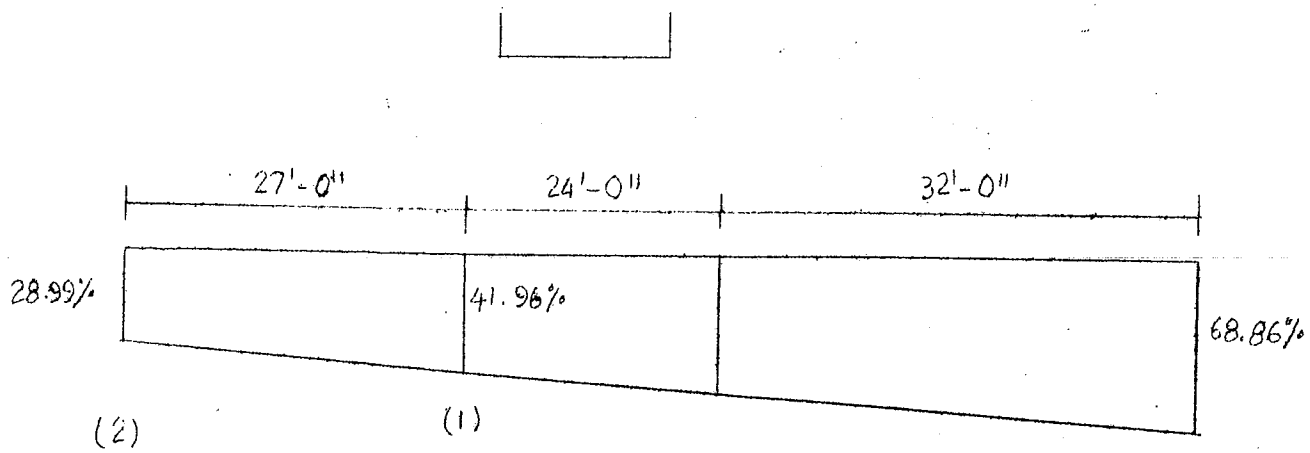
$$\text{Correction} = 19 \left[\frac{10.5}{83}\right]^{1/2} \left[\frac{8.5 + 10.5}{85}\right]^{1/2}$$

$= 1.51\ \%\ (-ve)$

ii) Correction due to thickness of floor

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

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

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

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

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

Design data:

1. Draingage basin area = 5 square miles
2. Maximum R/S W/1 in monsoon season (for average design conditions) = + 9.00 ft PWD
3. Minimum R/S W/1 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 PWD
7. Daily rainfall excess (10 year recurrence interval) = 1.8 inches
8. Crest level of embankment = 16.0 ft PWD

5.03 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).

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.

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

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.

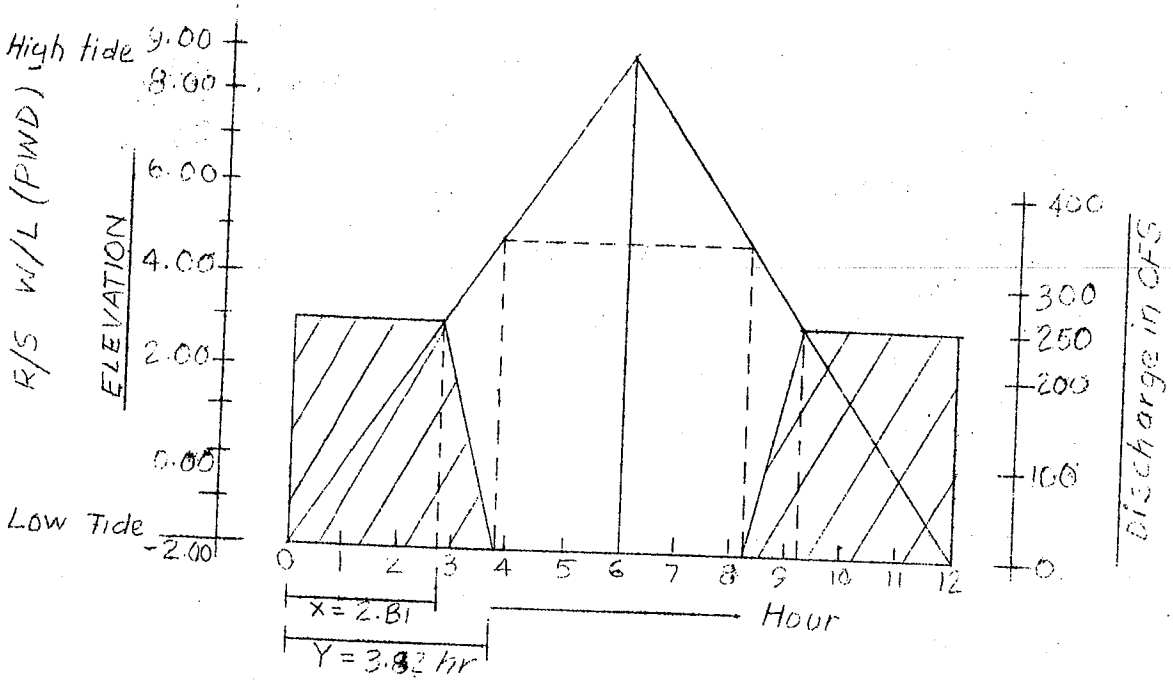


Figure 5.2 Tide and Discharge Curve

Drainage is constant, by similar trainagles, for

x = 6/11 x 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'

5.04 **Design of outlet basin**

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 x 15 x tan 18.43^{o} = 24 ft.

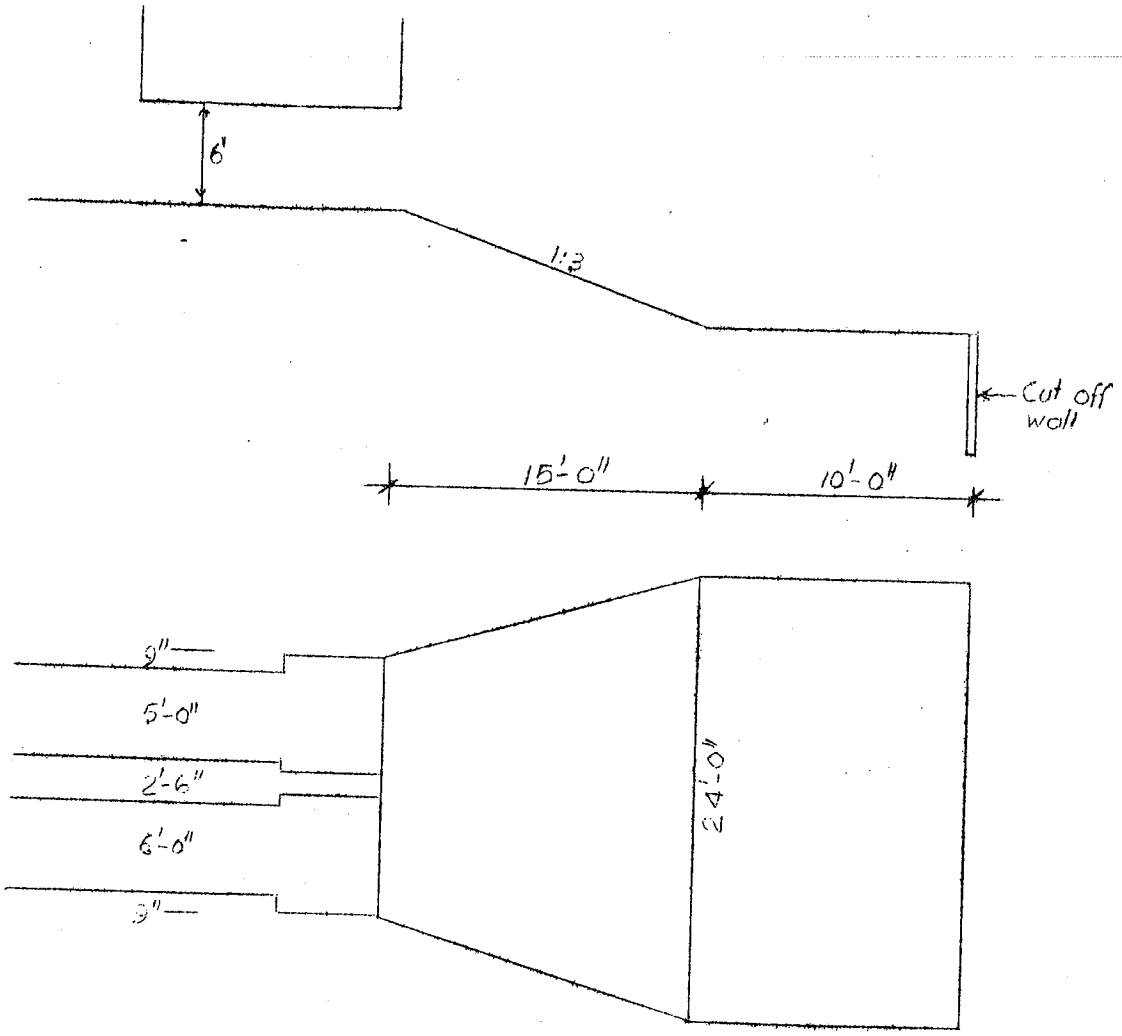


Fig. 5.3 : Plan and Section of Sluice Outlet Basin.

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 7 in the Appendix) the tailwater elevation required (D_2) for formation of jump is given by

$$D_2 = d_2 (1.1 - F_1^2/120)$$

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

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

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

$L_B = 7.4$ ft 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.

5.05 **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 2500 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 feet, 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.

REFERENCES :

1. Design of Small Dams, U.S. Department of Interior, U.S.A 1977

2. Irrigation Engineering and Hydraulic Structures - Santash Kumar Garg.

3. "Hydraulic Charts for the selection of Highway Culverts" Hydraulic Engineering Circular No.5 U.S Department of Commerce, Washington D.C. U.S.A 1965.

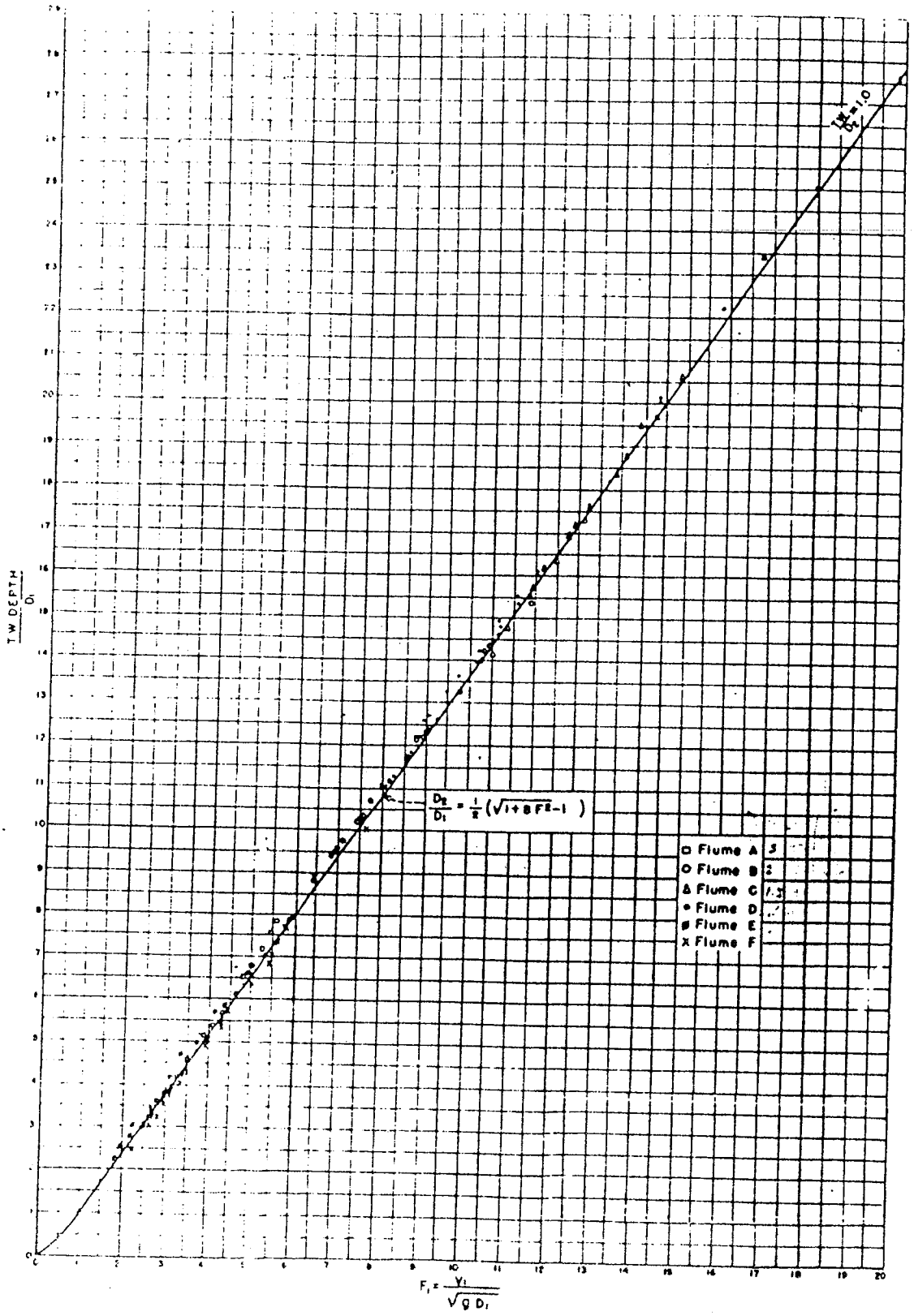
4. Leliavsky, S. Design Text Books in Civil Engineering Vol. Five, Wiers, Champman and Hall Ltd. Great Britain, 1965.

5. Handerson, F.M. Open Channel Flow Macmillan Publishing Co. Inc. Newyork, N.Y. U.S.A, 1966

6. "Design Charts for Open Channel Flow" Hydraulic Design Series No.3, U.S. Department of Commerce, Washington D.C. U.S.A.

APPENDIX A

DESIGN CHARTS AND FIGURES



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

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

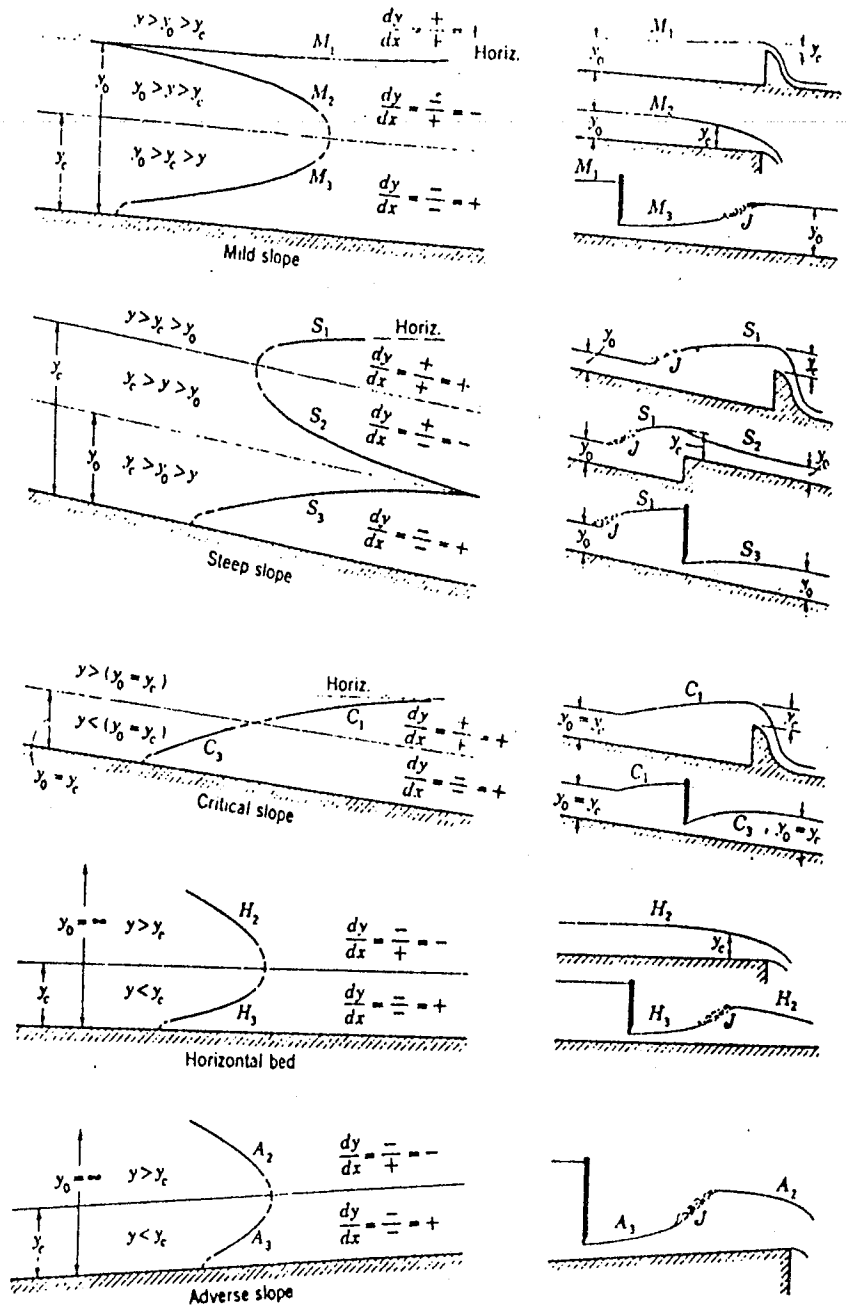


Figure: 2. Classification of Flow Profiles of Gradually Varied Flow

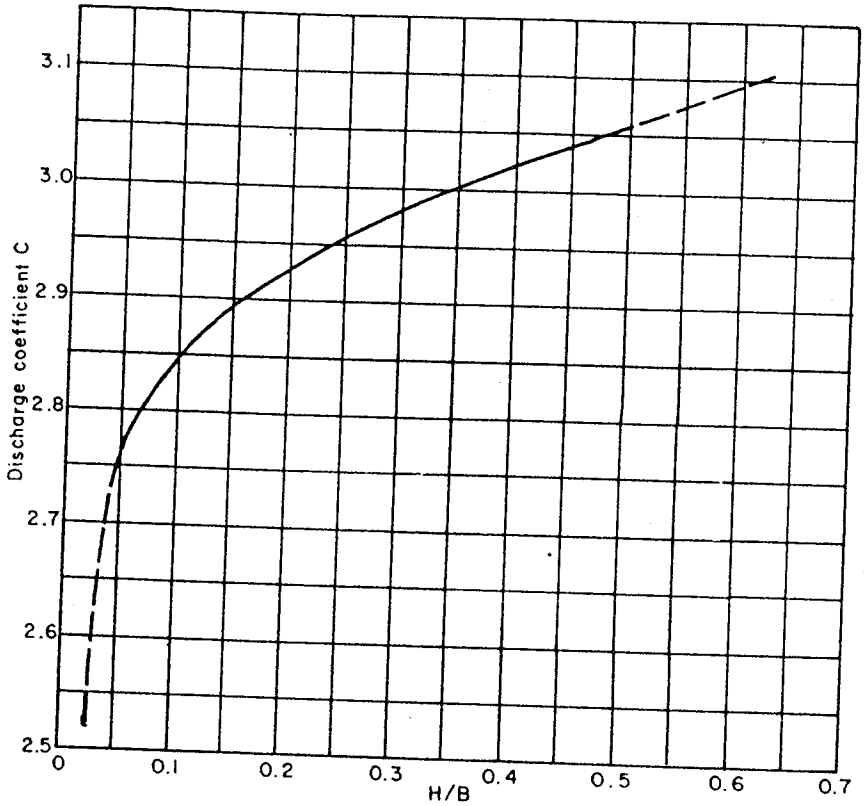
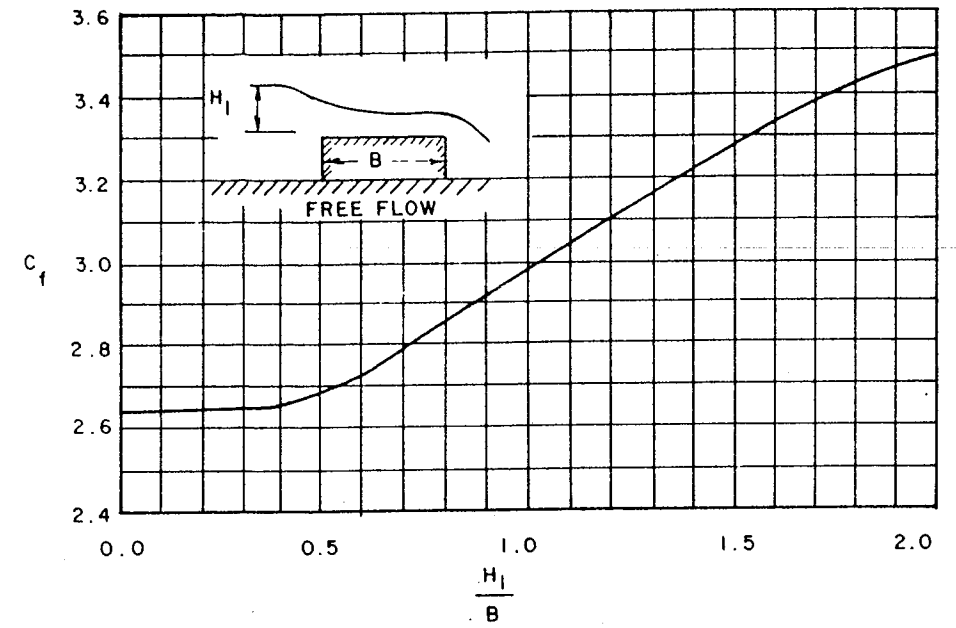
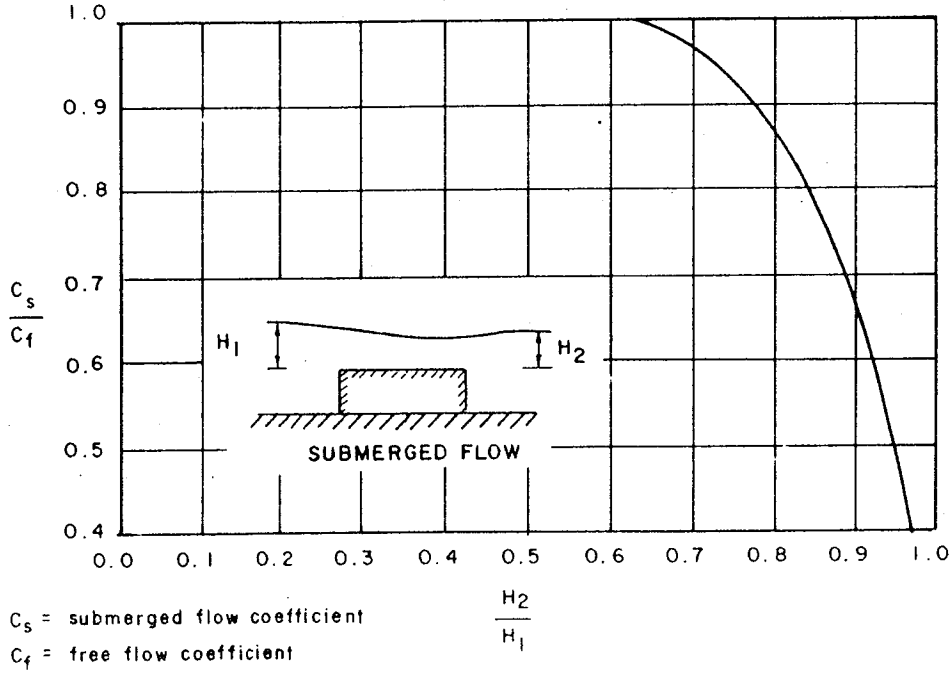


Figure 3. Coefficient of Discharge for Broad-Crested Weirs with Rounded Upstream Edge.





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

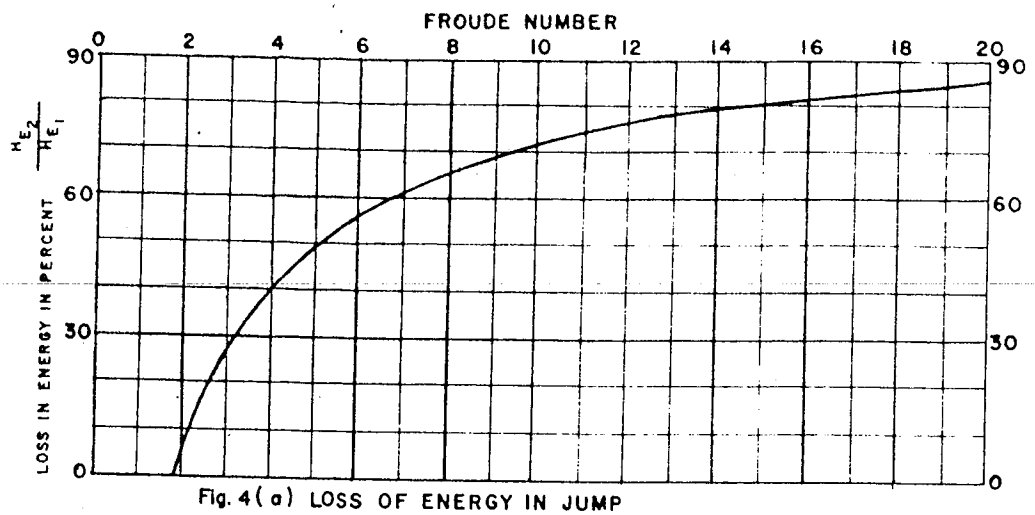


Fig. 4 (a) LOSS OF ENERGY IN JUMP

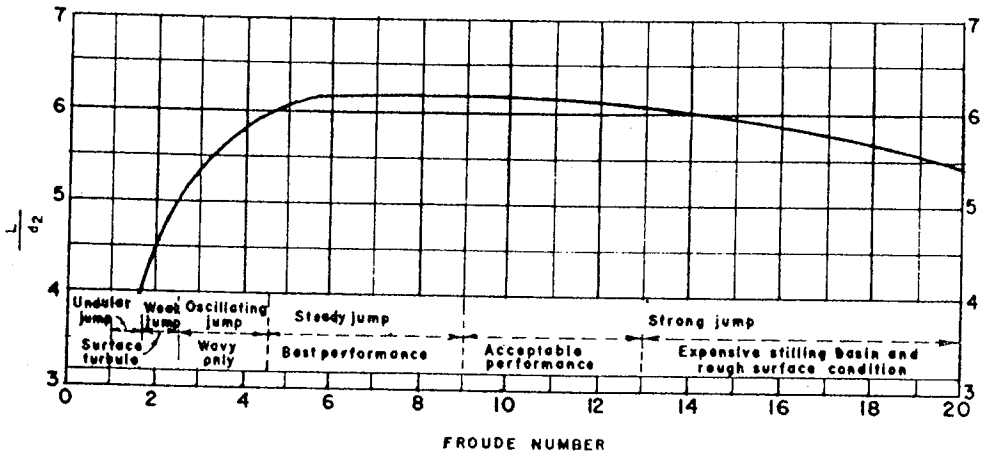
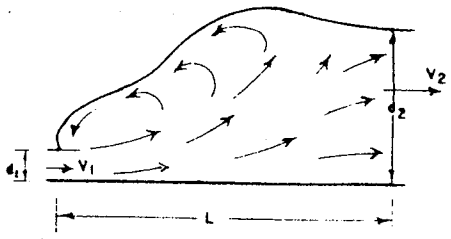


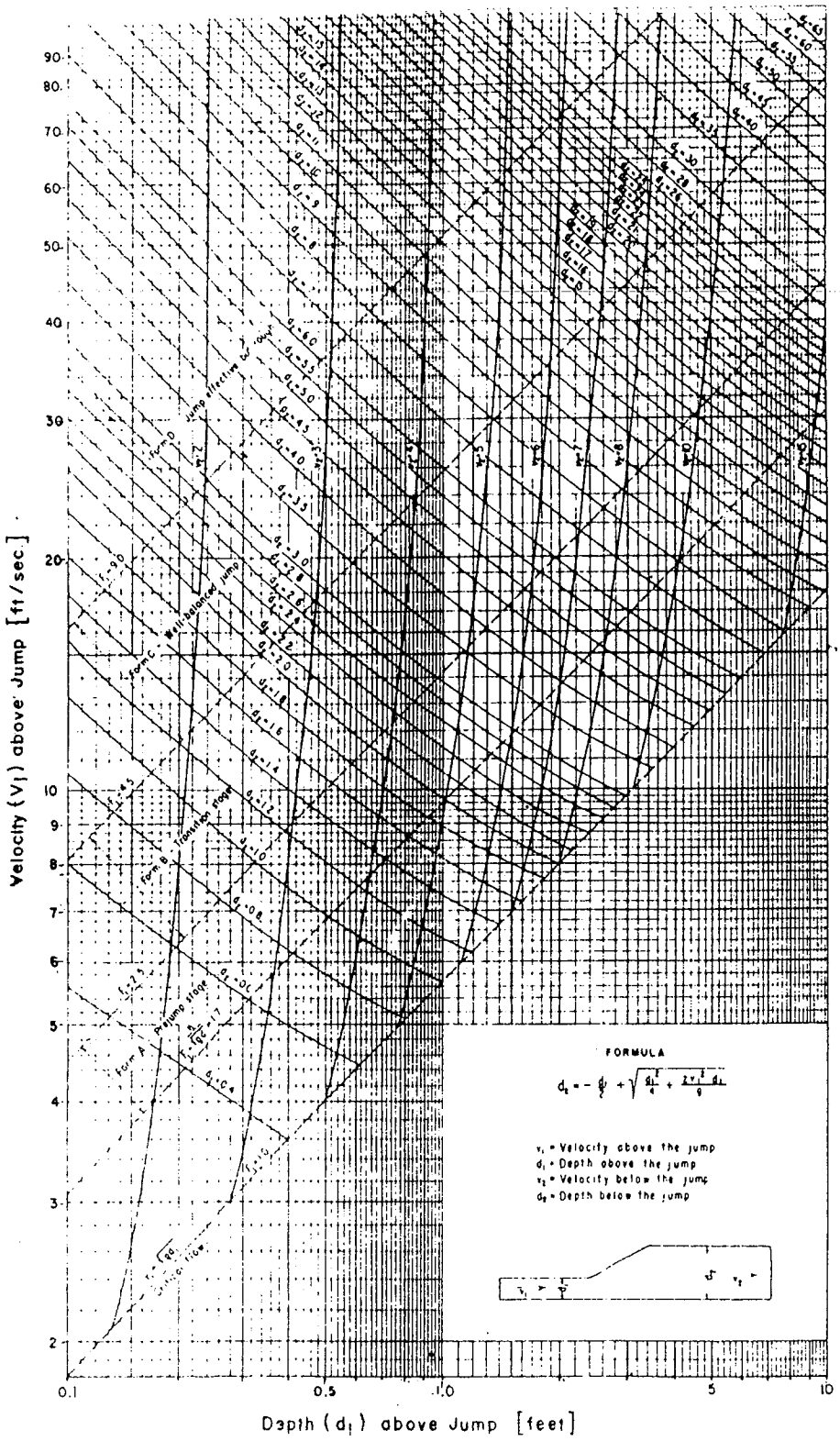
Fig. 4 (b) LENGTH OF JUMP



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 4. Properties of the Hydraulic Jump in Relation to the Froude Number.



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

Figure: 4.1 Relations Between Variables in Hydraulic Jump for Rectangular Channel

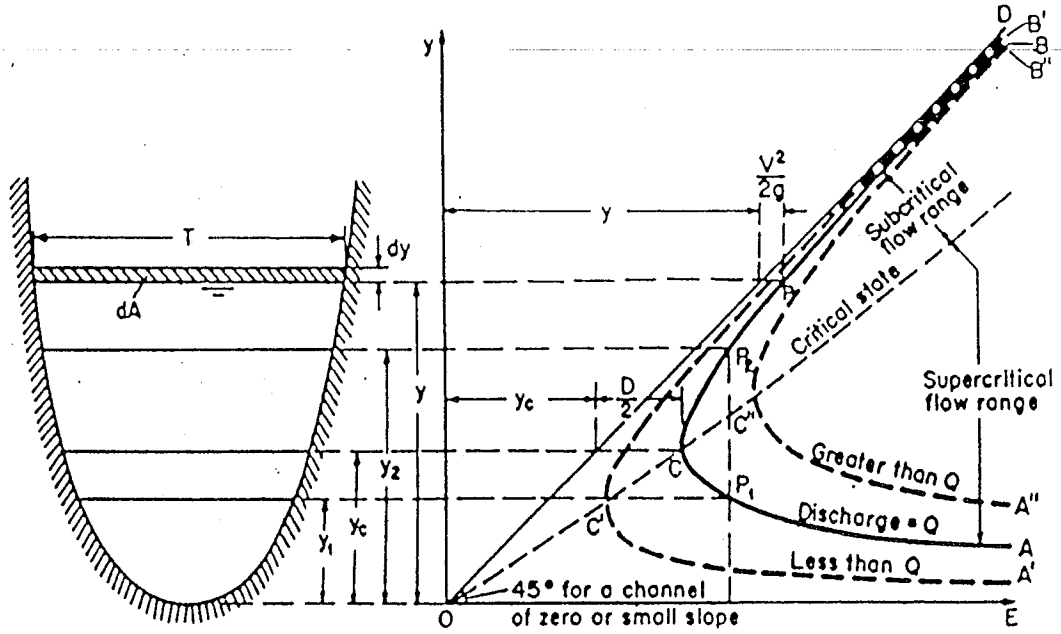


FIG. 4·2 Specific-energy curve.

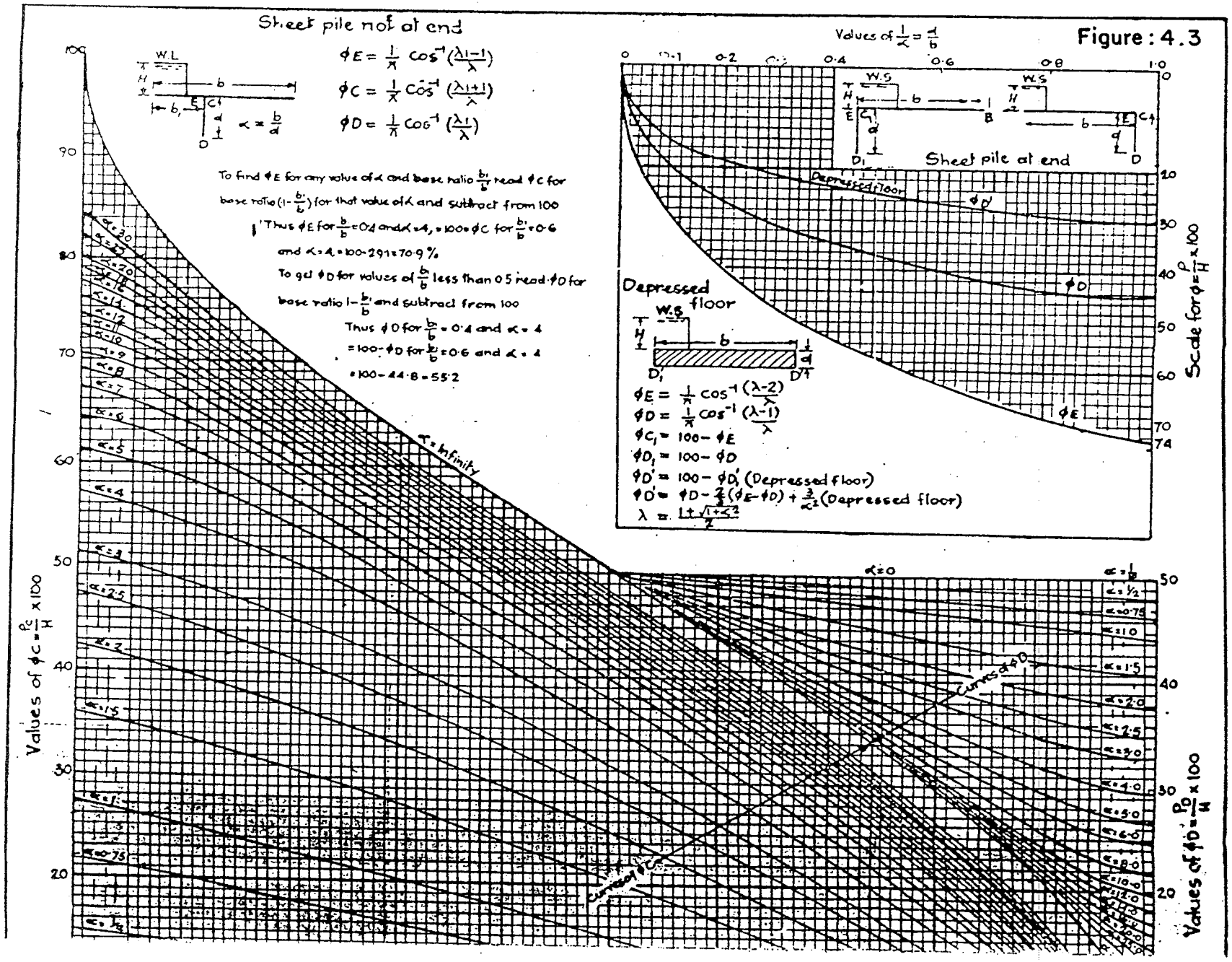


Figure : 4.3
Sheet pile not at end
$\phi E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$
$\phi C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$
$\phi D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$
$\alpha = \frac{b}{d}$
To find ϕE for any 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

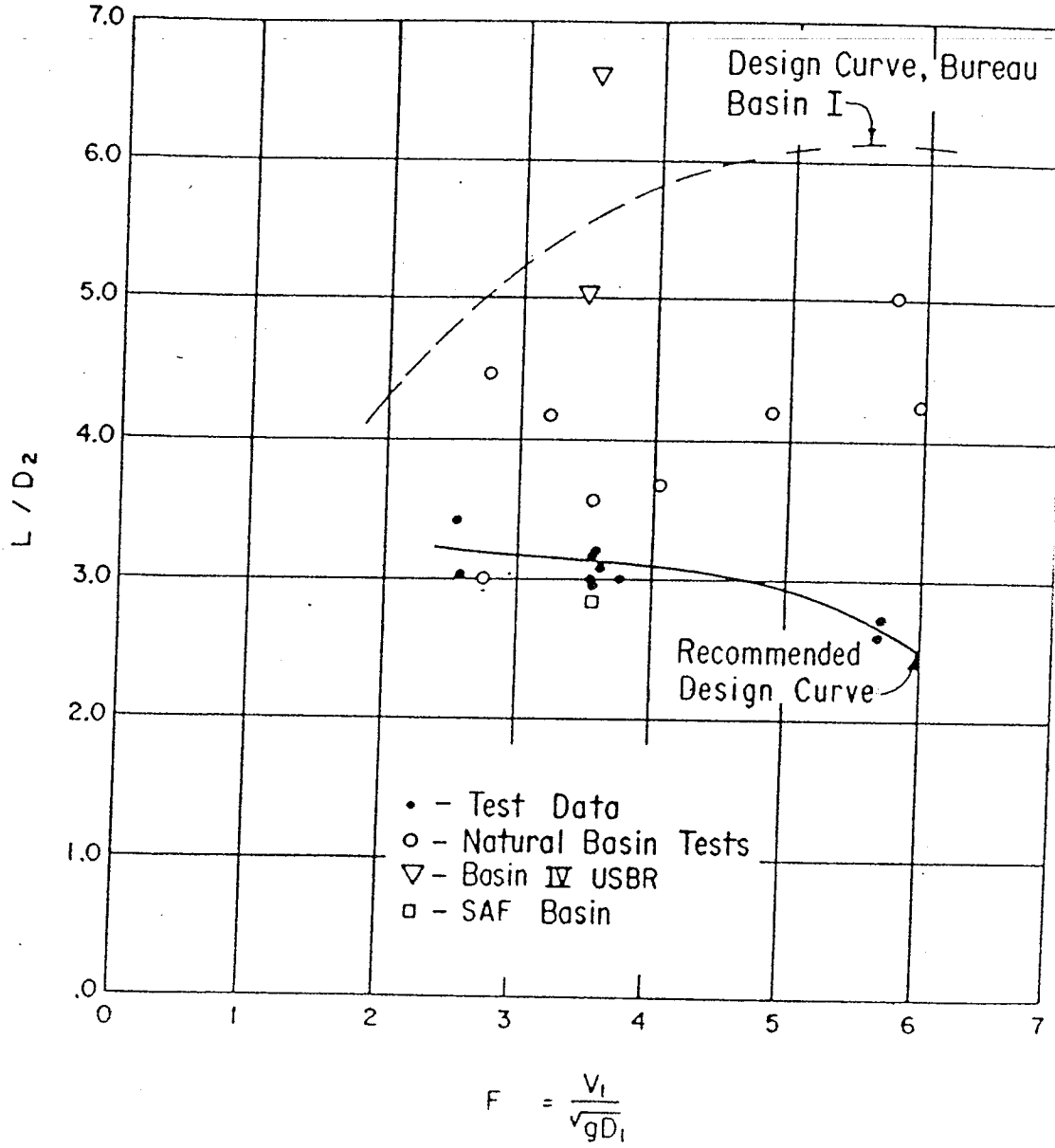


Figure 4.4 Dimensionless length of stilling basin.

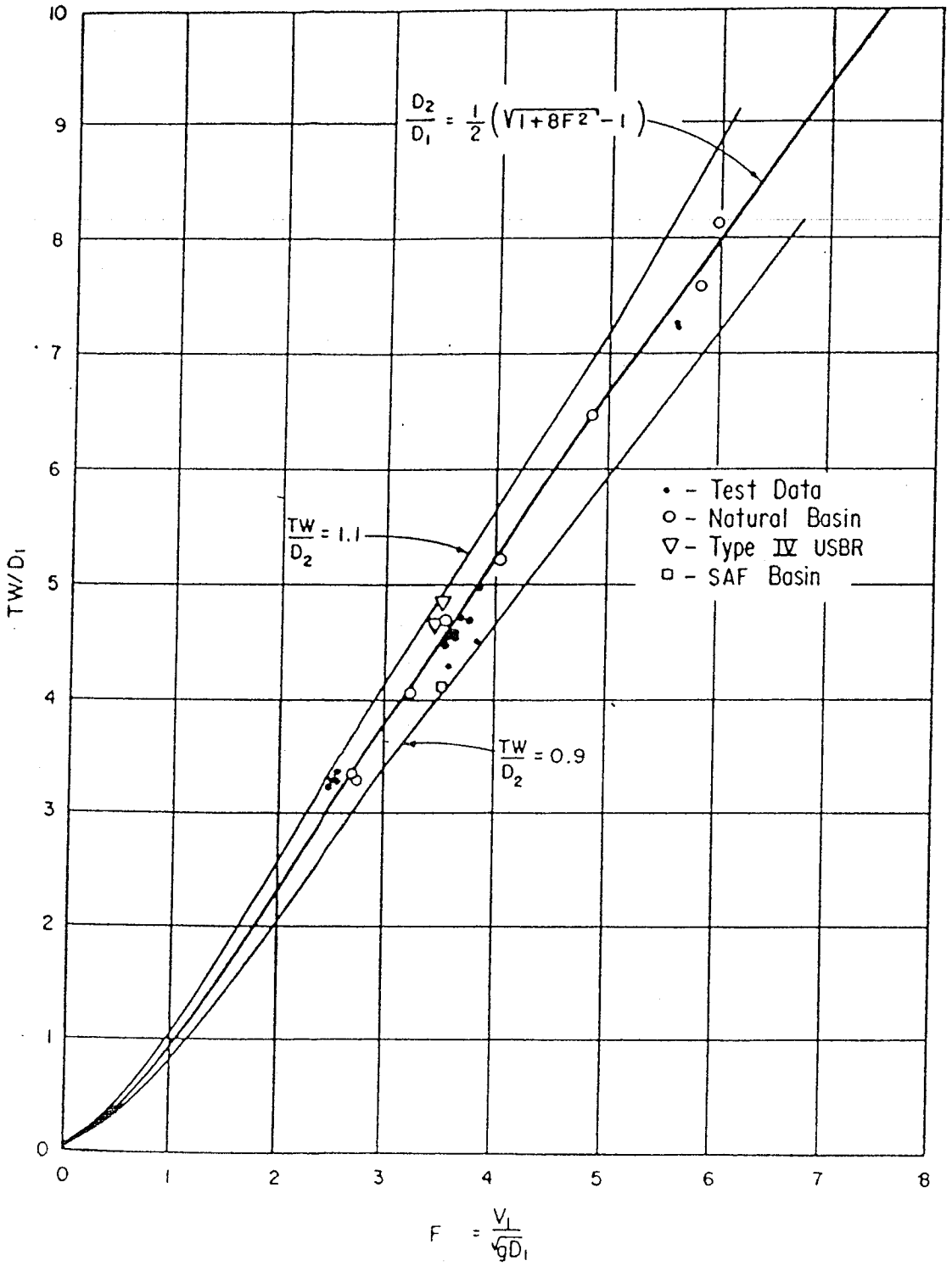
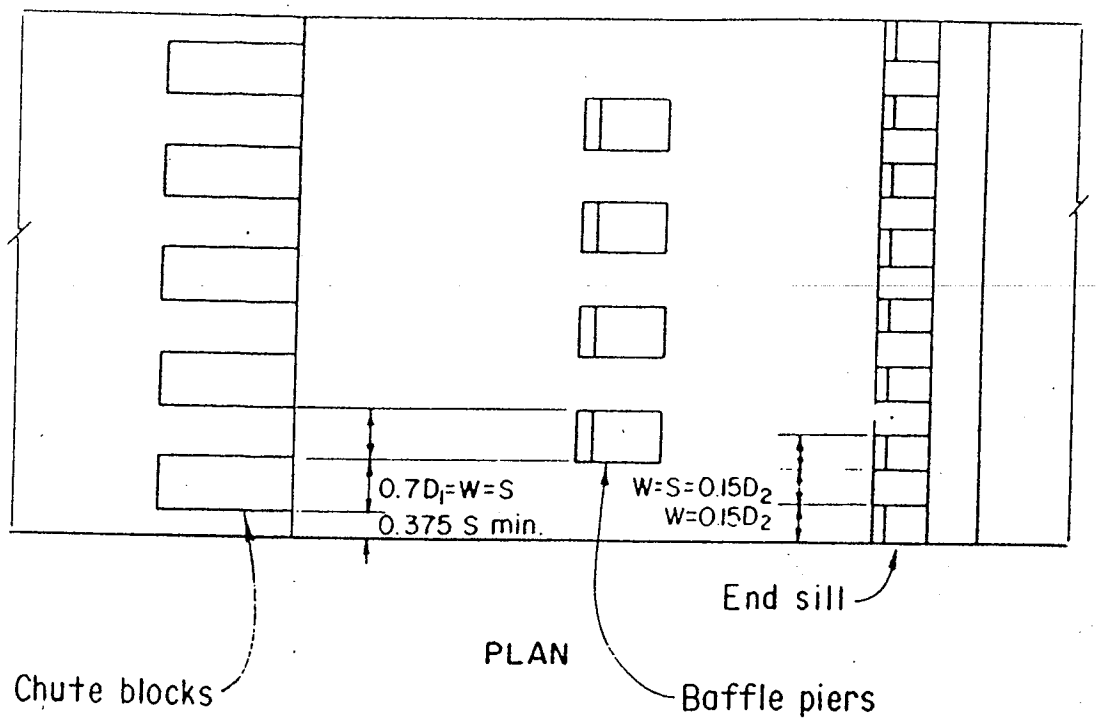
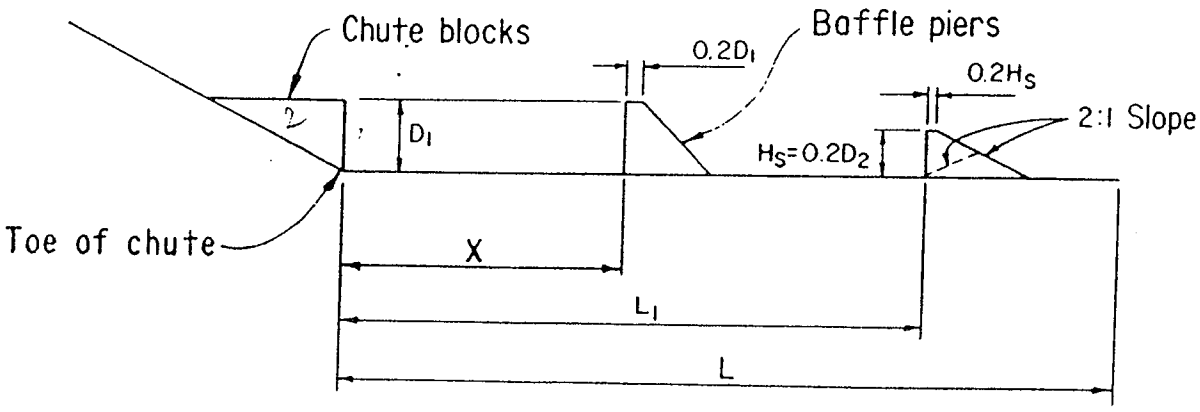


Figure 4.5-Ratio of tailwater to inflow depth.





ELEVATION

Figure 4.6-Recommended stilling basin ($2.5 \leq F \leq 5.0$).

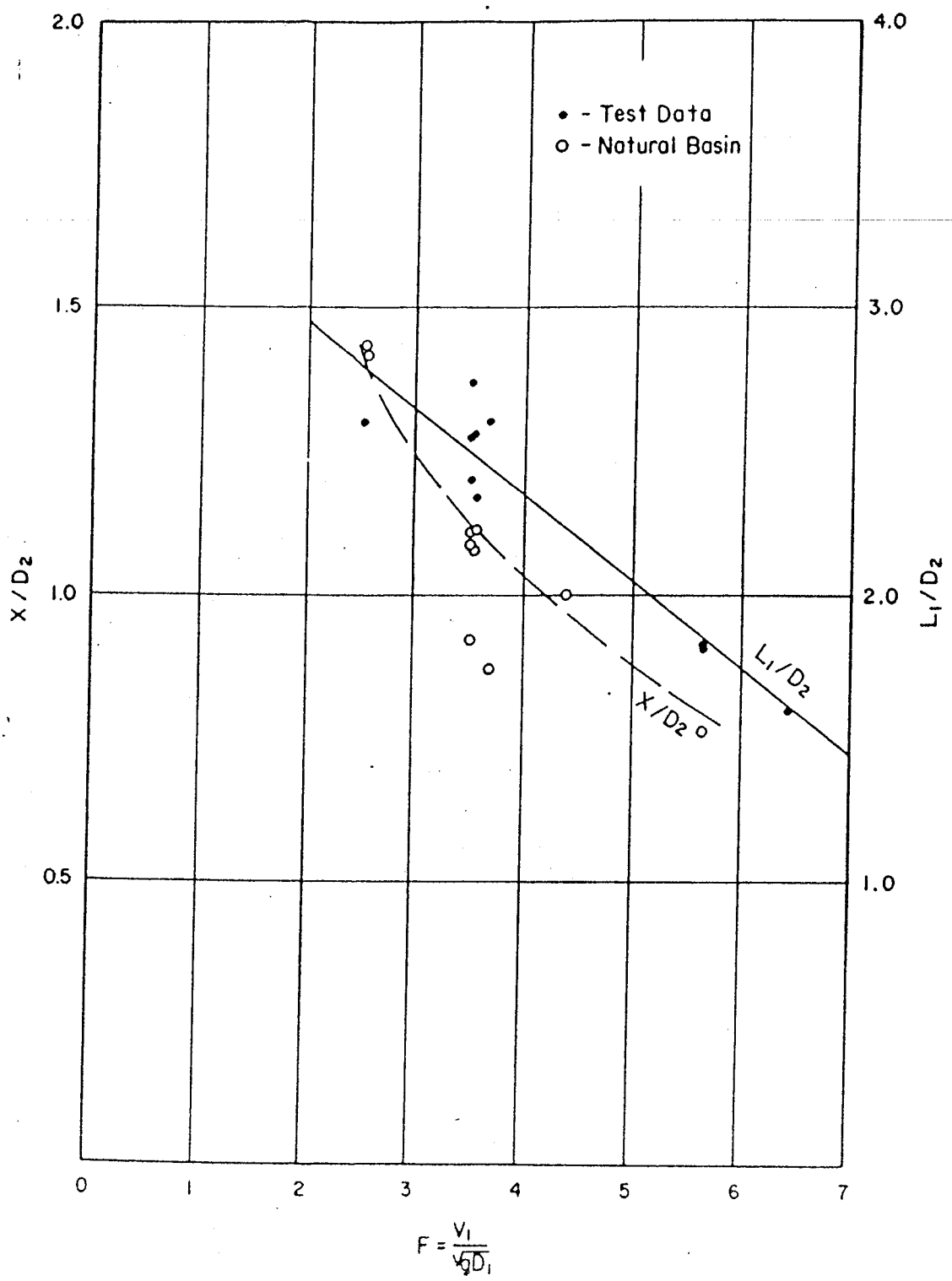
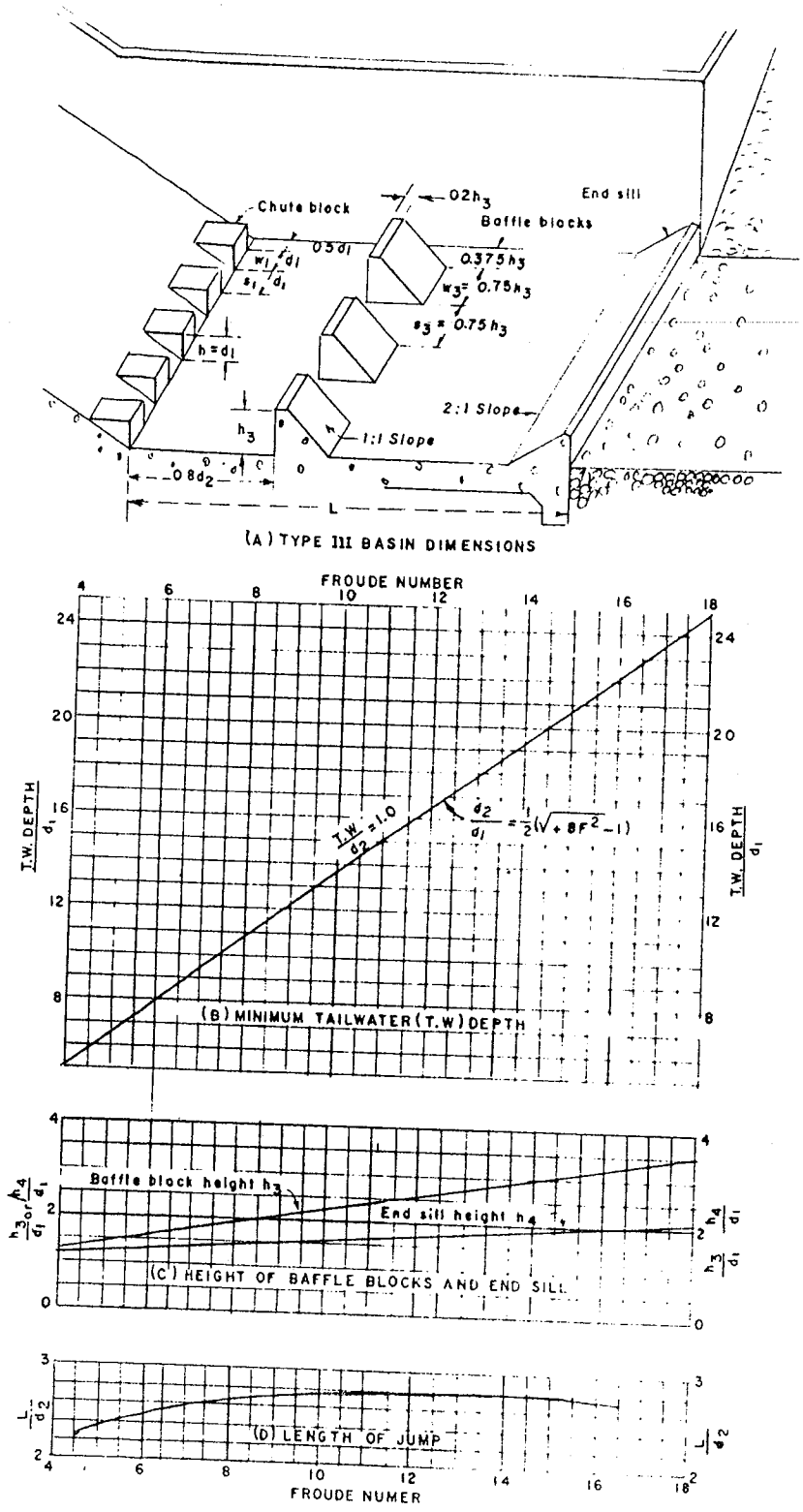
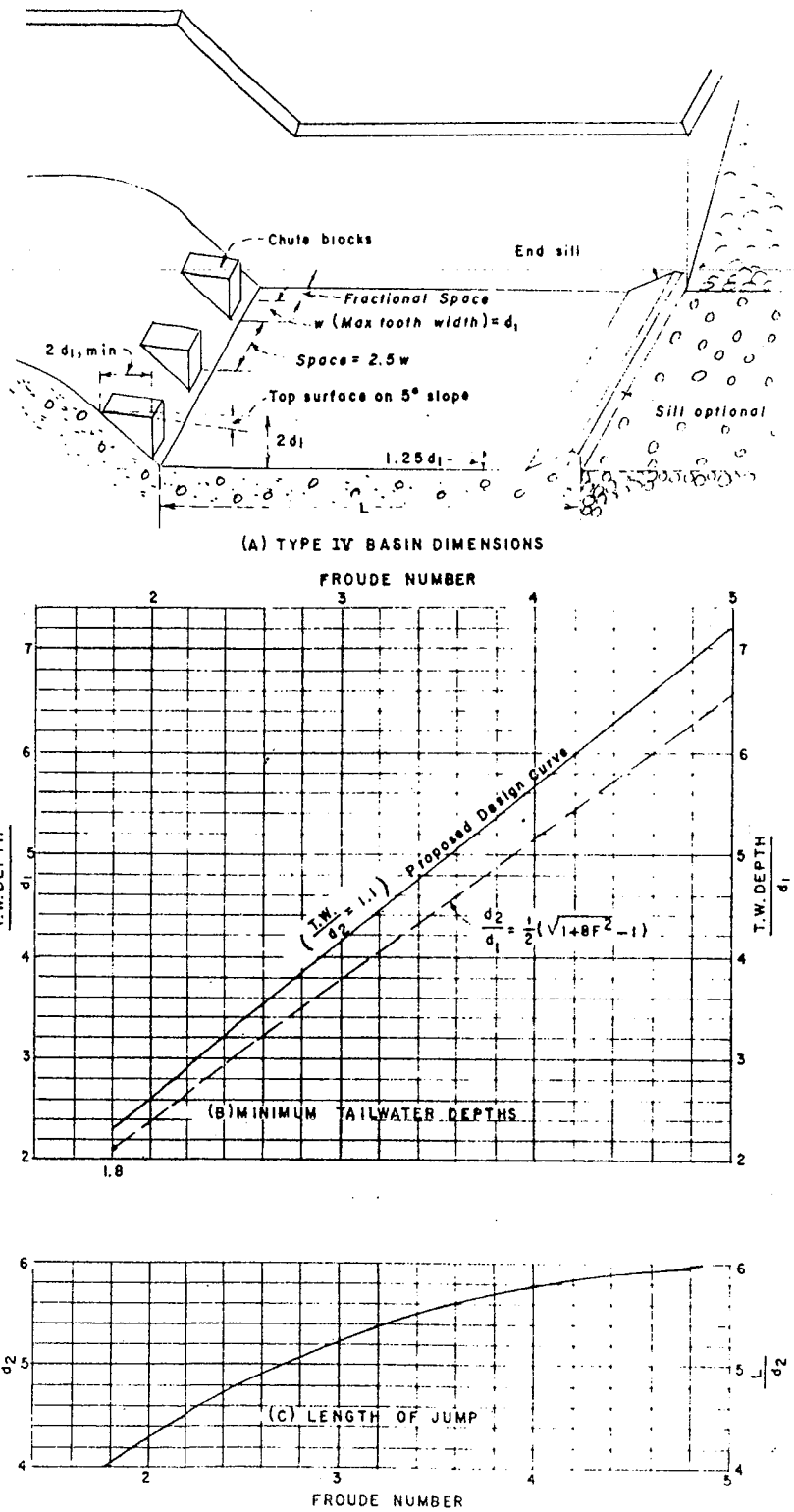


Figure 4.7 - Distance from toe of chute to baffle piers (X) and end sill (L)



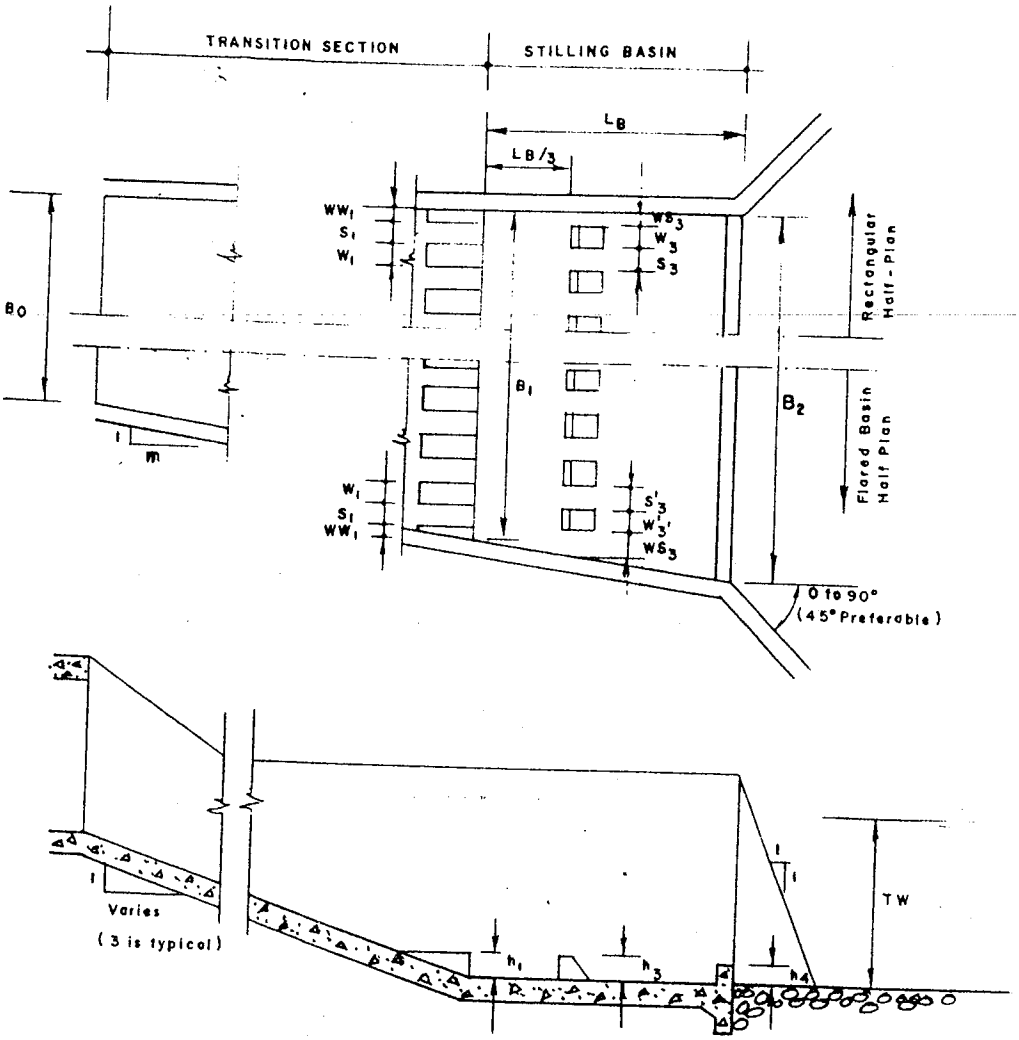
Source: "Design of Small Dams" by U.S. Department of the Interior

Figure 5. USBR Hydraulic Jump Stilling Basin Type III.



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

Figure 6. 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 7. SAF Hydraulic Jump Stilling Basin.

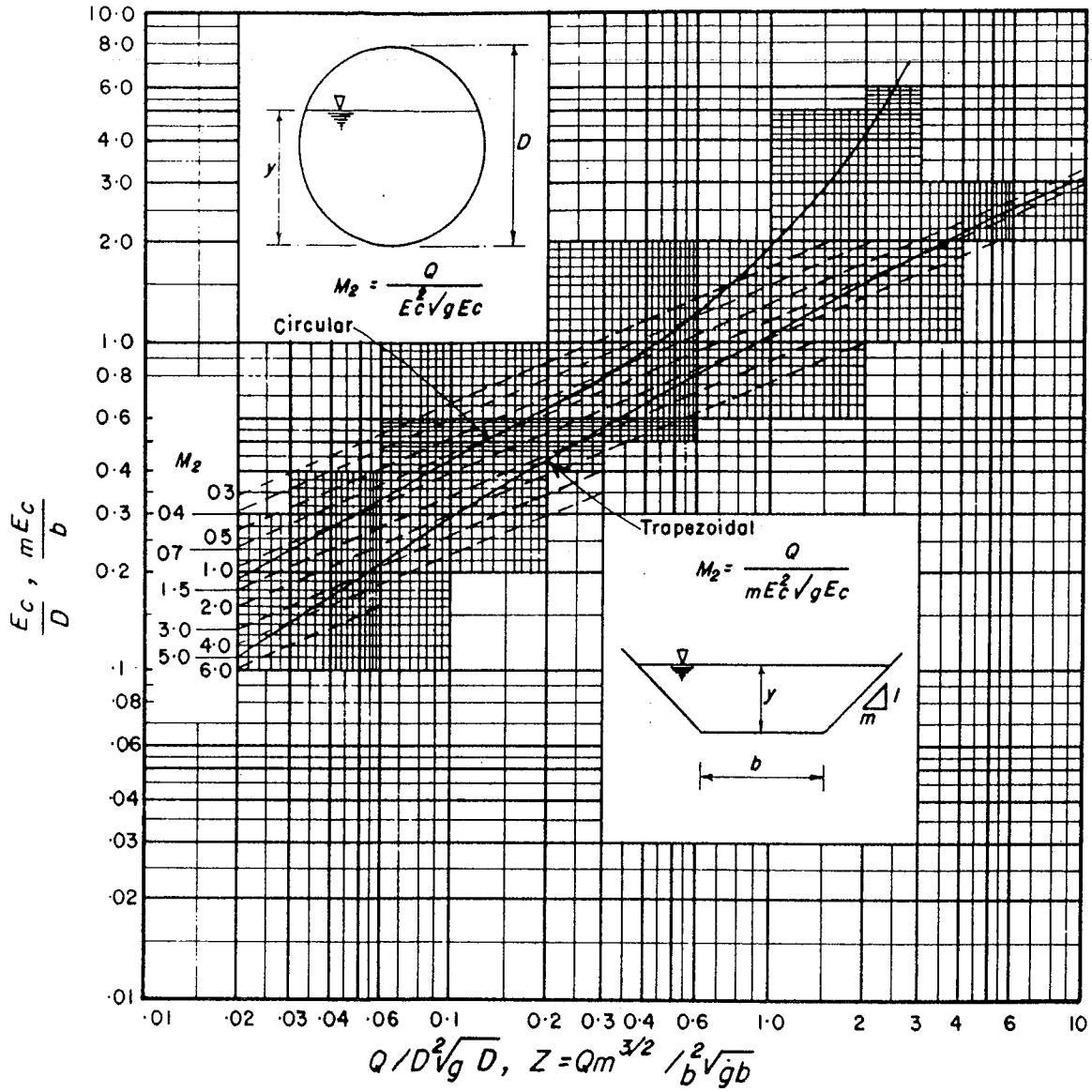


FIGURE 8 DIMENSIONLESS REPRESENTATION OF CRITICAL SPECIFIC ENERGY FOR CIRCULAR AND TRAPEZOIDAL CHANNEL SECTIONS

Source: Henderson, F.M., Open Channel Flow, Macmillan Publishing Co., Inc., New York, U.S.A., 1966, P. 55

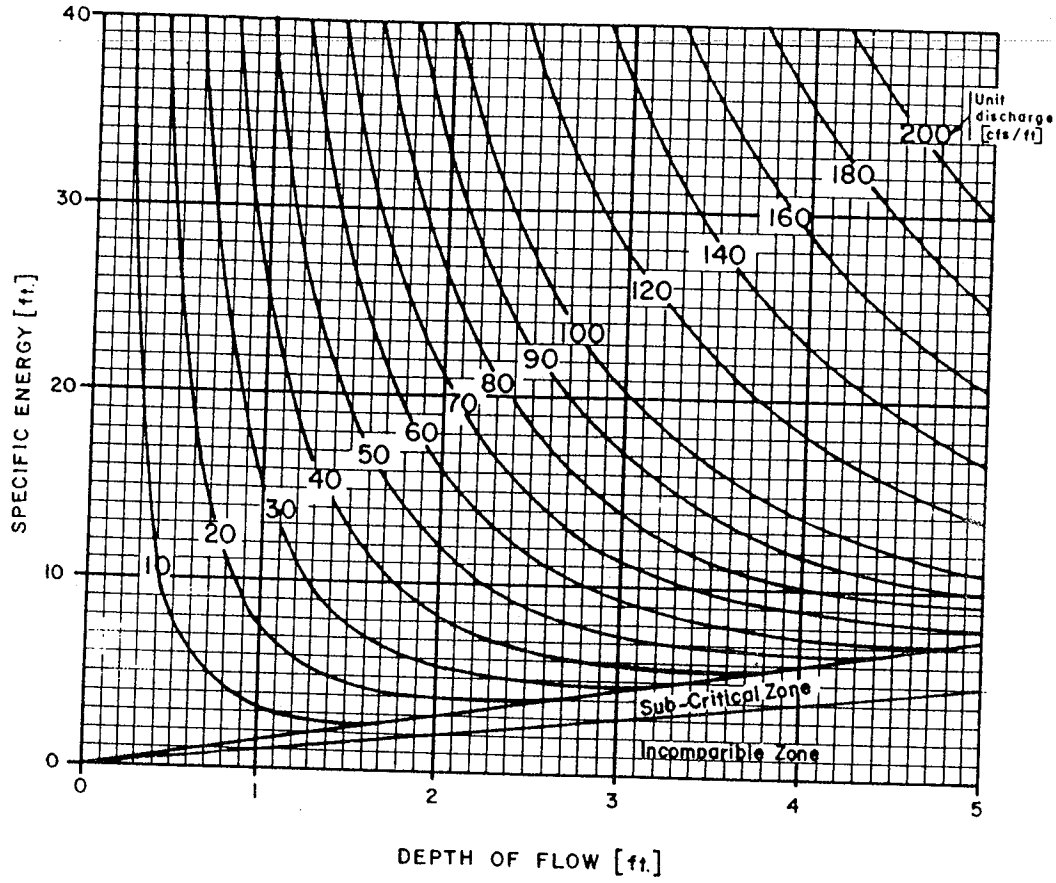


Figure 9. Depth versus Specific Energy and Unit Discharge for Super-Critical Flow

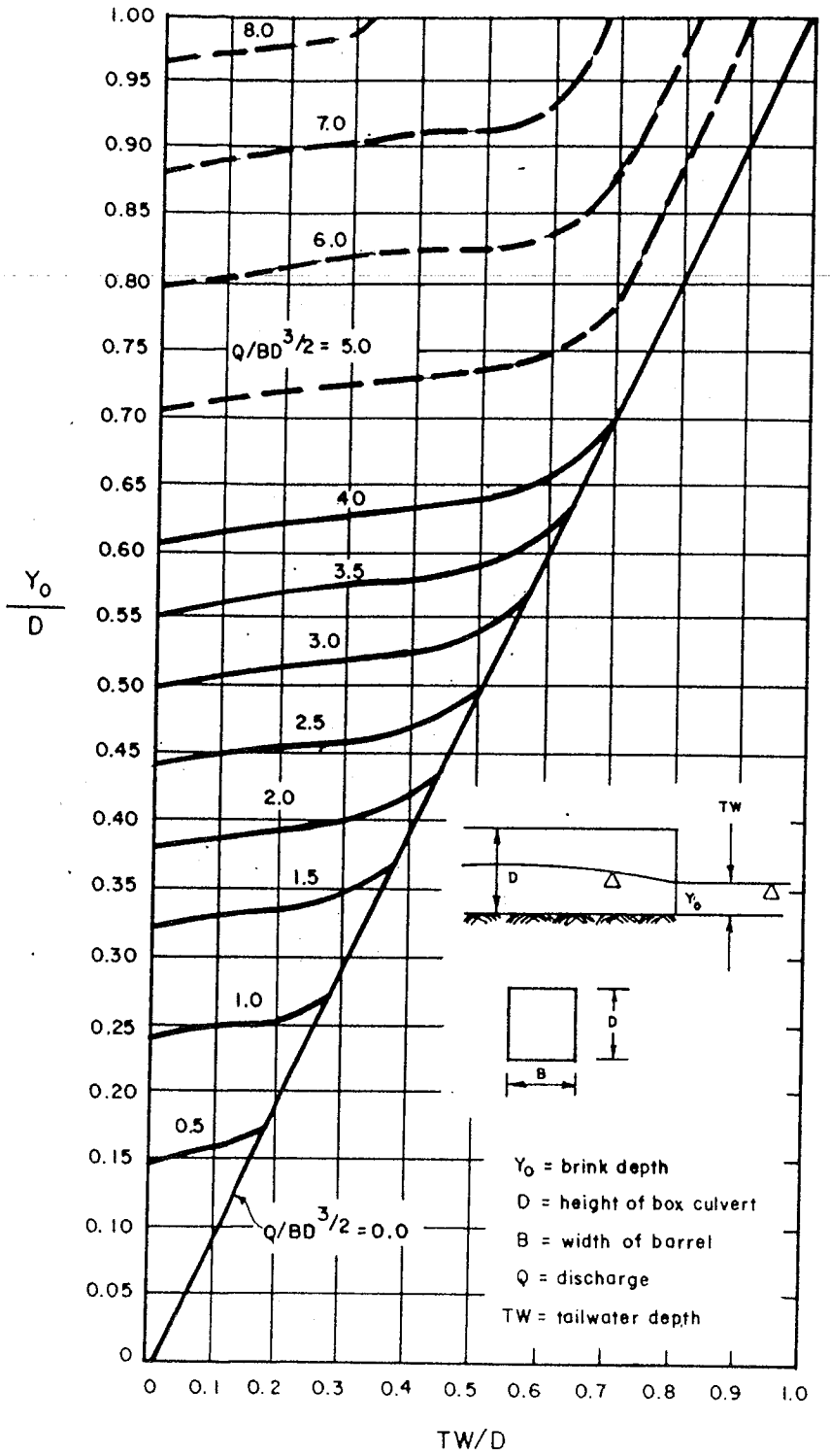
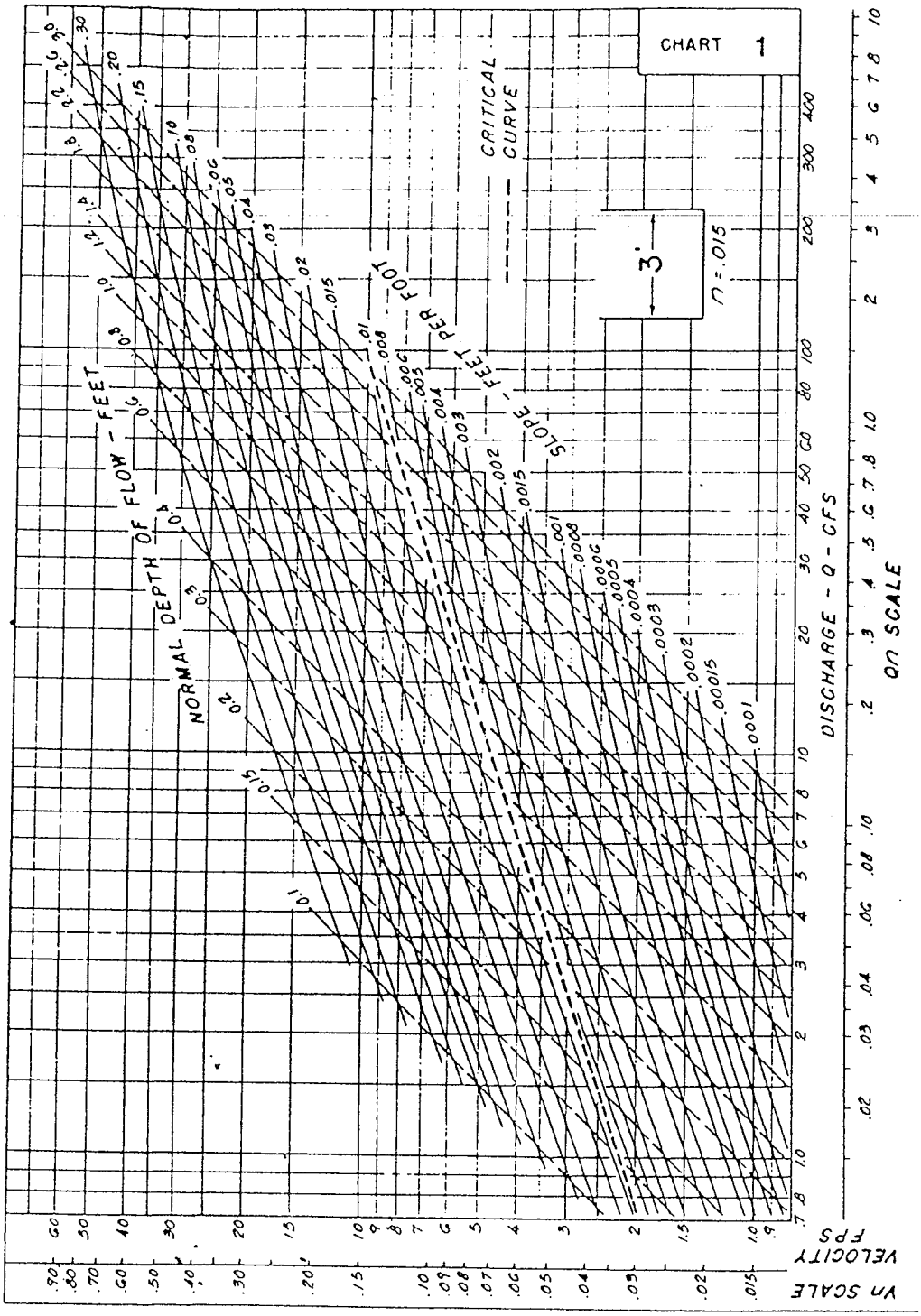
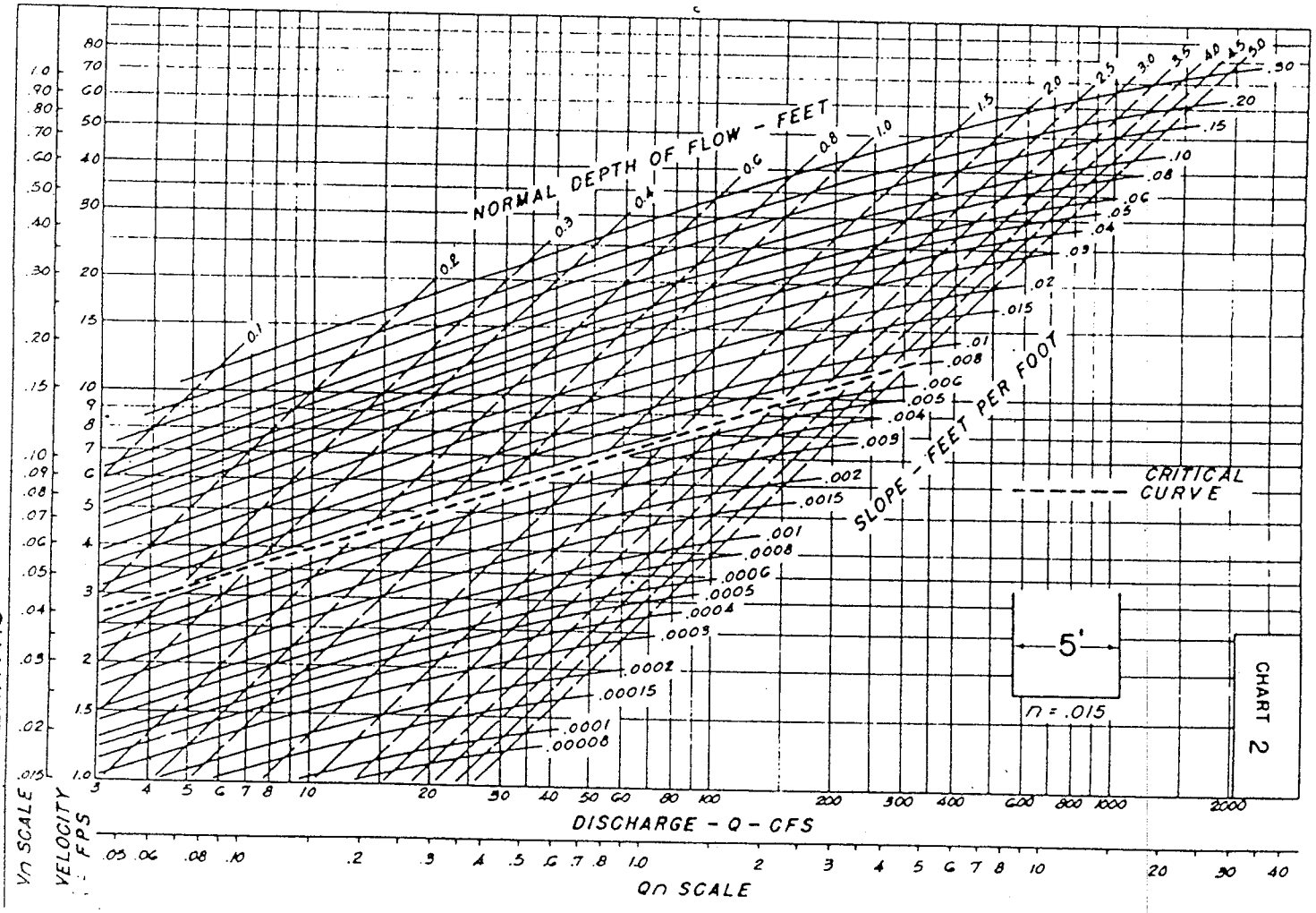


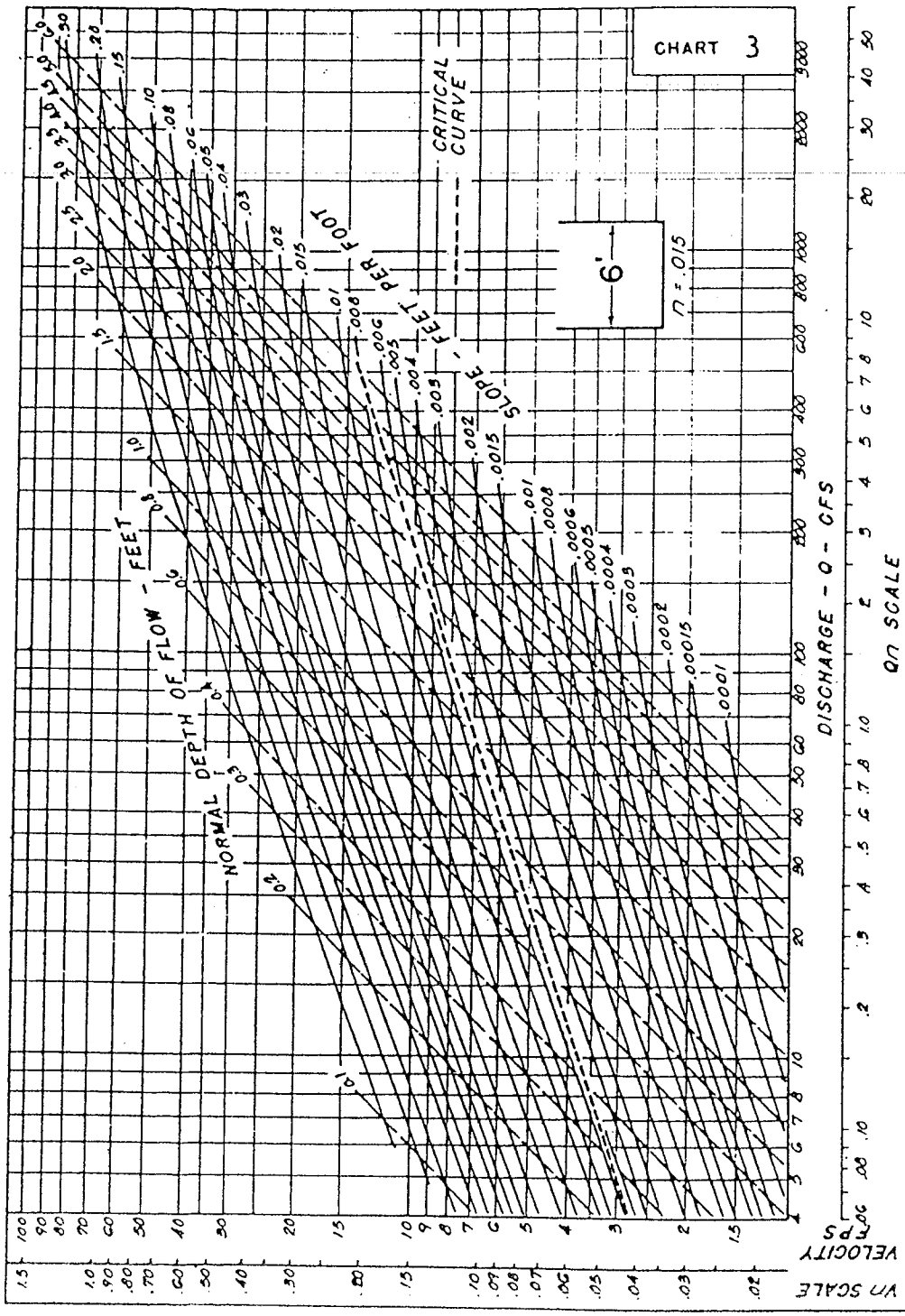
Figure 10. Dimensionless Rating Curves for the Outlets of Regulator Culverts on Horizontal and Mild Slopes



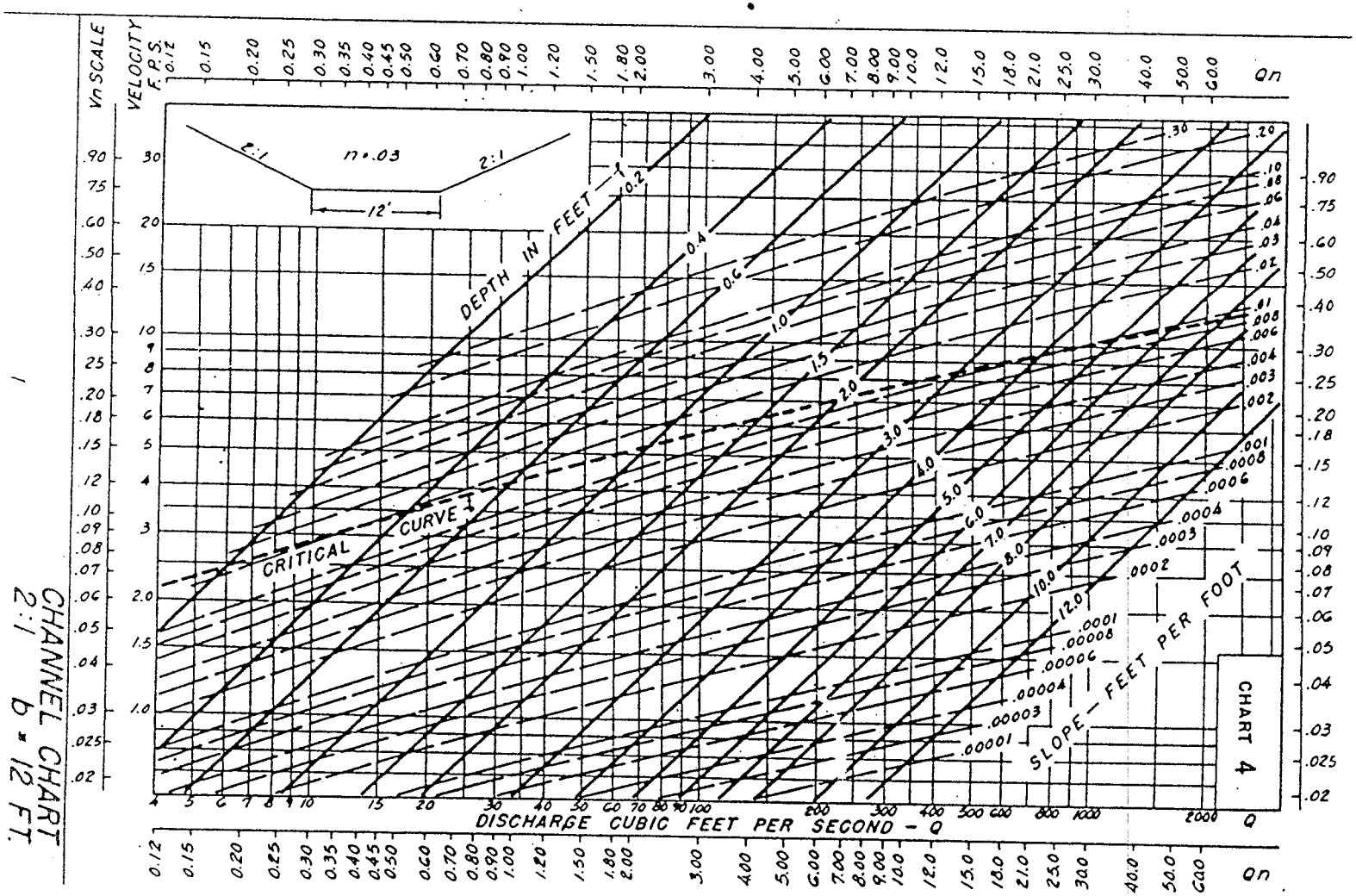
CHANNEL CHART
VERTICAL b = 3 FT.

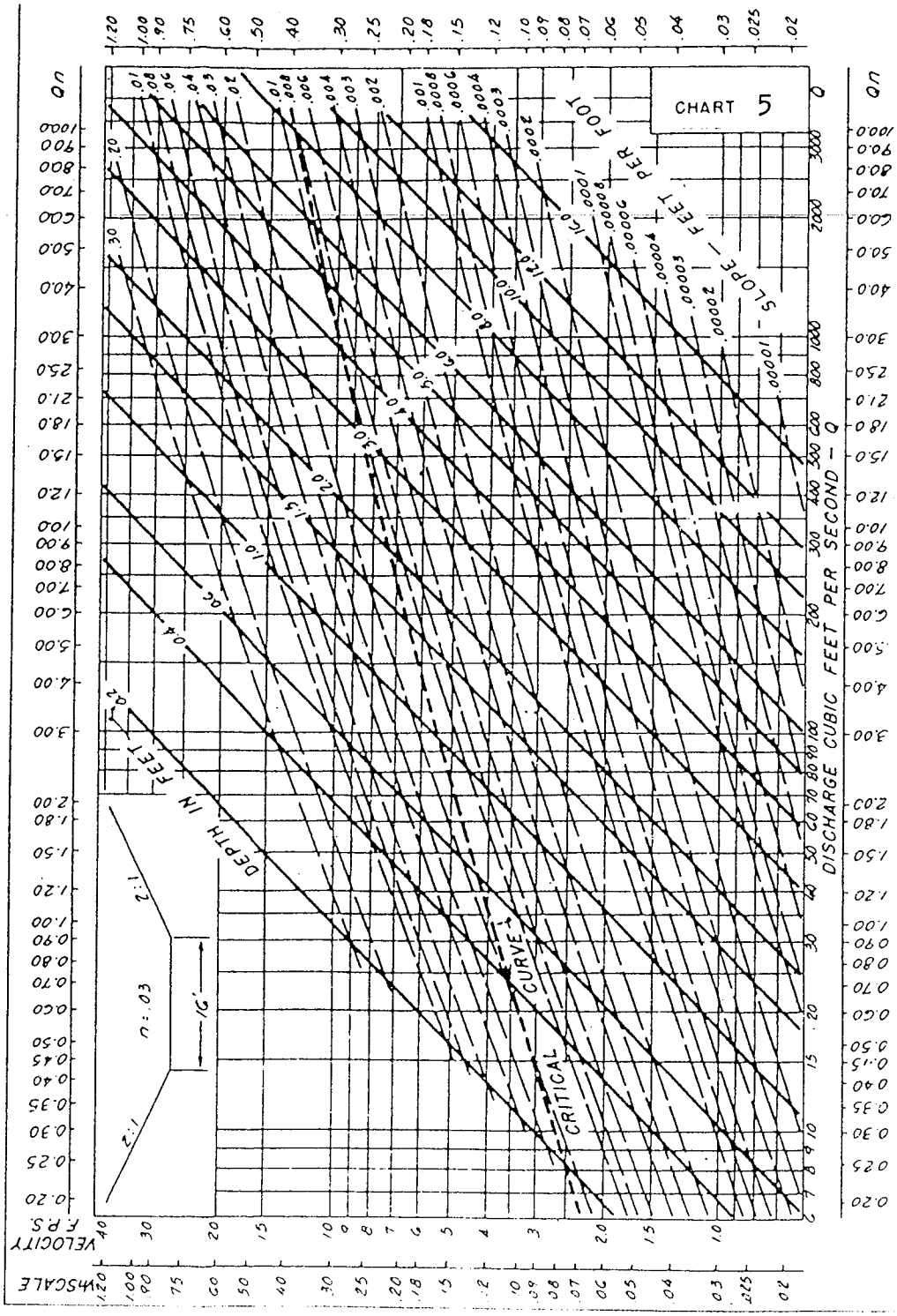


CHANNEL CHART
VERTICAL b = 5 FT.

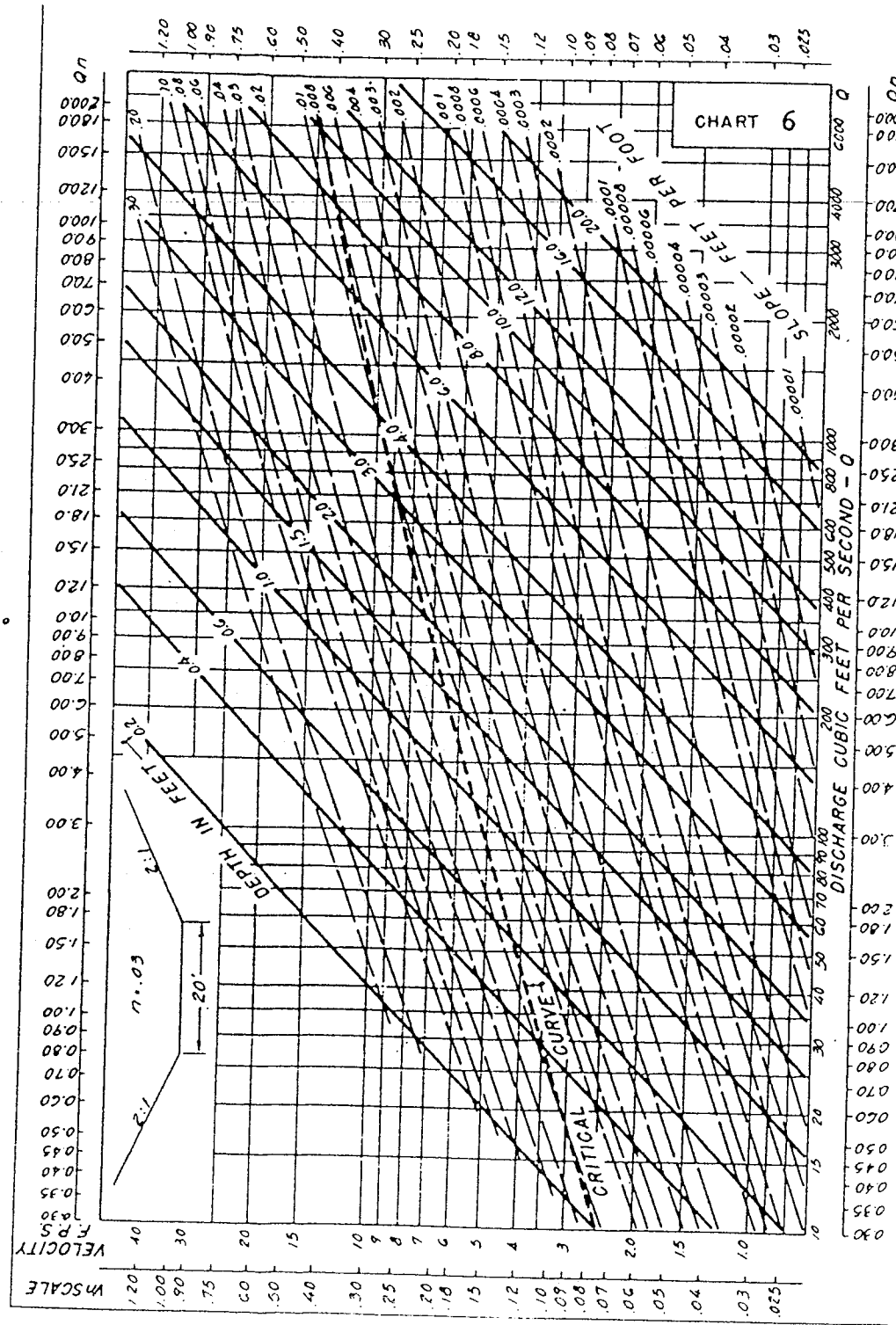


CHANNEL CHART
VERTICAL b = 6 FT.

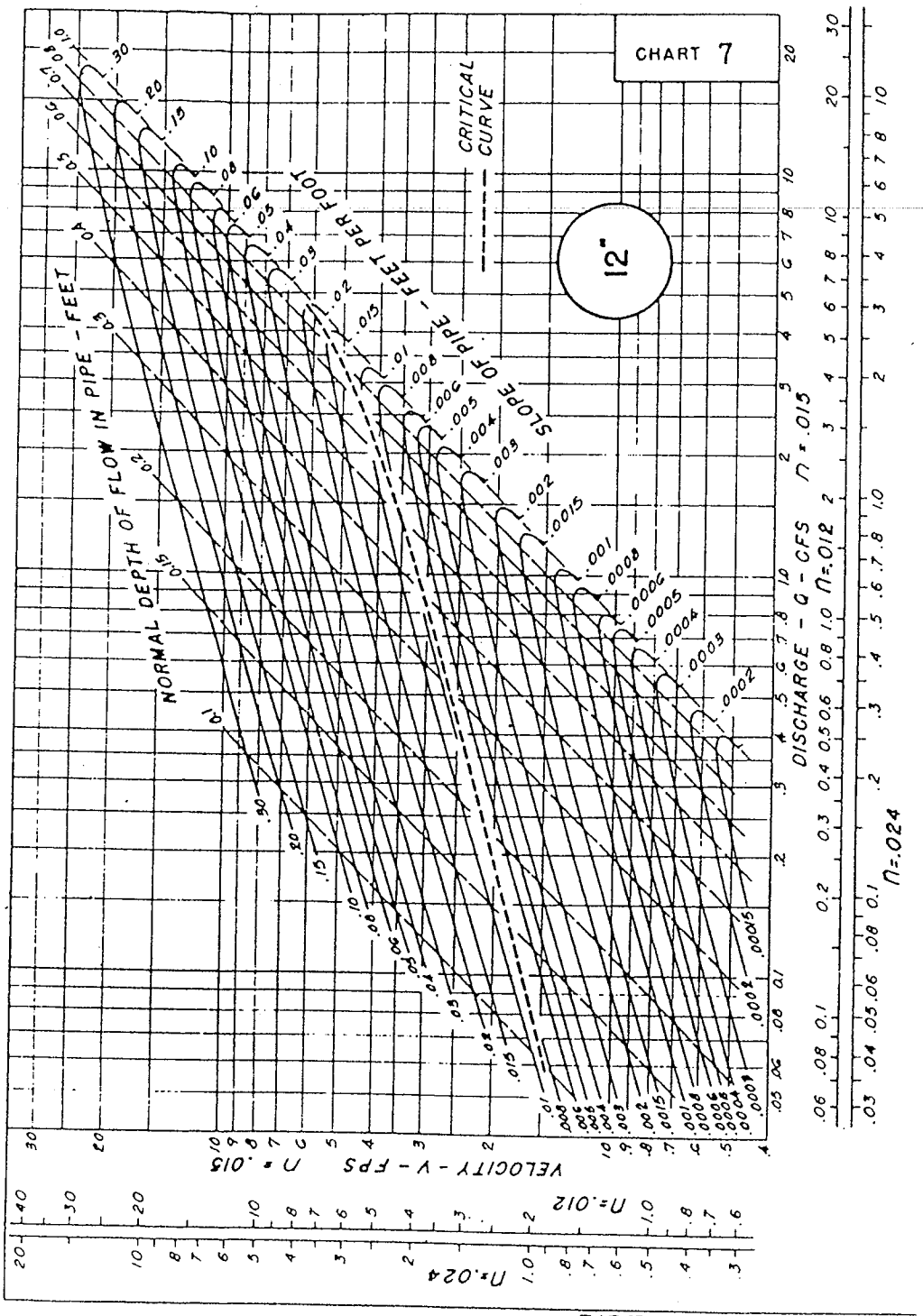

CHANNEL CHART
2:1 b = 12 FT.
Vn SCALE
VELOCITY F.P.S.
n = .03
2:1
2:1
12'
DEPTH IN FEET
CRITICAL CURVE
SLOPE - FEET PER FOOT
CHART 4
DISCHARGE CUBIC FEET PER SECOND - Q
Q
Qn



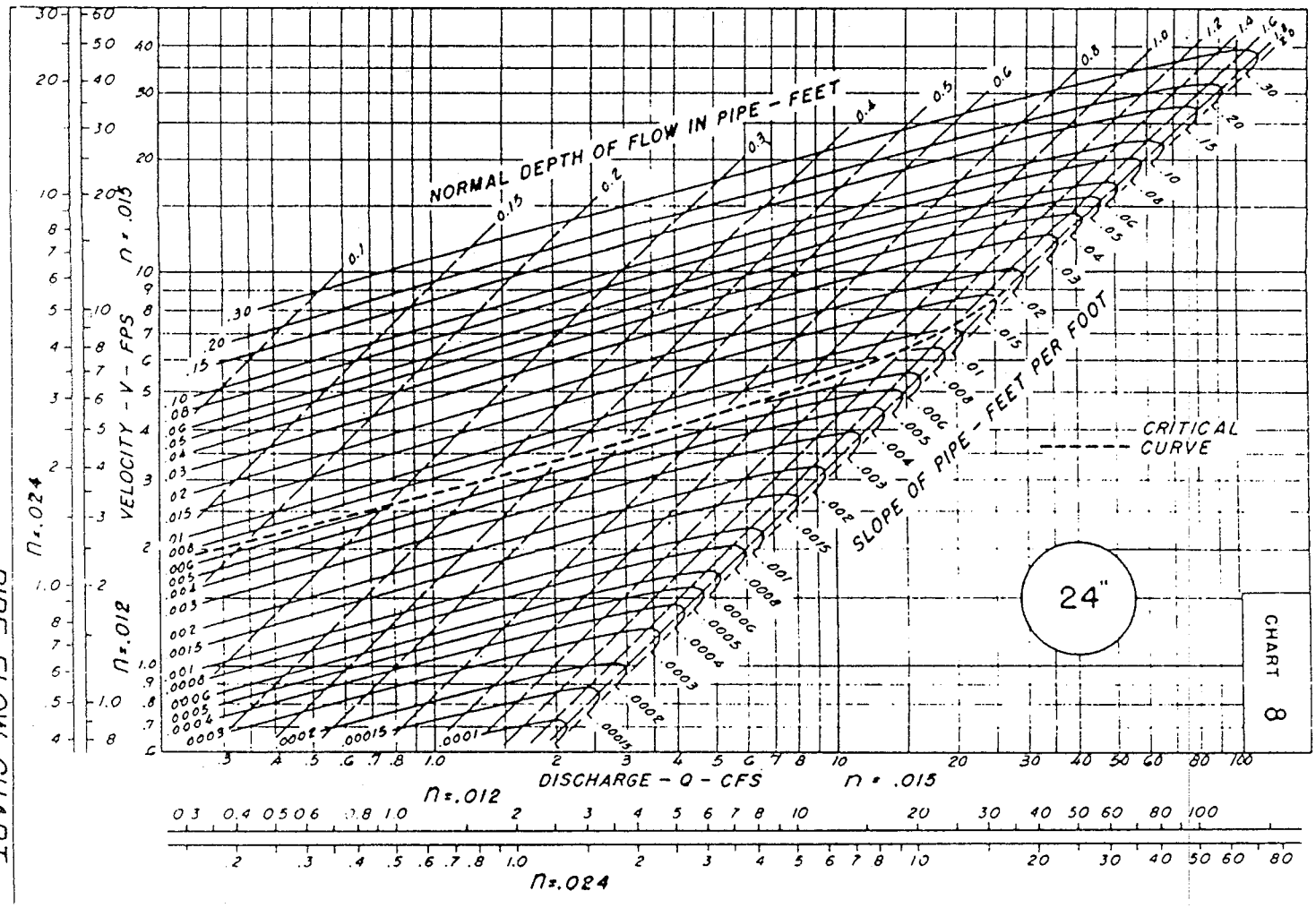
CHANNEL CHART
2:1 b = 16 FT.



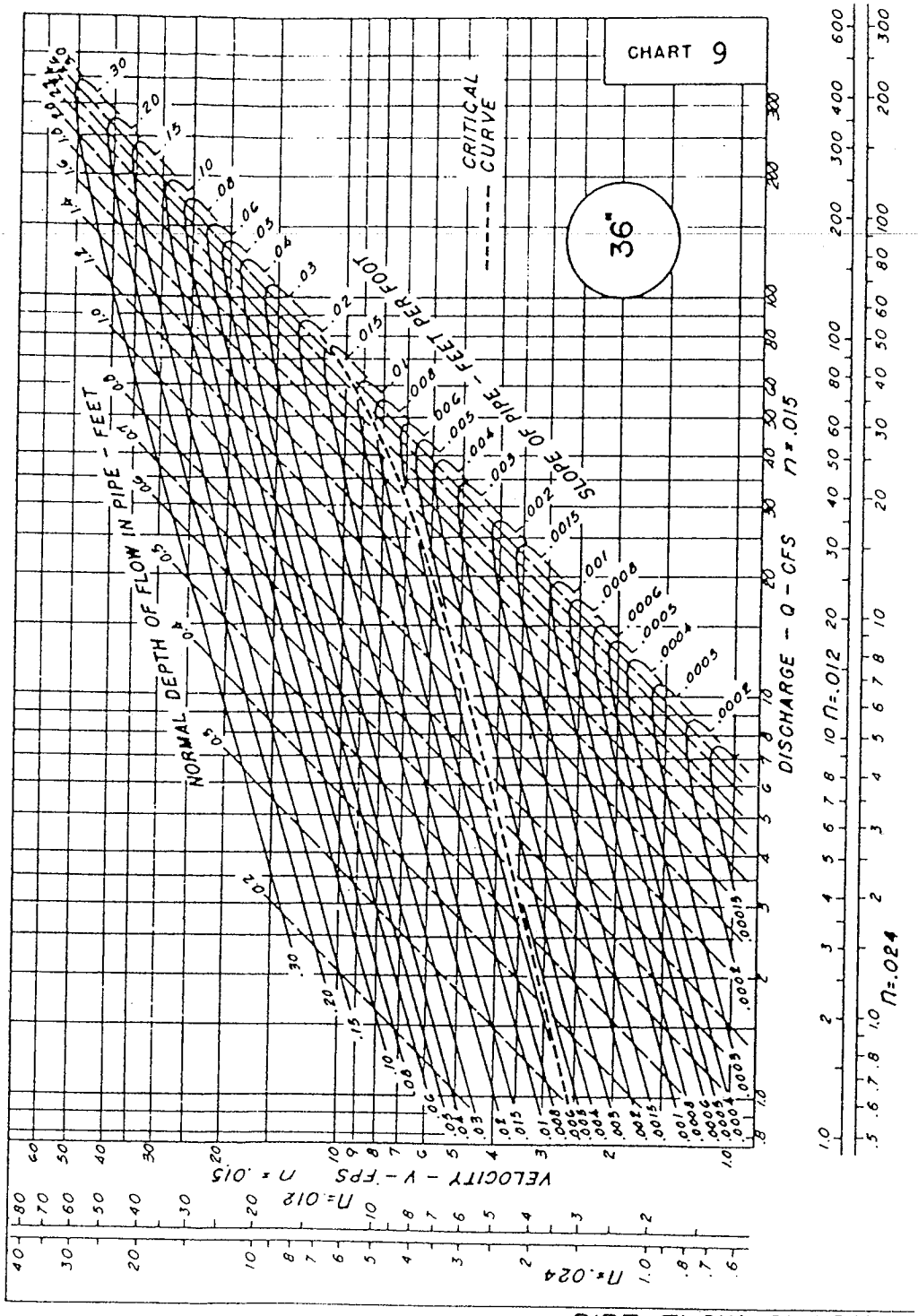
CHANNEL CHART
2:1 b = 20 FT.



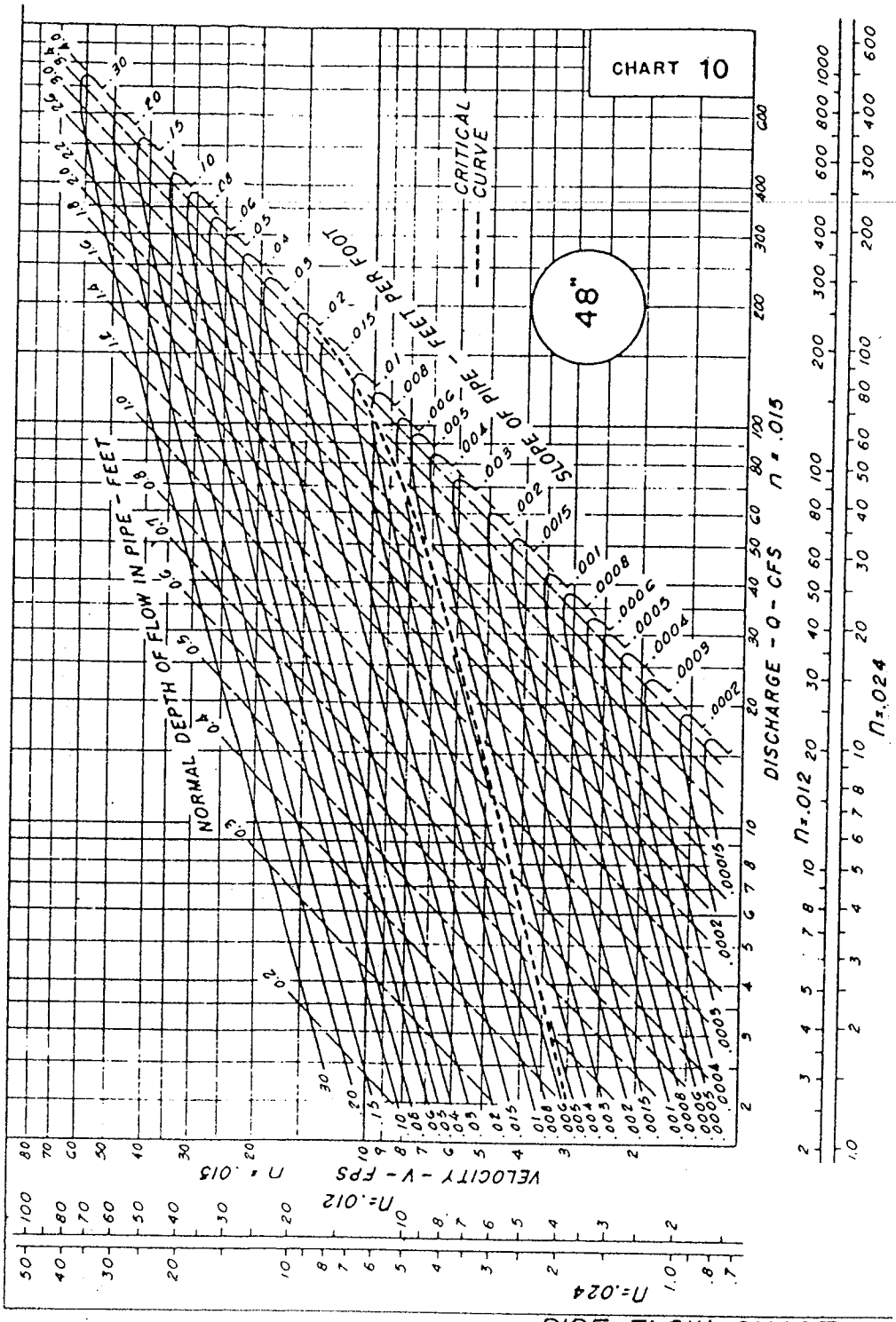
PIPE FLOW CHART
12-INCH DIAMETER



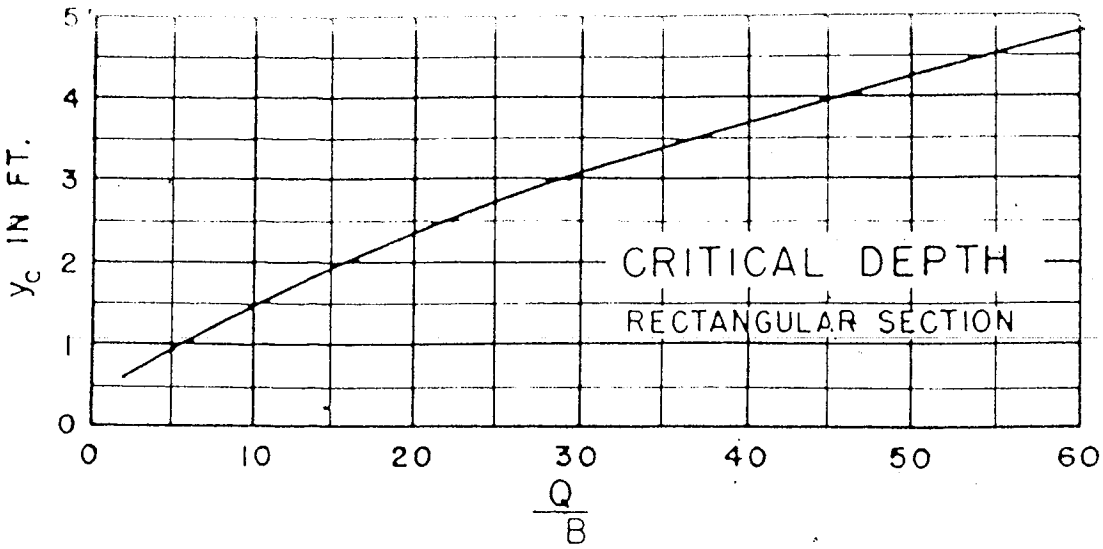
PIPE FLOW CHART
24-INCH DIAMETER

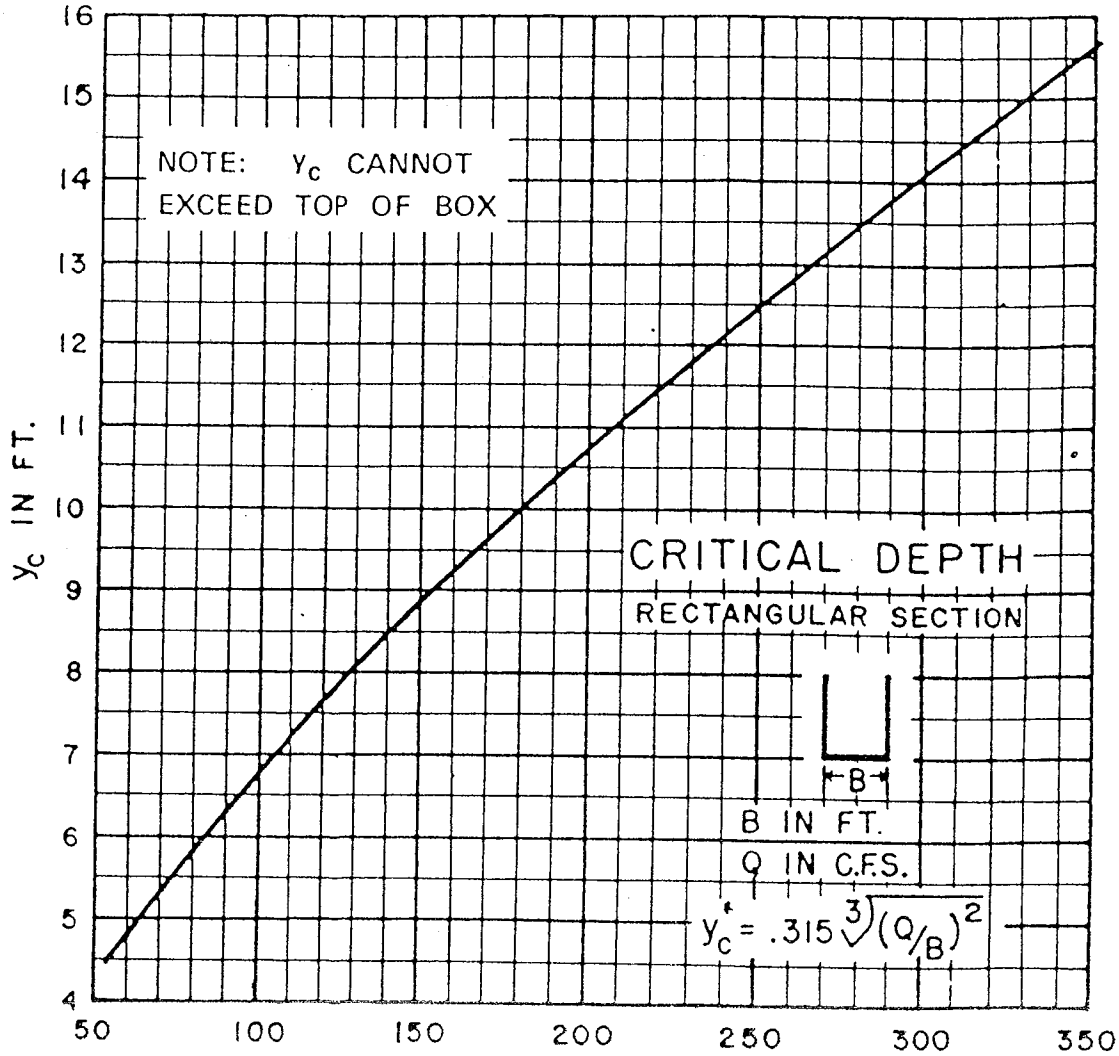


PIPE FLOW CHART
36-INCH DIAMETER



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	.06[illegible]2	.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	1.13	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	3.97
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)

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	4.58	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

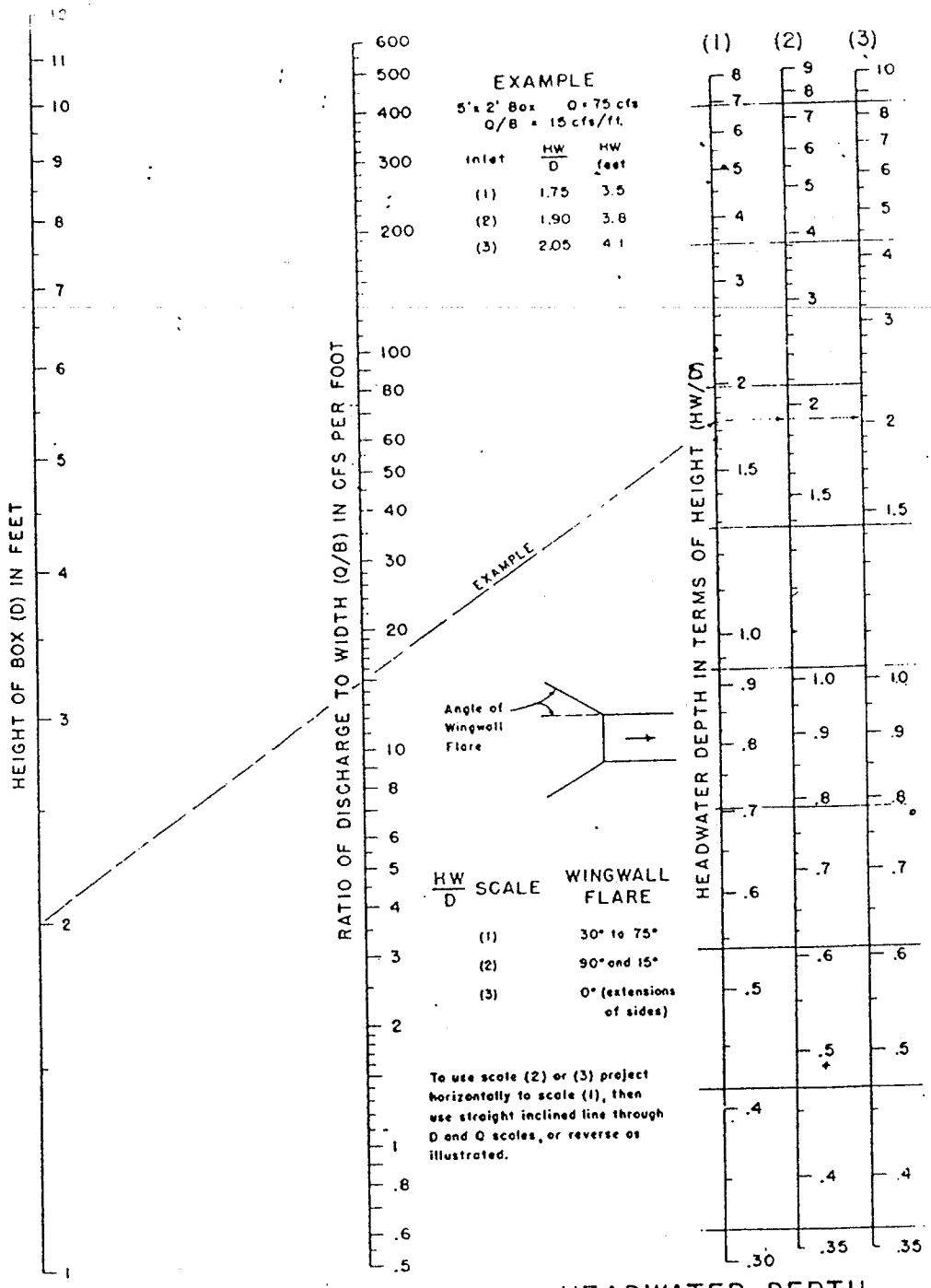
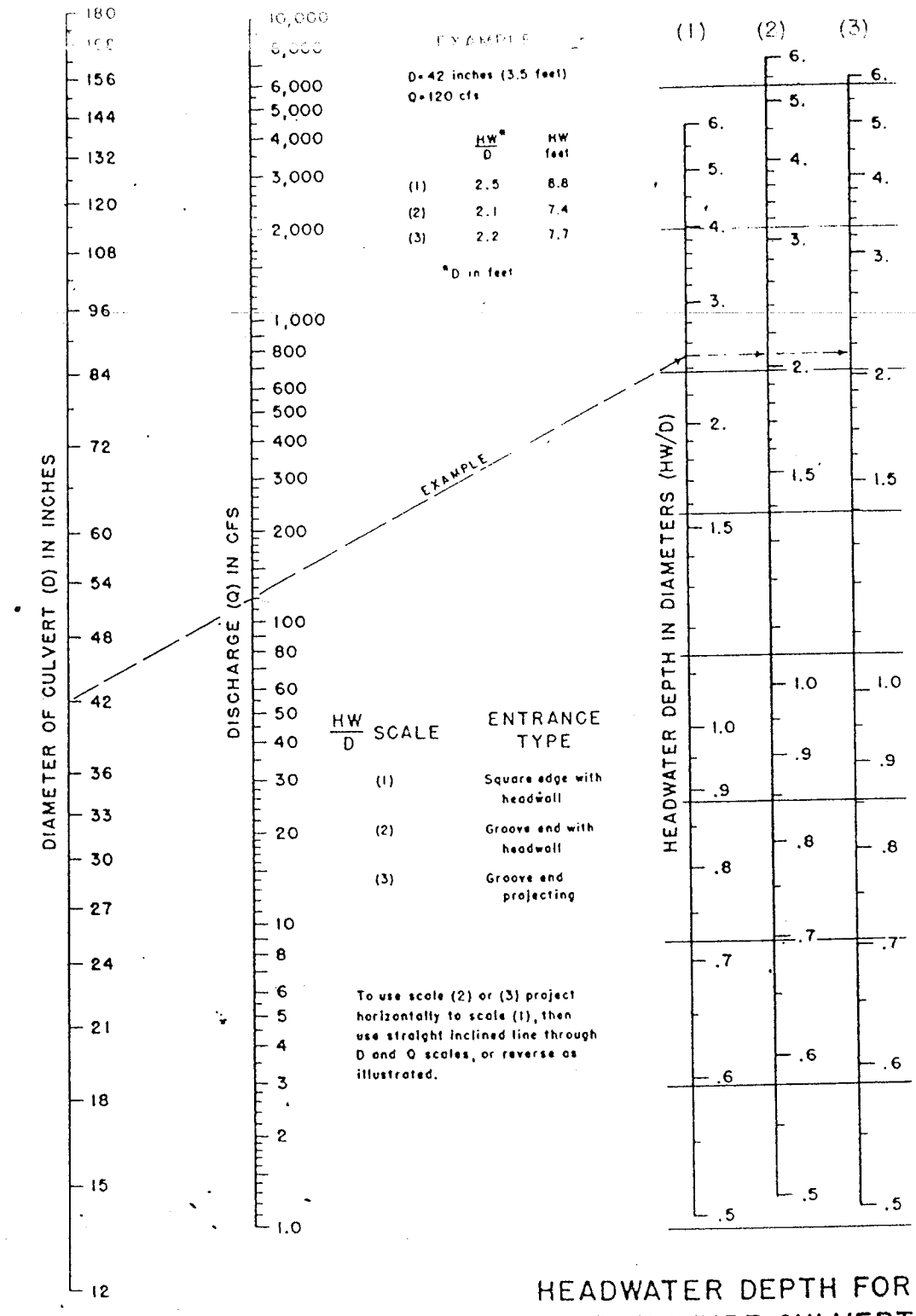


CHART 14. DESIGN MONOGRAPH - FLOW THROUGH CULVERTS

Taken from : Hydraulic Charts for the Selection of Highway Culverts.



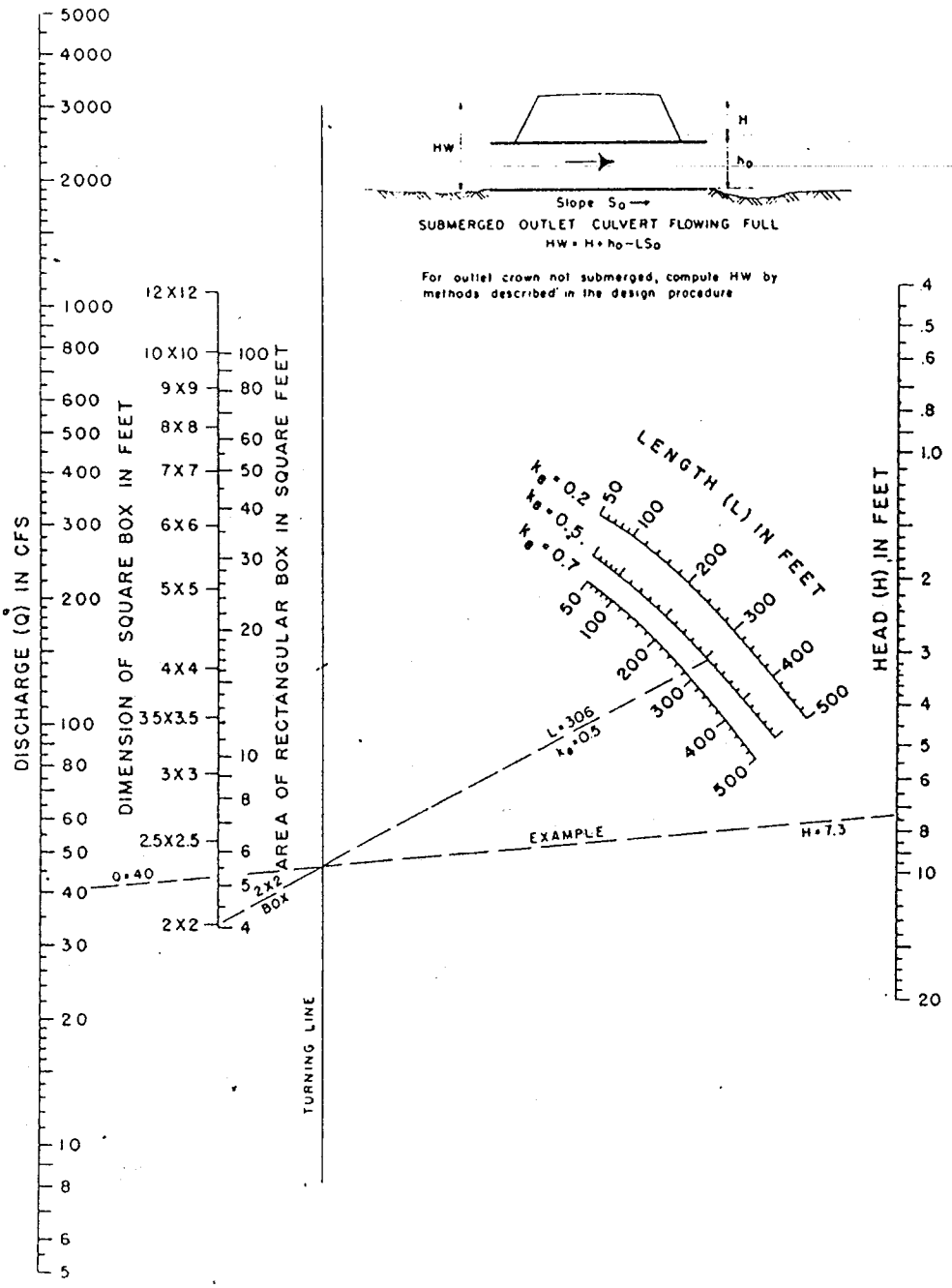
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 15. DESIGN MONOGRAPH - FLOW THROUGH CULVERTS

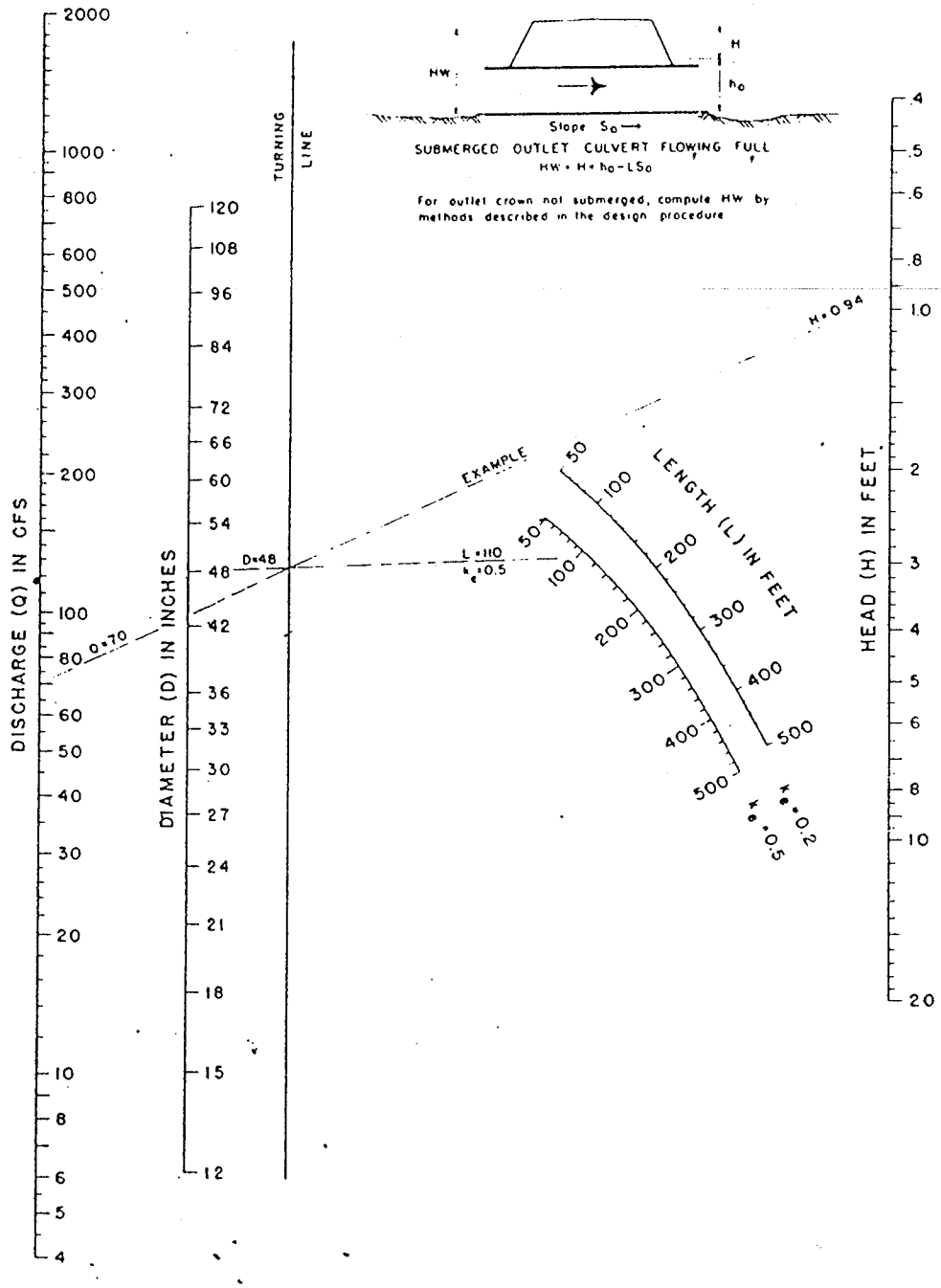
Taken from : Hydraulic Charts for the Selection of Highway Culverts.



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

BUREAU OF PUBLIC ROADS JAN. 1963

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] [7] (values shown are for velocities of 2 and 6 f.p.s.):	Manning's n range [8]
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	0.012
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:[9]	
A. Minor streams [5] (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 [4] 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 poor 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, Research, 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 shingles	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	0–5	7	5
Kentucky bluegrass	5–10	6	4
Smooth brome	Over 10	5	3
Blue grama			
Grass mixture	0–5	5	4
	5–10	4	3
Lespedeza sericea	0–5	3.5	2.5
Weeping lovegrass			
Yellow bluestem			
Kudzu			
Alfalfa			
Crabgrass			
Common lespedeza [3]	[4] 0–5	3.5	2.5
Sudangrass [3]			

[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/H	Factor
1.00	1.21
0	1.24
.5	1.25
.65	1.27
.80	1.28
.90	1.31
1.0	1.34

[1] Interpolations may be made. See derivation of factors on p. [illegible]

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 XII

SEEPAGE CONTROL DEVICES

Henryk Werszko

TYPES OF CUT-OFF EMPLOYED IN HYDRAULIC STRUCTURES

1. Introduction

Although it is not apparent to a visitor looking at the hydraulic structure under operation, e.g., weir, regulator, sluice etc., the understructure is more difficult to build and is more important than the superstructure which is exposed above the water as the stability of the entire work depends on the former.

The understructure in general, can be divided into two interdependable components: the horizontal floor and the vertical cut-offs.

The vertical cut-offs are of special importance in counteracting the undesirable effects of enforced percolation and scour effects. By forming a reasonably watertight and soiltight underground barrier, they protect the structure against undermining of the foundation strata, by increasing the creep path of the percolating sub-surface flow, thus reducing the exit gradient.

Analyzing the elementary seepage theories one can assume that a flat apron, if made long enough, or a single row of sheet piling driven deep enough into a permeable soil, should be sufficient. More advanced theories and practical experience, however, indicate that both the assumptions are neither correct nor practical or economically feasible.

At present, it is generally accepted in hydraulic engineering, that, except in temporary works, we must have both creeps -

vertical and horizontal.

Only by combining the horizontal and vertical creeps together (at a proper ratio) it is possible to have a hydraulic structure which will be safe from scour, undermining and exit gradient, as well as to provide an economical design.

The horizontal floor acts as a foundation supporting the superstructure and, as such, is designed to withstand all the resultant stresses and to transmit the entire load on the subsoil.

The vertical cut-offs, on the other hand, are normally designed as non load carrying members. They are designed for stresses which can incur during handling and installation.

In one particular case the vertical cut-offs can be subjected to a perpendicular load, i.e., when the scour hole develops next to it they act as a low retaining wall.

2. Types of cut-off

There are three basic types of cut-offs used in hydraulic structure:

(a) thick walls

(b) medium thick or cast in-situ concrete diaphragms

(c) thin partitions.

2.1 Thick Walls

- Indian type bottomless wells, sunk in such a way as to form a continuous curtain wall.

- RCC caisson, introduced in Bangladesh about 20 years ago, the sinking of which was mastered by BWDB engineers.

- Egyptian type cut-off, made of two parallel lines of sheet piling, driven 3 to 6 ft apart with concrete filling in between them casted under water.

RCC Caisson (general)

From our observation, during implementation of the CIDA and IDA projects, it appears that the RCC caisson is the cut-off that is most favoured by Bangladesh contractors and most of the BWDB engineers.

Because of the special status of the RCC caisson, we feel that it is only appropriate to reserve a few more lines within this paper concerning its application and design, with a view that it will invite further discussion on the subject.

As already mentioned the RCC caisson is commonly used in Bangladesh and its construction techniques has been mastered here to such an extent that there probably is at least one man in every village that knows how to construct and sink a caisson, using locally engineer's "know-how" on designing a caisson; be

it cut-off or load bearing caisson.

However, during our numerous visits to the construction sites and our extensive discussions with divisional engineers and contractors, we found that there are still many questions concerning differences of opinions. Let's list some of the questions - problems, and try to find the answers:

1. should the cut-off caisson be open or have bottom plugged?

2. thickness of walls (steining) and reinforcement?

3. dry sinking vs submerged sinking?

4. filling material?

Before we began colaborating on the answers to these questions a thorough search of the available written technical material was done in order to find out the practices of civil engineers in other parts of the world. To our surprise, not a single reference on RCC cut-off caissons, in the form used by BWDB, was found amongst the European, American or Indian material, to which we had access during this short available time. It appears that long caissons are not being used as hydraulic structure cut-offs neither in Europe nor in America.

In India, where the system originates, well foundations have been constructed for hundreds of years and at the turn of this century, Indian engineers were the first to introduce wells to

hydraulic engineering as cut-offs. The Indian type cut-off wells consists of a number of wells (in America called caisson) sunk one next to another so as to form a continuous underground barrier.

In some works the caissons remain open without bottom plugs (Khanki Lower Chanab canal weir sluices), in Khanki Lower Chanab Weir-Panjab, India, the bottom plugs were used but the wells were left hollow. In other structures (Norora Weir and Morala Weir chanab River) the bottoms were plugged and the wells were filled in with sand.

Bottomless vs Closed Caisson

In an attempt to answer question No. 1 bottomless vs closed caisson, one should analyze the conditions in which the element of the structure will be working, taking into account all the possible conditions.

Some of the engineers, whom we met, believe that by keeping the caisson open the creep path will be increased by double the depth of the caissons as the percolation flow line will follow the contact between the solid works and permeable soil.

This assumption will be correct only if the horizontal distance between the opposite walls of the caisson are greater than twice their depth, otherwise the percolation flow line will follow the short path.

Therefore there is no advantage (no extension of the creep path) in keeping the caisson open.

The purpose of the bottom plug in a foundation caisson is to transmit the entire load coming from the staining into the foundation material on which the caisson is resting.

The cut-off caissons are not considered to be external load bearing caissons, however, they can be exposed to a similar stress as the foundation caissons.

Properly designed caissons should be heavy enough to overcome resistance against sinking (skin friction) during the course of construction.

After the caisson is sunk to the final level it is desirable that it remains stationary, that there be no further settlement. More often than not, part of the superstructure (wing wall/return wall) is being constructed over the caisson, which imposes an extra weight. This extra weight in some cases can be enough to exceed the skin friction resistance and soil bearing capacity below the net cross sectional area.

In the case the caisson is founded on soil which is weak in shear strength (silts and soft clays), base failure and unpredicted settlement under the extra load coming from the return walls and apron can occur. With the bottom plug in place, the weight is

distributed over the gross cross section, therefore the load per unit area is reduced thus minimizing the possibility of further settlement/sinking.

From the above discussion it can be concluded that unless hard soil is encountered during the final stage of sinking, the cut-off caisson should be terminated in a bottom plug. In India 1:2:4 concrete is used in the bottom plug.

Thickness of Staining

Thickness of Staining (Caisson Walls) and reinforcement should be derived from structural requirement, taking into account the horizontal and vertical external forces only. The inside fill material and water pressure should be ignore (caisson hollow), however, the weight of the filling should be taken into account while calculating the foundation stresses.

The thickness of staining determined from the structural requirement should be checked for the sinking effort which is defined as below:

$$\text{Sinking effort in cwt/sq. ft.} = \frac{\text{Weight of the caisson at the design depth-buoyancy in cwt}}{\text{Outside surface area of the caisson in square feet.}}$$

The sinking effort varies from 2 to 5 cwt/sq.ft. in pure sand to clay respectively.

In order for the caisson to sink under the self weight, the

design sinking effort should be about 10% higher than the value given for the particular soil.

If the caisson is too light it may not move at all or may move very slowly. It becomes necessary to put an extra load and then to remove it repeating this for each lift. This slows down the progress of sinking and also increases its cost. In case of sand the caisson will not move steadily. Only after the sand is removed clear below the cutting shoe it starts to slide slowly while caving in of the ground around the caisson, thus weakening the foundation of the hydraulic structure.

Dry vs Submerged Sinking

There are two methods of sinking the caisson, open sinking and pneumatic sinking. In our practice only the open sinking method is used, therefore, there is no need to discuss the other within this paper.

The BWDB contractors use two different techniques of construction, namely (a) by pumping water out from inside the chambers and manually removing the soil in dry, and (b) by using grab bucket (dredger) and removing the soil from under water.

There is no need to elaborate on either as both are sufficiently mastered by practicing contractors and the BWDB supervisory engineers.

The selection of either technique is governed by the contractor's preference and his experience, however, it has been noticed that the supervisory engineers do not always agree with contractors. Some engineers state that the dry caisson with water pumped out will not sink easily, as the external forces (soil pressure and unbalanced water pressure) are greater and hold it in place. This statement is false because in the case of dry well there is no buoyancy and the sinking effort is higher, therefore, the caisson should sink easier.

Before the method of sinking is selected, the soil boring report should be analyzed.

Dewatering of the caisson by pumping out water is generally not recommended when sinking is done through sand.

Sinking in clay is done in dry.

In case of silt and sand mixed with clay it is not so easy to select the correct method, soils and ground water conditions have to be analyzed and engineering judgement used, taking into account the past experience in the area and/or with similar soils.

Sinking Through Sand

If sinking is done through sand, a large quantity of sand keeps flowing into the caisson and the quantity taken out always exceeds the volume displaced by the caisson. This results in

loosening of the foundation material and subsidence of soil at the bed level.

The loosening of the soil material around the caisson in the process of sinking can not be avoided, however, the amount of the "extra" excavation and disturbance of the foundation soil can be controlled through proper sinking techniques.

The inflow of sand into the chamber from outside below the cutting edge is caused by ground water flow. This can be reduced if the static level of water inside the chamber is maintained higher than the outside by pumping water into it. The difference in water level results in continuous flow of water under the cutting edge from inside to outside and prevents the flow of sand into the chamber.

Providing of thick staining (extra weight) also reduces the inflow of sand and the caisson sinks steadily every time the soil is removed.

Sinking Through Clay

The mechanism of caisson sinking through clay is slightly different from that for granular materials. Clay can stand even vertically, due to cohesion. The skin friction and bearing resistance of clay are much greater than those offered by sand.

The soil does not fall freely into the dredge hole and caisson remains still even after the dredging pit has been excavated to a considerable depth below the cutting edge. The soil beneath the

cutting edge is being "sliced" while the caisson moves down.

To overcome the much greater (than in sand) sinking resistance, a higher sinking effort is required. This is accomplished by pumping water out, and quite often placing temporary extra weight on top of the staining.

Once the chambers are dewatered, very little pumping is required since permeability of clay is very low, little water is seeping in. The effect of buoyancy is eliminated and the effective weight of the caisson gets about double.

In a completely dewatered chamber, the excavation can be carried out much more conveniently and speedily. Workers enter inside and proceed with manual excavation right to below the shoe.

Sinking in clay does not proceed at a steady pace but in spurts which may be more than 3 feet at a time. "Hanging" of caisson is common. Horizontal cracks, even separation of the staining can occur. Heavier, stronger staining with extra vertical bend road should be designed for sinking in clay.

Special Problems of Open Sinking

Blow of sand, occurs in layered soils with water in the upper sandy layer under pressure created by the dewatering of caisson partially sunk into the impermeable lower stratum.

The difference in the hydraulic pressure inside and outside the caisson causes the flow of water along the surface of contact between the outside wall and clay. If the caisson is not

sufficiently deep into the clay the velocity of flow is high. The flowing water enlarges the passage washing out the clay.

At one point water and sand gushes into the caisson, filling it with sand and water in matter of seconds.

This presents extreme danger to the men working inside. While standing at the bottom, once their feet get covered with the moving in sand, men can not get freed in time to get to the surface and drown.

Seepage of water should be carefully watched when the sinking of caisson is done by dewatering and it can be checked by putting sand bags over the area where leakage takes place. Excavation should be continued in the rest of the area (chambers) and the area which is giving the blow should not be touched till the caisson has sunk about 6 to 12 inches.

If a blow takes place it is necessary to cover it at the source before complete dewatering can be done again. Usually a funnel shaped depression forms outside the wall on the side blow has taken place. Scrap gunny bags and green branches or straw are thrown into the funnel while dredging and pumping of water continues. Gradually the thrown in material is sucked into the clay layer and blocks the passage of water.

<u>Quick Sand (sand boiling)</u> conditions exist when fine sand or silt under artesian pressure is overlaid by an impervious layer.

As soon as the impervious layer is pierced through a steep hydraulic gradient is established, the sand looses its internal friction and starts flowing upwards or in extreme case the caisson sinks into it as the material does not offer any resistance.

As long as the water and sand flows upwards sinking of caisson can not proceed.

The quick condition can be arrested by keeping water level in the caisson higher than the hydraulic pressure in the pierced layer.

Further sinking of the caisson has to be done under water. If it is not practical to reach course sand or impermeable layer, the bottom plug also has to be done under water.

Caisson Filling Material

Sand is generally accepted by BWDB engineers as the only material with which the caisson chamber can be filled. To date, very little was published about caisson application for cut-offs in hydraulic structures. They have been used quite successfully in India, but have not been adopted in Europe and America. This was probably due to the time consuming and the high labour input required for their construction.

However, the caisson as a deep foundation is widely used all over the world. References about their construction and design can be found in majority of the foundation engineering books.

No doubt, analogy exists between the load bearing and the cut-off caisson. Both are subjected to similar loads, but there are also some differences. They serve different purposes, therefore, the design requirements should not be the same.

Any practicing engineer, being behind the design desk or supervising construction, should be aware of its purpose and should understand the behaviour of the elements of the structure for which he is responsible.

Coming back to the caisson filling material, let's first see <u>what is its purpose?</u>

- It cannot transmit the weight of the superstructure to the bottom plug due to its settlement.
- Staining is designed to withstand the external forces with the caisson being dewatered.

Therefore it does not necessarily contribute towards the structural strength of the caisson. It only partly counteracts the stresses caused by soil and water pressure acting from outside the staining.

In case of caisson as a deep foundation for bridge piers and high towers, the main purpose of the filling is to increase the stability of the structure by increasing its weight near the base.

What about other structures where the bending moments are low but the load coming form the structures itself is high i.e. central

caisson for hydraulic regulator, heavy buildings etc.

The designer resorted to deep foundation (caisson in this case) because the bearing pressure was already too high-by filling in caisson it is further increased.

When the caisson is completely filled in, the top plug or floor apron can be constructed directly over it without providing any shuttering.

<u>Water material is being used for caisson filling?</u>

In India - sand or lean concrete, caisson partially or completely filled up.

In America - sand and concrete

In Europe - sand, gravel rocks with sand and concrete.

No reference concerning specifications for the sand used for caisson filling could be found in Indian and American technical publications.

European authors are more flexible in their requirements by recommending non-cohesive materials for caisson filling.

Silt is also non-cohesive and its engineering properties are essentially these of fine sand.

Clays are not recommended due to their shrinkage and expansion causing high lateral pressure. Swelling pressure in some expansive clays exceed 10 tons per square foot.

In practice, European engineers base their selection of the filling material on economical analyses. Sand is the most commonly used material because of its low cost, i.e. the lowest rates for excavation and filling.

In Bangladesh hundreds of caissons are constructed every year and even more are being designed at present. This probably more than in any other country. Yet, the design specification is limited to just a simple, "sand filling", without giving the unit weight, FM factor, maximum clay or organic material content.

We also know that the sand filling is not always diligently carried out by the contractors. Local materials like silt and silty sand are being used in place of the sand filling. In spite of this, to date there was no caisson failure on the BWDB hydraulic structure.

2.2. Medium Thick of Cast In-Situ Concrete Diaphragms

- Cement bound curtains
- RCC cut-off walls

The cement bound curtains are constructed by successively overlapping individual piles. Each mix-in-place pile consists of a column of soil intimately mixed with mortar to form a pile like structure within the soil. Such a pile is constructed by injecting mortar through a vertical rotating shaft, the lower end of which is equipped with a mixing head for combining the soil with the injected mortar. The piles may be reinforced as required. the average thickness of such a wall is about 18

inches and can be constructed, with equipment presently available, to a depth of 50 feet.

As the injected mortar is being mixed with the foundation medium, this method is not applicable for clayey soils. It can be successfully used in granular soils, including silt, sand and gravel but it is not capable of handling cobbles and boulders.

The RCC cut-off walls are at present, the most commonly used cut-offs on the BWDB hydraulic structures. Besides simplicity in constructin and in-service reliability, in Bangladesh they are the most economical.

In our practice, thickness of the RCC cut-off wall ranges from 6 inches to 12 inches, depending on dimensions of the wall and the potential scour hole that can be expected.

From the practical experience (inspection of numerous sluices and regulators under repair, with cut-off walls partially exposed by scour holes), it can be gathered that a minimum reinforcement is sufficient for the wall to withstand any soil load that can be incurred should the scour hole develop.

A major drawback of the RCC cut-off walls constructed in-situ is the need to retain the site excavated to the lowest level of the structure free of water. This is not always practical and if boiling sand is encountered, it is very difficult to control it under surface dewatering.

In that case it is better to resort to other types of cut-off, instead of carrying out a very costly dewatering. Also construction of RCC cut-off walls in -situ requires a large amount of excavation below the structure which will have to be field back. This disturbing of the original foundation material is undesirable as the structure becomes more vulnerable to seepage and local settlement.

2.3 Thin Partitions - Sheetpiling Walls

A sheetpiling wall consists of a series of sheet piles driven side by side into the ground, thus forming a continuous vertical wall, for the purpose of retaining and/or protecting the soil.

The sheetpiling walls are of special importance in the design of hydraulic structues. By forming a reasonably, watertight and "soiltight" underground barrier, they protect the structure against undermining of the foundation strata, by increasing the creep path of the percolating sub-surface flow.

In hydrotechnical engineering construction practice, the sheet-piling walls are usually used as cut-off walls at the end of the upstream and downstream aprons. The cut-off walls are driven down below the maximum depth of likely scour.

Sheet-piling walls placed under the central portion of the structure, or under the headwalls, in addition to increasing the under-ground water seepage path, also improve the soil bearing capacity by acting similarly to pile foundation.

In design and construction of hydraulic structures, i.e. water retention structures, sluices and regulators, the sheetpiling walls can be used in place of the traditional RCC cut-off walls or caissons.

Some of the more important aspects of sheetpiling walls are:

a) Reduced construction time; while the sinking of a caisson on the average requires about 3 months, for a medium size structure, driving of a sheet pile wall takes about 3 weeks.

b) Undisturbed foundation soil; the undisturbed floor and side slope soils have great effect on construction quality and overall cost reduction.

Types of Sheet Piling

In respect of material used, there are three types of commonly used sheet piles:

- steel sheet piles
- wood sheet piles and
- concrete sheet piles

Steel Sheet Piles

Steel sheet piling consists of precast steel sections provided with interlocks connecting successive piles to one another to form a continuous walls.

There is a variety of available sheet piles manufactured in

light, medium or heavyweight sections to meet different structural requirements and foundation soils.

The steel sheet piles are the most universally used sheet piles in modern construction practice. They are easy in handling and transportation. The installation is easy, as they can be driven into different types of soil, ranging from clayey to gravelly and stoney soils. The steel pile can be used as a permanent structural member or as a temporary foundation formwork in difficult, usually underwater, foundation conditions.

The performance and longevity of the steel sheet pile can be regarded as independent of the variation of ground water levels.

Wood Sheet Piles

Wood sheet piles can be made of single wood planks driven edge to edge in one row, single wood planks driven in two rows, piles made of two or three planks joined together (Wakefield type) or prefabricated piles with different types of tongue and groove and splined piles.

Hardwood is the best suited timber for wood sheet piles, as it is durable and strong enough to withstand the hammer blows and soil resistance. The hardwood sheet piles can be driven into medium sandy soils, without providing a driving metal shoe.

For driving piles in hard sand or gravelly soils, the lower end of the piles should be protected with a driving shoe and the upper ends with straps made of 1/16 to 1/8 in. thick metal. If

hardwood is not available, a softwood timber can be used, providing it is of good quality and without knots.

Both hardwood and softwood piles can be considered permanent, if they are permanently below the groundwater level. Wooden sheet piles should not be used where the ground water level varies to such an extent that they can be exposed to free oxygen. This is due to a very fast decaying of wood under variable conditions.

In saline water, wooden sheet piles should also remain under water, and to be protected against potential damage from aquatic organisms, they should remain permanently underground.

The wooden sheet piles have been successfully used in sluice remodelling and repair works in Polder 48 and Polder 47/3 in 1983.

Types of sheet piles and construction method is illustrated in Figure 11.

Concrete Sheet Piles

Concrete sheet piles can be made as precast reinforced concrete members (full or hollow section), precast member of prestressed concrete or concrete piles driven into the ground next to each other without tongue and groove joints.

The concrete sheet piles, because of their bulk and weight, usually are manufactured at the construction site. A large amount of reinforcing steel is required and their cost is high. They are difficult in transport and because of their weight,

heavy handling and driving equipment is required.

The reinforced concrete sheet piles can resist substantial vertical loads and are more durable than those of steel or wood piles, however, because of the above negative points their use in civil engineering construction practice is limited.

3. Comparative Cost of Cut-offs (BWDB structures)

In 1984/85 the average cost per square feet of installed cut-off wall was:

RCC caisson	Tk 370
Steel sheet pile	310
Wood sheet pile	150
RCC cut-off wall	170

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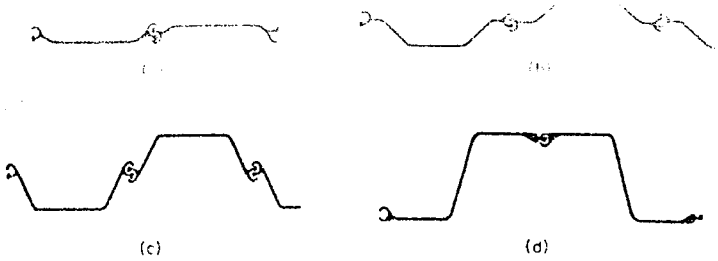
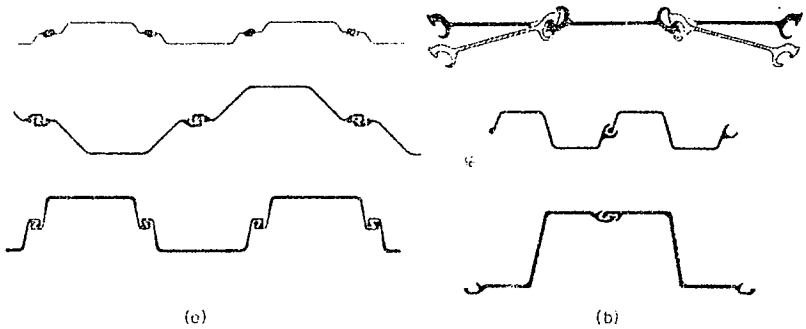
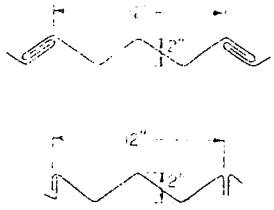
(a) STEEL SHEET PILES

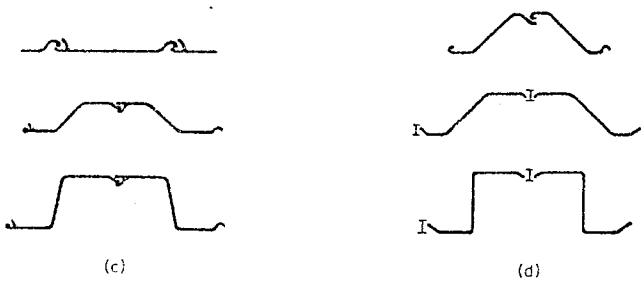


Steel sheet piles: (a) straight web piles; (b) shallow arch piles; (c) deep arch piles; (d) Z-piles. Straight web piles are used where piles are subjected to tension and interlocking strength is of primary importance–primarily in cellular cofferdams. Finger-and-thumb type interlocks provide watertightness and strength. Arch web piles are used to resist large bending–cantilever and anchored walls. They are suitable where difficult driving and retration are contemplated and have finger-and-thumb type interlocks. Z-piles have the highest bending strength. Ball-and-socket type interlocks offer least driving resistance.





Lightweight steel sheeting. Armco Steel Corp.



Some European steel sheet piles: (a) Larsen piles (after Western Service Corp.); (b) Arbed-Belval piles (after Amerlux Steel Products Corp.); (c) Hoesch piles; (d) Krupp piles.

(b) REINFORCED CONCRETE SHEET PILES

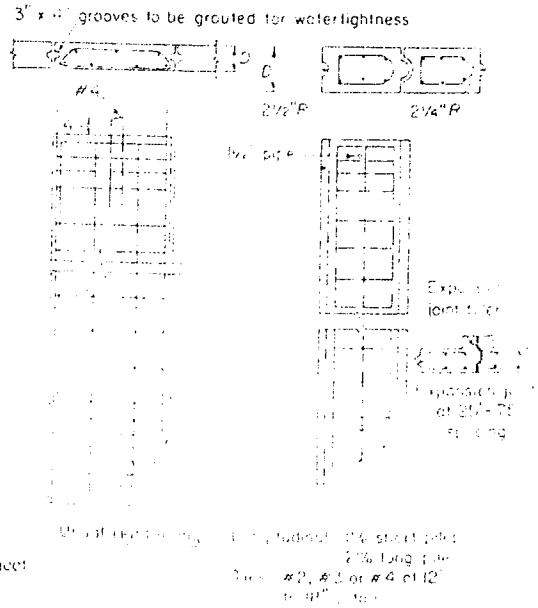
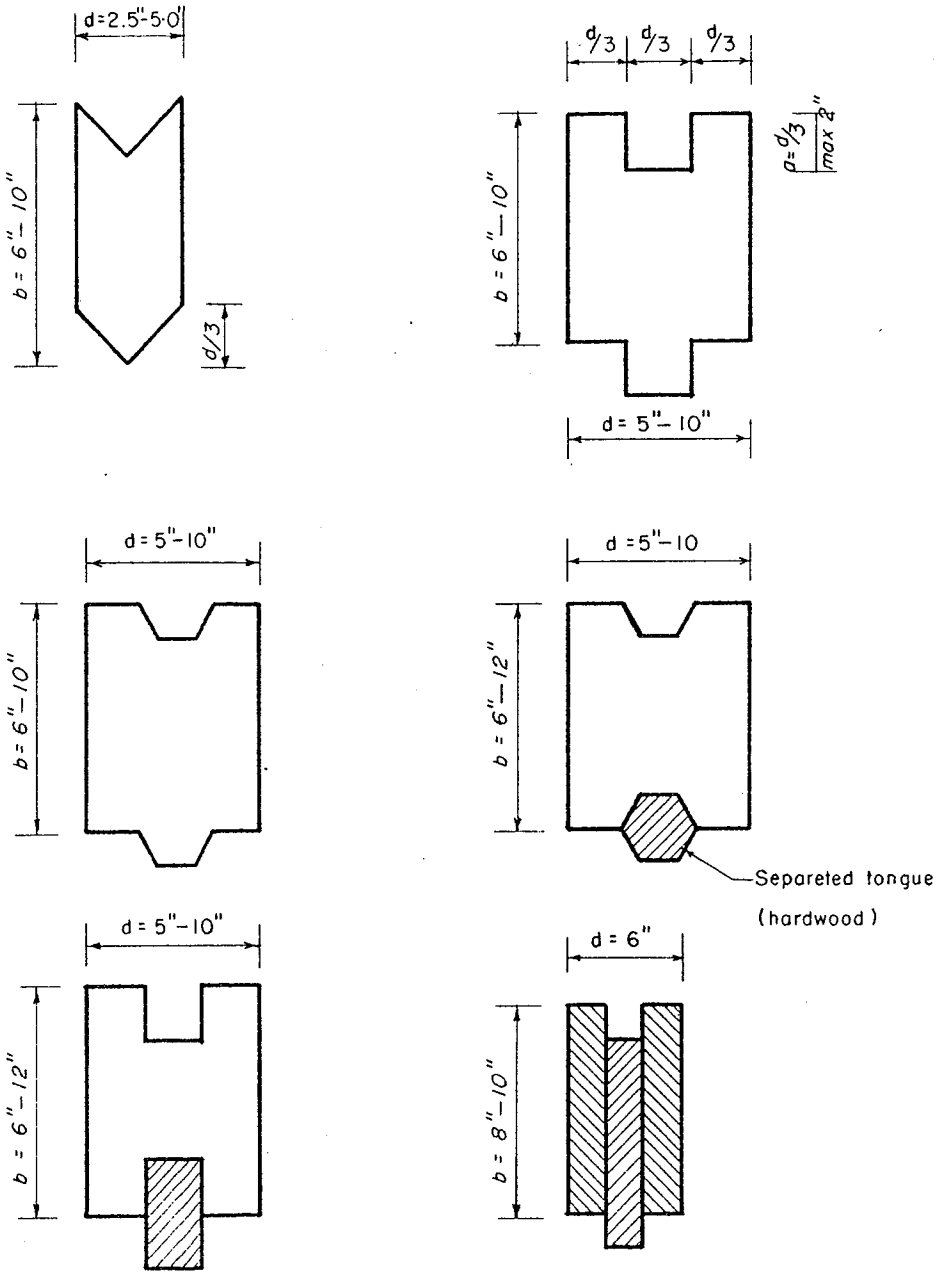


Figure 11 Concrete sheet piles.

Figure II(c) Types of wood sheet piles



NOTE : Tongue should be little longer than groove depth for better contact (sealing)

Figure II(d) Types of wood sheetpiling walls

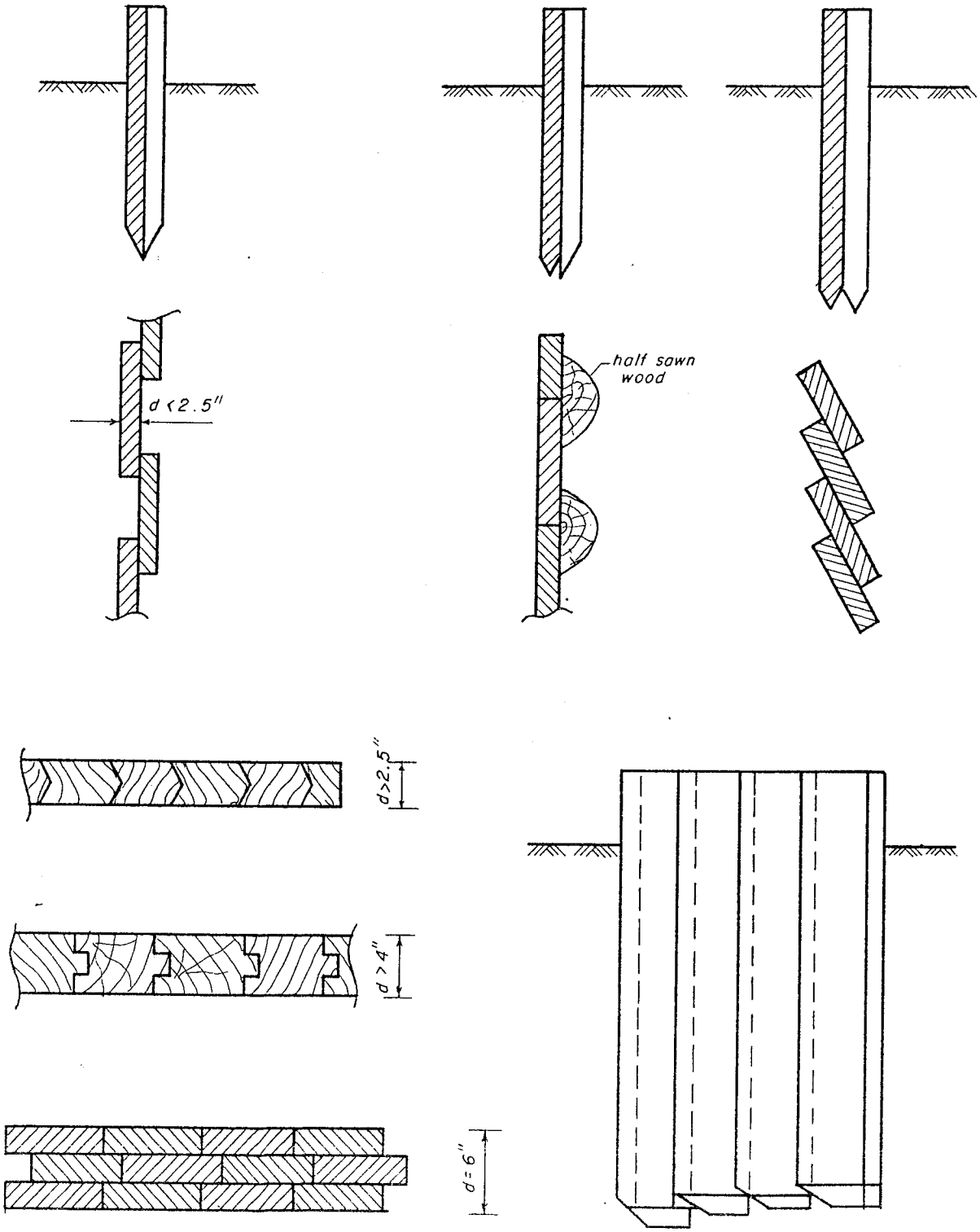
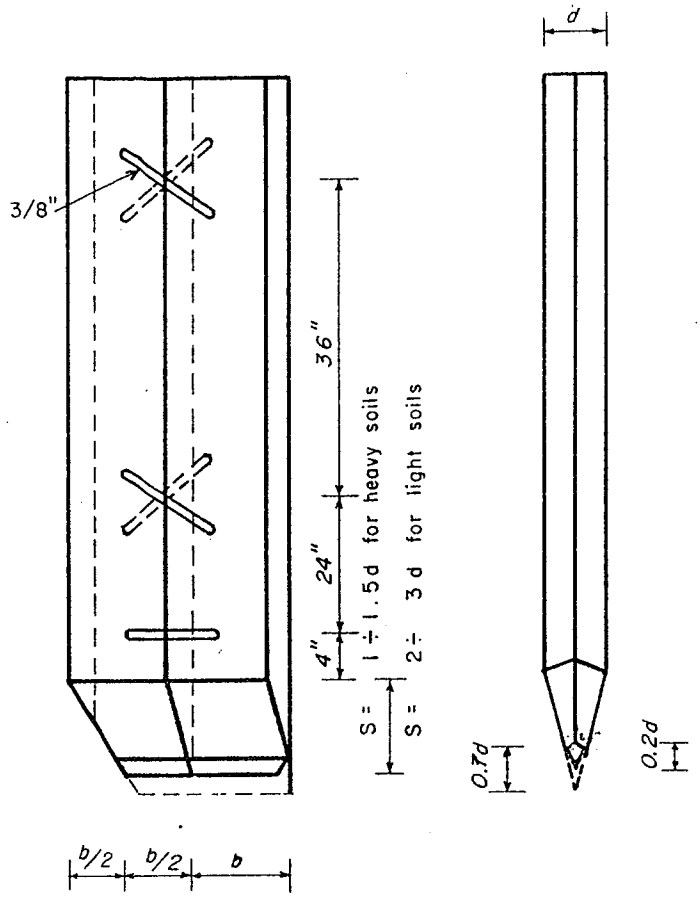
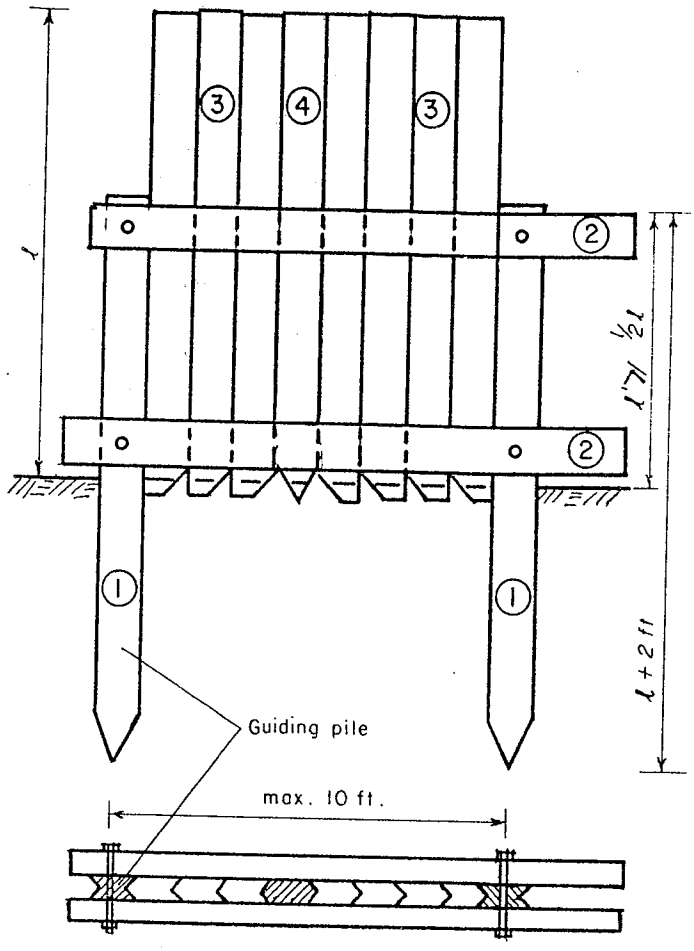


Figure II(e) Joining and sharpening of wood sheet piles

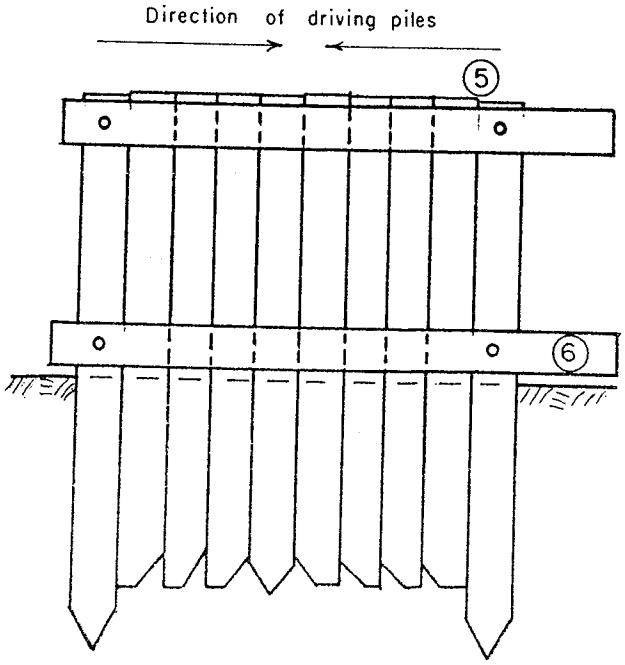


TYPE OF SOIL	DEPTH OF SHEET PILE DRIVING (ft.)	THICKNESS OF SHEET PILE (in.)
Light Sand	< 6	2.5
Sandy loam	6 – 8	3.0
Soft clay	8 – 10	3.5
	10 – 15	4.0
	> 15	5.0
Hard sand, gravel,	< 6	3.0
Heavy clay	6 – 8	3.5
	8 – 10	4.0
	10 – 15	4.5
	> 15	6.0

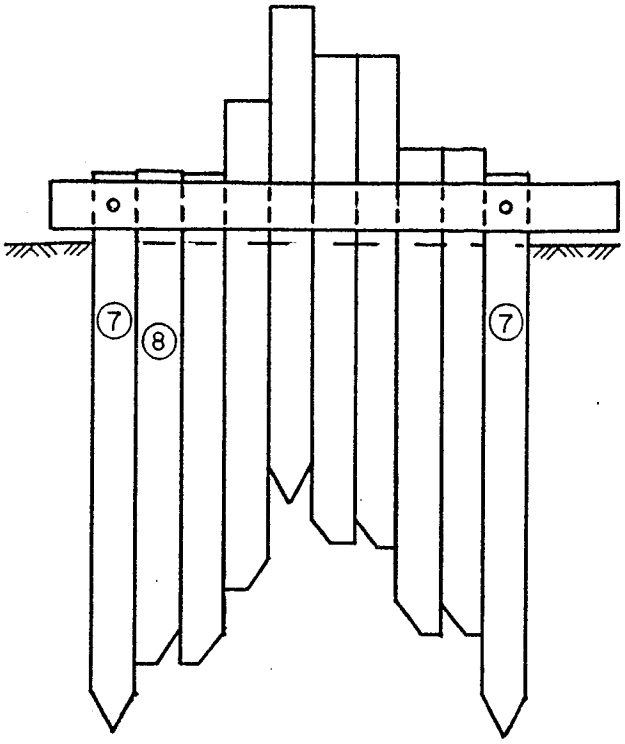
Fig. II (f) CONSTRUCTION OF WOOD SHEETPILING WALL



1. Drive guiding piles $\frac{1}{2}$ depth
2. Fasten braces (upper and lower to the guiding piles with $\frac{3}{4}$" dia. steel bolts.
3. Place sheet piles between braces with tongues towards the guiding piles.
4. Place control sheet pile (wedge) double tongued precut to the required width to fill in the remaining space.

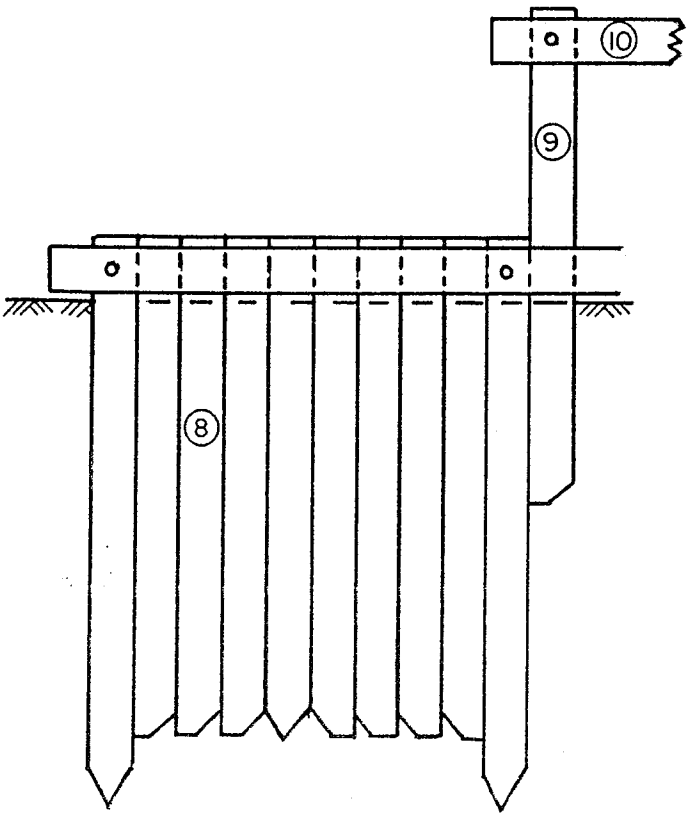


5. Drive down the sheet piles gradually, $1-\frac{1}{2}$ ft at a time, starting from guiding piles towards the center to the level of upper braces.
6. Remove the lower braces.



7. Drive down the guiding piles gradually 1-1½ ft at a time to the required elevation without removing braces.

8. Drive down the sheet piles gradually from left and right towards the center to the required elevation.



9. Drive down new sheet pile next to the guiding pile ½ depth.

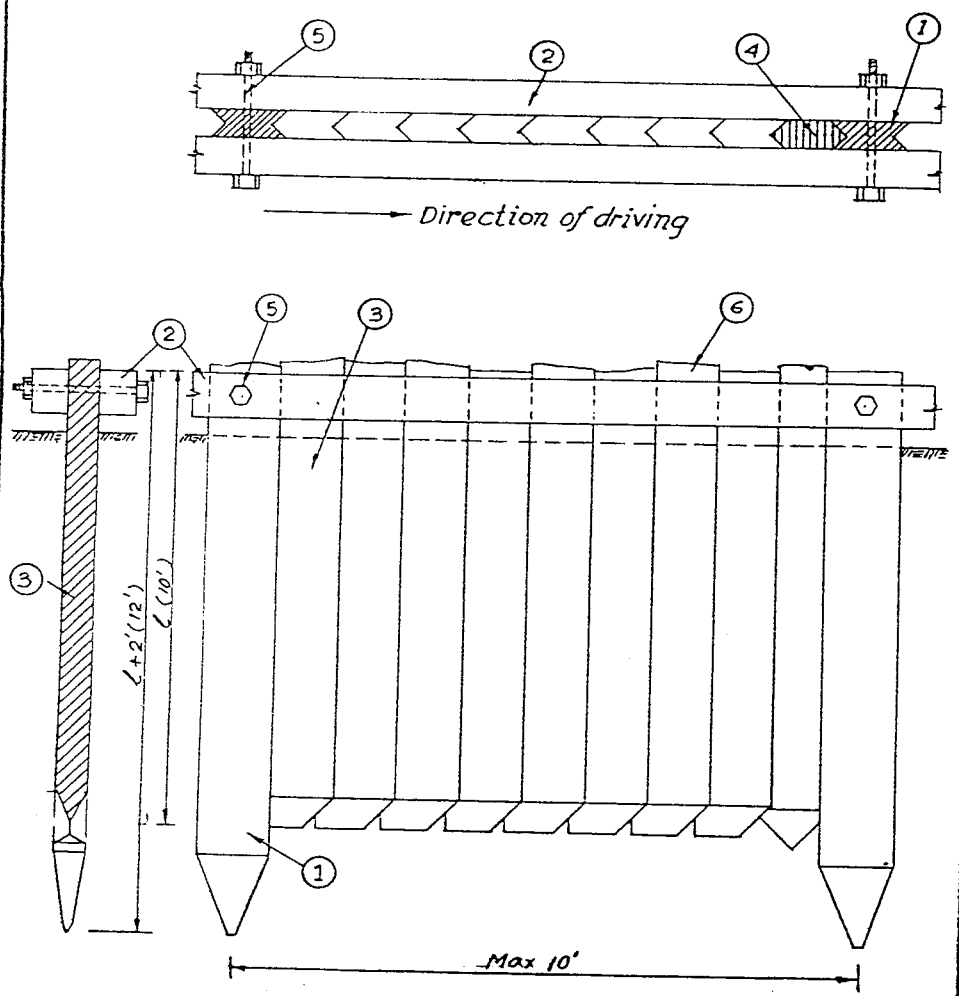
10. Fasten upper braces to the new pile (9) and next guiding pile.

. Fasten lower braces.

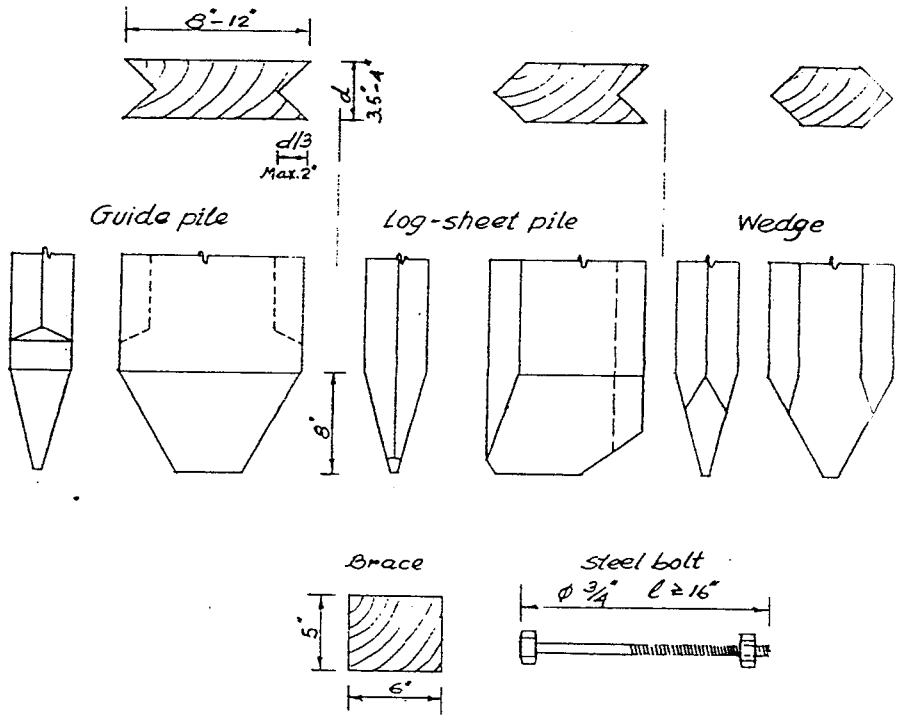
11. Drive down next section of sheet piling wall as above.

12. After the entire wall is driven down, saw off top of the sheet piles square to sound undamaged wood.

Protect heads of the piles with one coat of zinc lead paint, bitumen or pitch applied hot.



1-Guidepile, 2-Braces, 3-Logs, 4-Wedge log,
5-Steel bolt with nut, 6-To be cut off.



REQUIREMENT OF MATERIALS
FOR 10 RFT OF CUT-OFF WALL

DEPTH OF WALL	6'	8'	10'	12'
THICKNESS OF LOGS	3½"	4"	4"	4"
NO. OF LOGS:				
8" WIDE	20	20	20	20
10" "	15	15	15	15
12" "	12	12	12	12
VOLUME OF WOOD IN CFT: (1)				
LOGS 8" WIDE	29.2	42.0	51.4	60.7
10" "	27.8	39.8	48.5	57.2
12" "	26.9	38.6	47.0	56.4

(1) INCLUDING BRACES AND 5% WASTAGE

IDA SMALL SCALE DRAINAGE AND FLOOD CONTROL PROJECT BSS-BD
BANGLADES WATER DEVELOPMENT BOARD

WOODEN SHEET PILE
TYPICAL 10 FT CUT-OF-WALL

northwest hydraulic consultants ltd.
dhaka bangladesh

DESIGNED: H. Yousuf
DRAWN: T. RHK
CHECKED: G.H.A. Nabeel
DATE: 1 AUG. 1965

M. F. Rodman
CHIEF ENGINEER
CANADIAN TECHNICAL UNIT

CHAPTER IV

STRUCTURAL DESIGN

A.H. BHUIYAN

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STABILITY & STRUCTURAL DESIGN OF R.C.C. REGUALTOR/SLUICE.

1. 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 - $U_t = (1.7\sqrt{f'c})/D \leqslant 160$ psi

Bottom bar - $U_b = (2.4\sqrt{f'c})/D \leqslant 160$ psi

Shear Stress :

Flexural - 1.1 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. 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 regualtor 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 $P_n = Ca\ h$, 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

$$Ca = \left[\frac{\sin (B - \phi)}{(n + 1) \sin B}\right]^2 \times \frac{1}{\sin B} \quad \text{where}$$

$$n = \frac{\sin \phi}{\sin B}$$

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

$$P_1 = \frac{2\xi V}{L}, \quad P_2 = 0$$

When Resultant is outside middle third

$$P = \frac{2V}{3a}, \quad a = L/3$$

3. Structural Elements of a Regulator :

The type of Regulator or Sluice structure normally built in this country consits 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. 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/Returnwails 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.

Beel 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.
- 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)

<u>Loading Conditions</u> - As in Plate No. 3.

<u>Design Concept.</u>

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 as 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 $1/12\ WL^2$ 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 + 1/12 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 1/12 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

$E = 4.75S + 3.2$, where E = width of slab over which a wheel load is to be distributed and S = span length

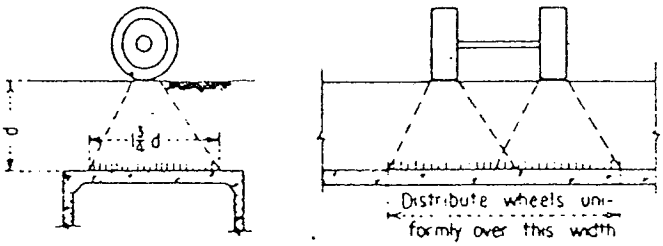
(2) Spans over 12 feet

$$E = \frac{10N + W}{4N}$$

where N = total number of lanes (@ 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.

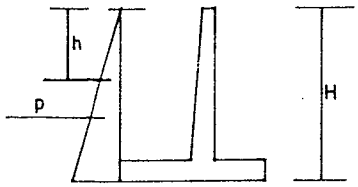
c. In no case shall the total width of distribution exceed the total width of supporting slab.

C. IMPACT

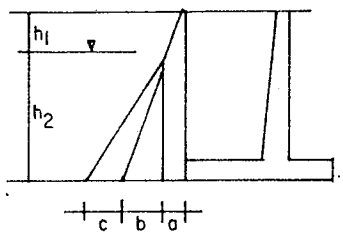
For fills	0"	to	1'-0"	Impact = 30 %
" "	1'-1"	"	2'-0"	" = 20 %
" "	2'-1"	"	2'-11"	" = 10 %
" "	3'-0" and over			" = 0 %

PLATE NO 1

Earth pressure with horizontal backfill



Earth pressure with saturated backfill (partly)



Earth pressure on inclined wall with horizontal backfill

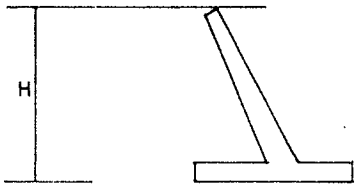
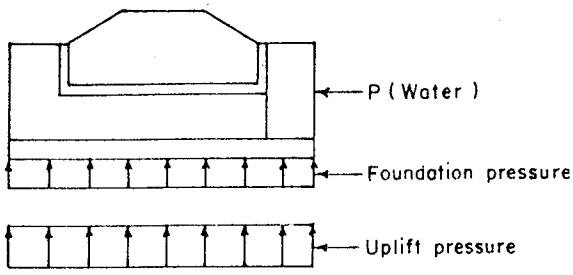
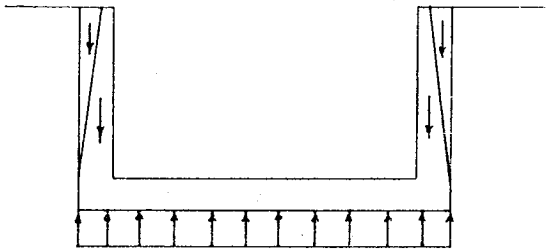


PLATE NO. 2 EARTH PRESSURE ON RETAINING WALLS

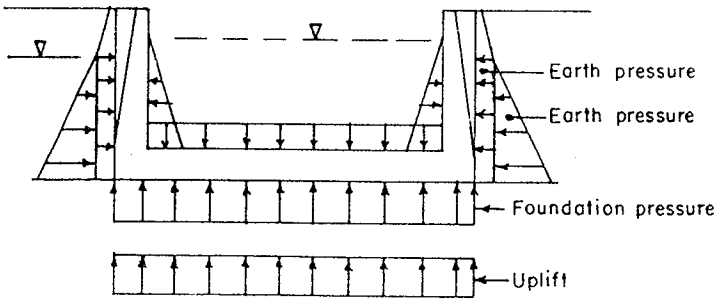
Loading condition for the stability analysis of main part of the Regulator



LOADING CONDITION – I
(Just after Construction, No Backfill)



LOADING CONDITION – 2
(During operation – Gates open)



LOADING CONDITION – 3
(During operation – Gates closed)

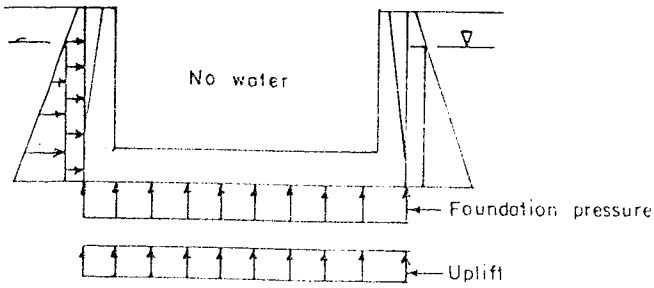
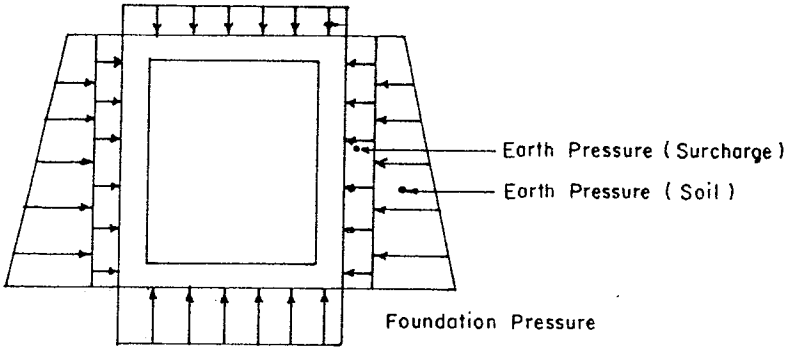


PLATE NO. 3 LOADIN CONDITION FOR THE DESIGN OF APRON SLAB & WINGWALL IN MONOLITHIC CONSTRUCTION

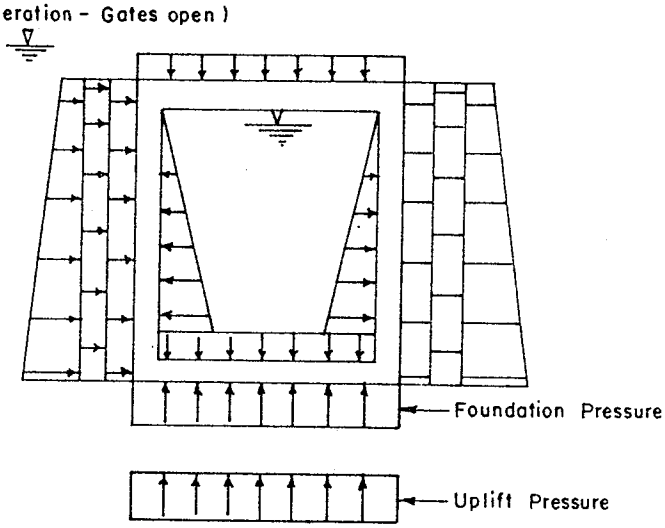
LOADING CONDITION – 1

(After Construction)



LOADING CONDITION – 2

(During operation - Gates open)



LOADING CONDITION – 3

(During operation - Gates - closed)

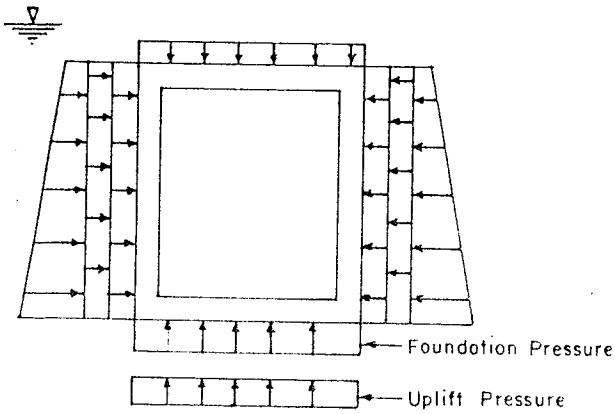
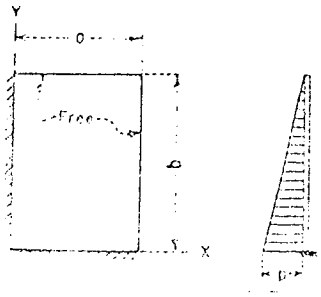


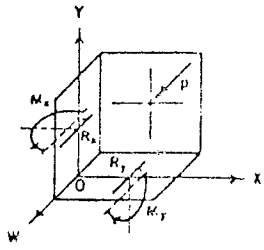
PLATE NO. 4 LOADING CONDITION FOR THE DESIGN OF BARREL

					M_x				M_y					
	y/b	R_y \ 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	+.0075	+.0007	+.0006	+.0004	+.0002	+.0001	0	0	0	0	0	0	0
	0.8	+.0052	+.0017	[illegible]	+.0006	+.0003	+.0001	0	+.0003	+.0002	+.0001	+.0001	+.0001	+.0001
	0.6	+.0050	+.0021	+.0020	+.0011	+.0005	+.0001	0	+.0006	+.0004	+.0002	+.0001	+.0000	-.0000
	0.4	[illegible]	+.0040	[illegible]	[illegible]	[illegible]	[illegible]	0	+.0009	+.0006	+.0003	+.0001	-.0000	-.0001
	0.2	+.0364	+.0052	[illegible]	+.0015	[illegible]	[illegible]	0	+.0010	+.0006	+.0002	-.0001	-.0003	-.0004
	0	[illegible]		[illegible]	[illegible]	[illegible]	+.0011	0	0	+.0007	+.0021	+.0038	+.0056	+.0072
		R_x \ R_y	[illegible]	[illegible]	[illegible]	[illegible]	+.1819	+.2706						
a/b = 1/4	1.0	[illegible]	[illegible]	[illegible]	[illegible]	+.0016	+.0007	0	0	0	0	0	0	0
	0.8	+.0056	[illegible]	[illegible]	[illegible]	+.0015	+.0005	0	+.0016	+.0011	+.0007	+.0005	+.0004	+.0004
	0.6	+.1026	+.0117	[illegible]	[illegible]	+.0015	+.0003	0	+.0024	+.0014	+.0007	+.0001	-.0004	-.0006
	0.4	[illegible]	+.0150	+.0085	+.0040	[illegible]	-.0002	0	+.0030	+.0015	+.0002	-.0009	-.0017	-.0022
	0.2	[illegible]	[illegible]	+.0050	+.0022	+.0001	-.0006	0	+.0024	+.0008	-.0006	-.0017	-.0026	-.0032
	0	[illegible]		[illegible]	[illegible]	[illegible]	+.0035	0	0	+.0024	+.0069	+.0121	+.0175	+.0221
		R_y \ R_x	[illegible]	[illegible]	[illegible]	[illegible]	+.3009	+.4232						
a/b = 3/8	1.0	[illegible]	[illegible]	[illegible]	+.0066	+.0037	+.0015	0	0	0	0	0	0	0
	0.8	[illegible]	[illegible]	[illegible]	+.0066	+.0029	+.0008	0	+.0036	+.0023	+.0012	+.0004	-.0000	-.0002
	0.6	[illegible]	+.0133	[illegible]	[illegible]	+.0018	-.0002	0	+.0047	+.0024	+.0003	-.0013	-.0026	-.0034
	0.4	[illegible]	[illegible]	[illegible]	+.0045	+.0002	-.0012	0	+.0049	+.0017	-.0012	-.0037	-.0055	-.0067
	[illegible]	[illegible]	[illegible]	[illegible]	+.0016	-.0005	-.0009	0	+.0030	+.0006	-.0014	-.0028	-.0037	-.0042
		[illegible]		[illegible]	+.0024	+.0040	+.0057	0	0	+.0044	+.0119	+.0202	+.0286	+.0356
		R_y \ R_x	[illegible]	+.0309	+.1232	+.1911	+.3079	+.4185						
a/b = 1/2	[illegible]	+.0166	[illegible]	+.0167	+.0100	+.0046	+.0013	0	0	0	0	0	0	0
	0.8	[illegible]	+.0295	+.0177	+.0090	+.0032	+.0003	0	+.0059	+.0032	+.0010	-.0007	-.0019	-.0026
	[illegible]	[illegible]	+.0336	+.0174	+.0069	+.0009	-.0013	0	+.0067	+.0027	-.0010	-.0041	-.0064	-.0079
	[illegible]	[illegible]	+.0309	+.0125	+.0037	-.0011	-.0021	0	+.0062	+.0012	-.0032	-.0067	-.0092	-.0109
	[illegible]	[illegible]	+.0162	+.0058	+.0010	-.0005	-.0004	0	+.0032	+.0004	-.0012	-.0018	-.0018	-.0017
	0	-.0079	0	+.0015	+.0039	+.0064	+.0089	0	0	+.0074	+.0193	+.0318	+.0445	+.0546
		R_y \ R_x	-.0079	+.0573	+.1665	+.2446	+.3698	+.4827						
a/b = 3/4	1.0	+.0974	[illegible]	+.0269	+.0107	+.0008	-.0028	0	0	0	0	0	0	0
	[illegible]	[illegible]	+.0487	+.0242	+.0084	-.0002	-.0027	0	+.0097	+.0040	-.0004	-.0039	-.0065	-.0087
	[illegible]	[illegible]	+.0459	+.0195	+.0046	-.0023	-.0036	0	+.0092	+.0020	-.0044	-.0093	-.0129	-.0155
	0.4	[illegible]	+.0300	+.0121	+.0010	-.0030	-.0029	0	+.0072	-.0003	-.0060	-.0096	-.0117	-.0132
	0.2	[illegible]	+.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	+.0896
		R_x \ R_y	-.0194	+.1105	+.2399	+.3236	+.4489	+.5505						
a/b = 1	1.0	+.1917	+.0662	+.0291	+.0056	-.0059	-.0077	0	0	0	0	0	0	0
	0.8	+.2461	+.0613	+.0243	+.0035	-.0056	-.0063	0	+.0123	+.0039	-.0019	-.0059	-.0089	-.0118
	0.6	+.2364	+.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						



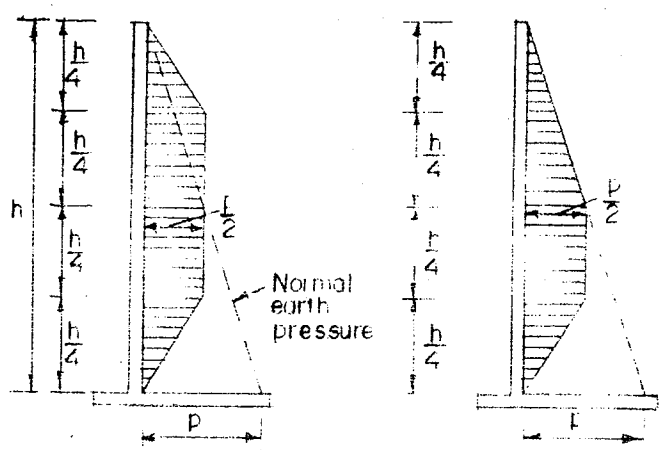
Moment = (Coefficient)(pb²)

Reaction = (Coefficient)(pb)



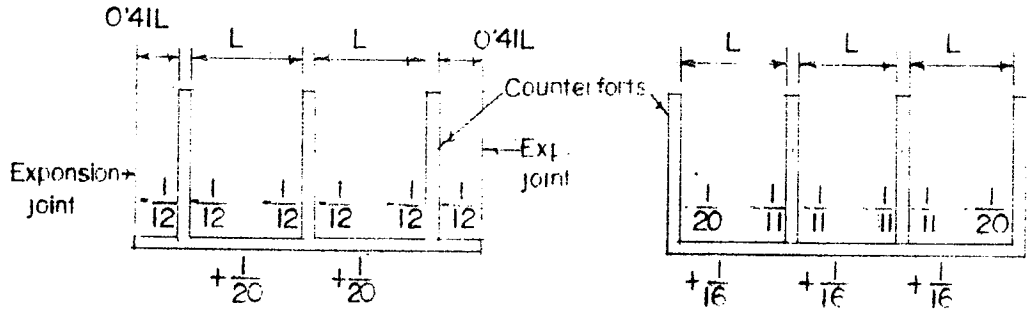
POSITIVE SIGN CONVENTION

PLATE NO.5- [illegible] fixed along two adjacent edges, moment and reaction coefficients, Load uniformly varying [illegible]



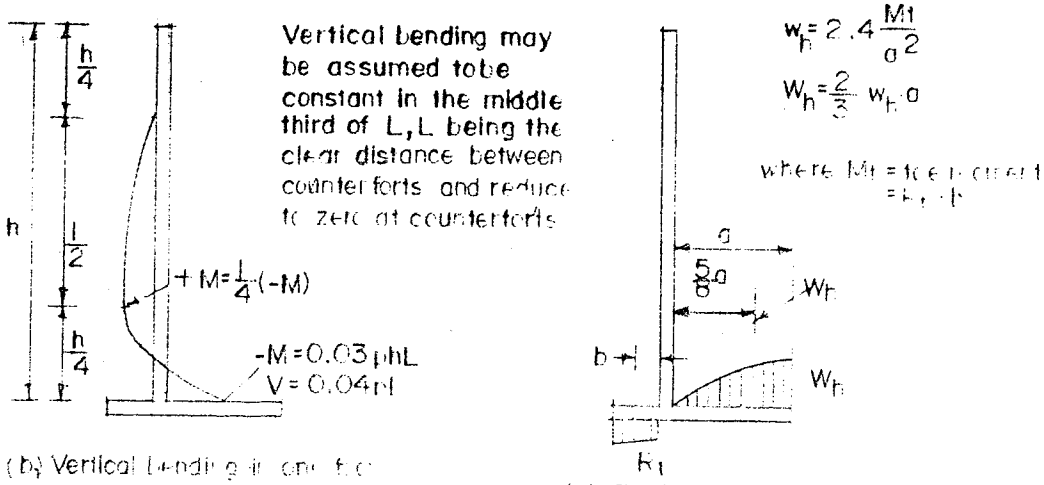
Pressure used for positive moment computation

Pressure used for negative moment computation



Moment coefficients used in conjunction with the above earth pressures

(a) Horizontal bending in the face slab



(b) Vertical bending in one foot strip of face slab

(c) Equivalent load (downwards) on the slab due to moment [illegible]

PLATE NO 6 HUNTINGTON'S DESIGN PROCEDURE FOR COUNTERFORT WALL

CHAPTER V

FOUNDATION DESIGN

A. Azam.

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1.0 BACKGROUND :

Bangladesh Water Development Board is associated with 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 were constructed along with bridges of long and short spans. It may be mentioned here that BWDB has already implemented about 8,500 sluices, regulators and bridges. In addition, it has constructed 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 problems of foundation are encountered in many cases. During the early period of implementation of Coastal Embankment Project so 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 other than bore-logs no information of soil parameters are supplied. As such, decision of foundation treatment becomes very difficult.

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1.1 GENERAL TYPE OF SOIL :

In the northern part of Bangladesh foundation soil is better and generally sands are found at shallow depth for most of the cases. The foundation soil is in a position to take care of superstructure load in most of the cases and as such special treatment is not required for normal structure. Moreover, these parts has no tidal effect and soil at foundation level is normally above ground water table. In the coastal zone 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 80'- 0" depth. 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 continous 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 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 conrete 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 active 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 recongnized 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.

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 foundatin 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 considerable below the lower part of the superstructure. Obviously there is no sharp dividing line between the two classes. Generally foundtion that has a width "B" equal to or greater than the depth of the foundation "D" is considered a shallow foundation. Shallow foundation include 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.

c) 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 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.

i) When the soil is of the expansive type a raft foundation or short piles foundation may be contemplated. Total and differential settlemnts 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

2.6 SAND PIER 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 founmdation 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 behafiour 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.

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 $qd = CNc + YDf\ Nq + 1/2\ YB\ NY$.

Where C = cohesion of soil

NC, Nq & NY are dimensionless bearing

Capacity factors that depends on "O" (Angle of internal friction). Terzaghi gives the values of Nc, Ny, Nq 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 Nc, Ny and Nq. 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"

$$q = 1.2\ C\ Nc + \gamma DNq + 0.3\ \gamma\ B\ N\gamma$$

Square footing of size "B"

$$q = 1.2\ C\ Nc + \gamma DNq + 0.4\ \gamma\ BN\gamma$$

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.

$$q = C\ Nc\ Sc\ dci\ c + Df\ Nq\ Sq\ dq\ iq + 1/2\ B\ N\ S\ D\ i$$

Where c = cohesion of soil
γ = surcharge
d = depth of factor
i = incination 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 & ' ', 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 is not strong enough to withstand the load of a small structure, lot of consideration regarding foundation is to be taken into account. Soil conditions at a particular site are not generally ideal and simple economic foundation 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 medium dense sand, 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. While studying all of these 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 interfere 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 it comparitively a light structure, 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 cohesoinless 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 :

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 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 DB 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

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

$$qd\ net = 5L(1 + 0.2\ Df/B)(1 + 0.2\ B/L)$$

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 'qa' 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.

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. 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.

$$S = \frac{H}{1 + eo} \; Cc \log_{10} \frac{Po + \Delta P}{Po}$$

where S = Settlement of the layer in cm

ΔP = 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

cc = compression index of soil layer

Value of Δp 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 $\frac{\Delta P}{(Po' - Po)} > 1$ 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 & 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 AND PIERS :

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 :

$$Qult = \pi R^2 (\gamma DNq + 0.6 R\gamma N\gamma) + 2\pi Rhfs \text{ (for round pile)}$$

$$Qult = R^2 (\gamma DNq + 0.4 B\gamma N\gamma) + 4Bhfs \text{ (for square pile)}$$

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 :

$$Qult = 2\pi RL(\gamma Z + q) K \tan \phi \text{ (round pile)}$$

or $$Qult = 4 BL(\gamma Z + q) K \tan \phi \text{ (square pile)}$$

where L = total length of embankment 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

$$Eg = 1 - \frac{(n-1)m + (m-1)n}{90\ mn.}$$

where m = number of columns of piles in a group.

n = number of rows
= 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 ' P' beneath pile group when the froup 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 stnd 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	Clay	6
Footing	Sand	4
Raft	Clay	6 - 10
Rigid Box Type	Sand	4 - 6
Structure	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 can not be managed. 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 as pre-compressing 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 other data are required before giving any decision. Excessive settlement in pier-part may cause P.V.C. to crack resulting ultimately 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) Ehere 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.

17.0 EXAMPLE :

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. (fig.1)

Assume the following parameters

eo = 1.28, Load/sft. = 1.0 tons
Cc = 0.350. Size of structure 25' x 35' (B x L)
γ = 115 lb/ft^3

a) check whether the soil is pre-loaded.
b) Find out settlement.
c) Upper sand layer is 15'-0" deep.

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 x 12 + (115 - 62) x 26 = 2698 lb/sft.

As Po' < Po, it is not pre-loaded.

Net load/sft = 2000 - γDf = 362 lb

P at centre of compressive layer

$$= \frac{0.362 \times 25 \times 35}{(25 + 20)\ (35 + 20)} = 0.128\ K$$

$$S = \frac{Cc\ H}{1 + e_o} \log_{10} \frac{\Delta P + Po}{Po}$$

$$S = \frac{0.32 \times 120}{1 + 1.28} \log_{10} \frac{128 + 2698}{2698}$$

= 18.42 x 0.020 = 0.37 inch.

Beyond central part, load is approximately 0.375 T/sft.

No net pressure, settlement will not occur

Diferential settlement = .37 inch < 1.50 inch

So the structure is not likely to create problem.

Example 2. Same soil condition. Consolidation curve as in Fig. 2.

Po' = 1.8 Ton/sft.
Cc = 0.188
eo = 1.05

Po = 2698 lb/sft. Po < Po'

So the soil is pre-consolidated. So there will be less settlement than calculated. Use Po' instead of Po.

$$\text{therefore } S = \frac{0.188 \times 120}{1 + 1.05} \log_{10} \frac{.064 + 1.88}{1.88}$$

$$= 11.00 \times 0.015 = 0.16 \text{ inch.}$$

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 3.

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.

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.

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 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.

$$q_{ult} = 4N^2 B\ RW + 12\ (100 + N^2)\ D\ R'W,$$

$$N = 7\ (min),\quad d = 20' - 0"$$

$q_{ult} = 4 \times 49 \times 10 \times .5 + 12\ (100 + 49) \times 20 \times 0.50$

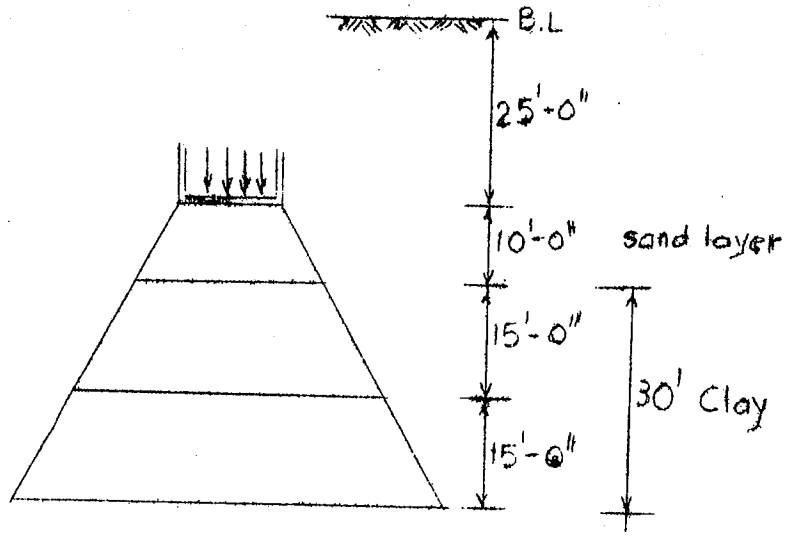
$= 18.86$ K

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) = 2.65 K/sft.
eo = 1.53
Cc = 0.58

$$\Delta P = \frac{185.00}{(10 + 25)\ (15.67 + 25)} = 0.13 \text{ K/sft.}$$

$$S = \frac{0.58 \times 30 \times 12}{1 = 2.53} \log \frac{2.60 + 0.13}{2.60}$$

$= 82.53 \times .0211 = 1.75"$

Without Liveload ΔP = 0.113 K therefore S = 1.52 inch

At first support load is 135 K.

ΔP = 0.094 lb/sft.

S = 82.53 x .0156 = 1.28 inch

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 4.

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 and 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" = 541.4 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.

Full DL + LL : 1.82 ^{k}/sft.

SPT value - minimum 5

Ó value - 28° (from table given, use it) , if the test values of o is

C = 0

From Hansen's bearing capacity equation for foundation on granular soil :

$$q_{ult} = 0.5\, B\, \gamma_s\, N_\gamma\, S_\gamma\, d_\gamma + \gamma_{moist}\, D_f\, N_q\, S_q\, d_q$$

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_γ = 14.72 (from Table) ( ) N_γ = 16.72

S_γ = 1 - 0.4 (b/L) = 1 - 0.4 (14.25/38.0) = 0.85

d_γ = 1.00

D_f = 12.0

S_q = 1 + (B/L) tan ϕ = 1 + (14.25/38.0) tan 28° = 1.20

d_q = 1 + 2 tan ϕ $(1 - \sin \phi)^2$ (D/B) (D<B)

= 1 + 2 tan 28° $(1 - \sin 28^o)^2$ (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 x 14.25 x 52.6 x 16.72 x 0.85 x 1.00 + 110 x 12.0 x 14.72 x 1.20 x 1.25

= 5326 x 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 full DL + LL condition

$$= 1.82\ ^{k}/sft - Df$$

$$= 1.82\ ^{k}/sft - 0.110 \times 12.00$$

$$= 1.82 - 1.32 = 0.5\ ^{k}/sft$$

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 die 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. 5.

<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

$$\frac{qult - df}{F} = \frac{34472 - 12 \times 110}{2} = 16576 \text{ lb}$$

$$= 16.58 \text{ k/sft.}$$

$$> 1.82 \text{ k/sft. O.K.}$$

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 faialure and also from determental settlement.

Example. 6.

<u>Example for foundation analysis on cohesive soil</u> : The same structure with the applied load if now requires to be constructed on a soil with clay and silt on top with fine sand in bottom and 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

$C_c = 0.20$

$e_o = 0.40$

<u>Bearing capacity</u> :

For this type of soil ($\phi = 0^{o}$),

$$qult = 5.140\ (1 + S'_c + d'c) + \gamma\ Df$$

where, $C = 815$ psf

moist = 10 lb/sft.

$Df = 12' = 0'$

$$S'c = 0.2B/L = 0.2 \times \frac{14.25}{38} = 0.075$$

$$d'c = 0.4\ D/B = 0.4\ \frac{12}{14.25} = 0.337$$

$$Qult = 5.14 \times 815 (1 + 0.075 + 0.337) + 110 \times 12 = 7235 \text{ lb/sft.}$$

$$Qnet = \frac{Qult - \gamma Df}{F} = \frac{7235 - 110 \times 12}{3} = 1972 \text{ lb/sft } 500 \text{ lb/sft. O.K}$$

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

$$= Po = 12 \times 110 + 14.00 (115 - 62.50)) = 2055 \text{ lb/sft.}$$

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.

$$\text{So} \quad P = \frac{0.47 \times 14.25 \times 38}{(14 + 14.25)(38 + 14)} = 0.173 \text{ k/sft.}$$

$$\text{therefore new } S = \frac{(0.20 \times 28 \times 12)}{1 + 0.40} \log \frac{2055 + 173}{2055}$$

$$= 48 \times 0.0355 = 1.70 < 2''$$

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.

$Cc = .2$
$eo = 0.92$
$qu = 51.08 \text{ km/m}^2$

Net foundation pressure

$= 1.82 \text{ s/sft} - 0.11 \times 10$
$= 1.82 - 1.10 = 0.72 \text{ k/sft.}$

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.

PROBLEM - 6.

A bridge of span 120 ft. is located in a place whose foundation is very poor (bore log enclosed, Fig. 11). There is no alternate site design the foundation.

Solution : As the soil is poor the bridge is designed as consisting of simply suported span. Any settlement if occurs will affect the structure to leser degree than continuous bridge.

Load at central column is 190 K.

Soil parameters : N value 4

qu = o.45 T.sf : C = 450 k/sft.

Av. Pile dia = 8".

Length of pile = 40 ft.

Timber pile will be used by splicing. Allowable cohesion = 150 lb/sft.

Individual action :

3.14 x 150 x 40 = 12.62 kip.

Number of pile required = 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 1500 = 288 K. > 190 K.

GROUP EFFICIENCY FACTOR :

$$Eg = 1 - \frac{0(n - 1)m + (m - 1)n}{90\ mn}$$

$q = \tan - 1\ d/s$ therefore $0 = 11.64^{o}$

$$Eg = 1 - \frac{11.64\ (3 - 1)6 + (6 - 1)3}{90 \times 3 \times 6}$$

$$= 1 - \frac{139.68 + 15}{90 \times 18}$$

$= 1 - .095 = 0.905.$

therefore q allowable = 12.62 x 18 x .905 = 205 k. > 190 K.

So the pile design is OK.

SETTLEMENT :

As is is located in homogeneous clay, load is assumed to distribute at 2/3rd from top of pile.

Thickness of clay layer = 20 ft.

Depth of center of layer = 24 ft. from 2/3 pile depth.

$$\text{therefore} \quad P = \frac{190\ K}{(12 + 12 + 7)\ (12 + 12 + 17)} = \frac{190}{31 \times 41}$$

$$= 0.150\ k/sft.$$

$$eo = 1.10$$

$$Cc = 0.28$$

The whole layer is considered homogenous and settlement is considered in one layer only.

$$Po = \gamma H\ O = \gamma(26 + 17) = 53 \times 42 = 2226\ lb/ft.$$

$$S = \frac{H}{1 + eo}\ Cc \log_{10} \frac{Po + \Delta P}{Po} = \frac{20 \times 12}{1 + 1.10}\ 0.28 \log \frac{2226 + 150}{2226}$$

$$= \frac{240}{2.10} \times 0.28 \log\ 1.067 = \frac{240}{2.10} \times 0.28 \times .028$$

$$= 0.896\ \text{inch.}$$

This is actual settlement. Adjoining column may also settle. So it is considered stable from settlement consideration.

Example. 7.

In Khondolia bridge whose soil is shown in bore logs 9 is very poor. The span of the bridge is 50'-0". Middle span is provided 20'-0" ft. From consideration of friction only, pile foundation was provided.

From economic consideration in above cases timber piles were used. In all the above cases length of timber piles were increased by joint. This method of providing joint is popular in USSR. However, it is better to keep driving records and check by dynamic formulae.

18.0 GROUND TREATMENT :

When it is found from analysis that soil may not be suitable to withsrand the load to which it is being subjected, alternate site may be found out. But in case where it is not possible to change site or to get a better soil, question of alternate foundation design arises. When it is known that the ground conditions are likely to prove difficult for overall or part of a particular site, the engineer can either modify the structure, or he can make use of foundation engineering techniques such as piling or raft etc. or he can modify the ground either temporarily or permanently.

In case of deep foundation sites both timber or R.C.C. piles may be provided or caisson can be used. But as we are dealing with small structures, the question of economy arises. a good foundation engineer should be able to select the foundation i such a way that not only it will be stable, but be economical too. There are many methods to improve the soil. If soil can be improved artificially, the structure can be built over it directly. There are many ways to improve the soil artificially and they have been successfully practised in many parts of the

world. But these system may not be possible in case of small hydraulic structures being constructed in isolated places.

18.1 BASIC IMPROVEMENT CRITERIA :

To improve a soil means its bearing capacity is to be increased. Soil has wide variation in characteristics. With the distribution of grain size and voids its properties changes. For cohesionless soil if density can be increased, the physical properties are also increased. In case of fine grained soil, if moisture content is reduced by releasing pore water, it becomes very hard from original soft character. The objective of the improvement is to improve the strength by artificially changing some parameters of soil as discussed above.

18.2 METHODS :

There are many methods for improvement of soil. They may be done by physical methods, mechanical methods or chemical methods. Of the physical methods pre-consolidation is popular.

Soil improvement :

Densification - a) Surface compaction
b) Deep compaction

Drainage - a) Pre-loading (consolidation)
b) Vertical drains

Reinforcement - a) Addition of chemicals such as lime, cement etc.
b) Granular piles (or stone columns)

Preconsolidation.

In this method improvement of foundation is done by pre-consolidation of soils with a pressure equal to or greater than the one exerted by a structure. Principle of consolidation is applied to understand the method. The rate of consolidation settlement depends on time, the method is accelerted by artificial means so that the chance of settlement may be eliminated. Methods pf accelerating the consolidation is pre-loading. Pre-loading consists of applying a load to the ground equal to or greater than proposed foundation loading, so that the settlement of the ground will be completed jbefore the structure is erected.

This method is most suitbale for loose granular soils or granular fills where settlement is rapid. It is also suitable for treating saturated clay soils. Pre-loading is most economical over a large area where a surcharge material cana be provided in bulk. care must be taken while applying the technique of pre-loading to saturated clays. For example, shear faillure may occur if the application of pre-load is too rapid. This is because pore-pressures are slowly dissipated in clay soils, hence consolidation is lsow. To increase the effect of this surcharge and further accelerate consolidation it is necessary to reduce the drainage paths within the soil by providing granular drains usually spaced 1.5 m and diameter of granular column 0.3 m. General specification is spacing = 3d (minimum), where d = dia of granular drain.

Sand Drains.

Normal method of introducing drainage channel into a mass of soil by the construction of sand drains. This has been used for more than fifty years. The traditional type of sand drains consists of vertical bore-hole sunk through the compressible layer of soil to a relatively firm base and back filled with a relatively clean coarse sand. There are instances where drains have exceeded 30 m in depth. The diameter of sand drains varies from approximately 300 to 600 mm, in other words a sand drains need only be large enough to drain away expelled water and to allow for margin of safety against clogging. The distance between individual sand drains is influenced by the permeability of the soil but generally they are spaced somewhere between 2 and 5 meters centres. What is more, their spacing should be less than their depth.

The application of pre-loading has been done for design and construction of embankments in san Diego. The thickness of compressive layer ranges from 2 to 40 ft. The heights of embankment ranges from 11 to 90 ft.

Groundwater Lowering.

The groundwater lowering process which depends on pumping are essentially temporary remedies, but if the terrain is suitable, permanent local control of the water table can be achieved by drainage ditches, counterfort drains. Due to the lowering of water table, effective load on the soil is increased causing

compression and consequent settlement. Pumping from an aquifer containing soft clay overlain by compressible soils may cause significant settlements.

The amount of settlement undergone depends on the thickness of compressible layers and their compressibility, as well as on the amount of groundwater lowering. the permeability of soil and the length of pumping period influence the rate of settlement.

Dynamic Consolidation.

Dynamic consolidation is "the improvement in mechanical properties of a soil by the repeated application of very high intensity impacts to the surface". This is achieved by dropping a large weight, typially 10 to 20 tons from a crawler crane from heights of 15 to 40m. at regular intervals. The high intensity stresses developed by tamming can penetrate deeply into the ground enabling compaction to take place to depths as much as 12 m.

As far as coarse granular fill is concerned more energy is required to overcome the possiblity of bridging action than is necessary for similar depths of finer material. The total energy imparted varies from 200 to 400 Tm/m2.

In addition there are other methods such as :

a) Compressed air
b) Heat treatment
c) Grouting
d) Earth-reinforcement
e) Cement stabilisation
f) Lime stabilization
g) Bitumen stabilization
h) Vibro-compaction

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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 <0.2	N <4	Qc <20	Qd <30.0
Very loose	0.2 - 0.4			30° - 35°
Loose	0.4 - 0.6	4 - 10	20 - 40	35° - 40°
Semicompact	0.6 - 0.8	10 - 30	40 - 120	
Compact	70.8	30 - 50	120 - 200	40° - 40°
Very Compact		> 50	> 200	> 45°

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		Observed Settlement, in inches
					One Dimensional Consolidation Unmodified	One Dimensional Consolidation Modified For A Factor + Elastic Compression	
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 Desicoated) Over Med. Dense Sand and Silty	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		Observed Settlement in inches
					Hough[29] empirical method based on N value	Terzaghi[30] plate bearing test extrapolation correlated to N values	
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)

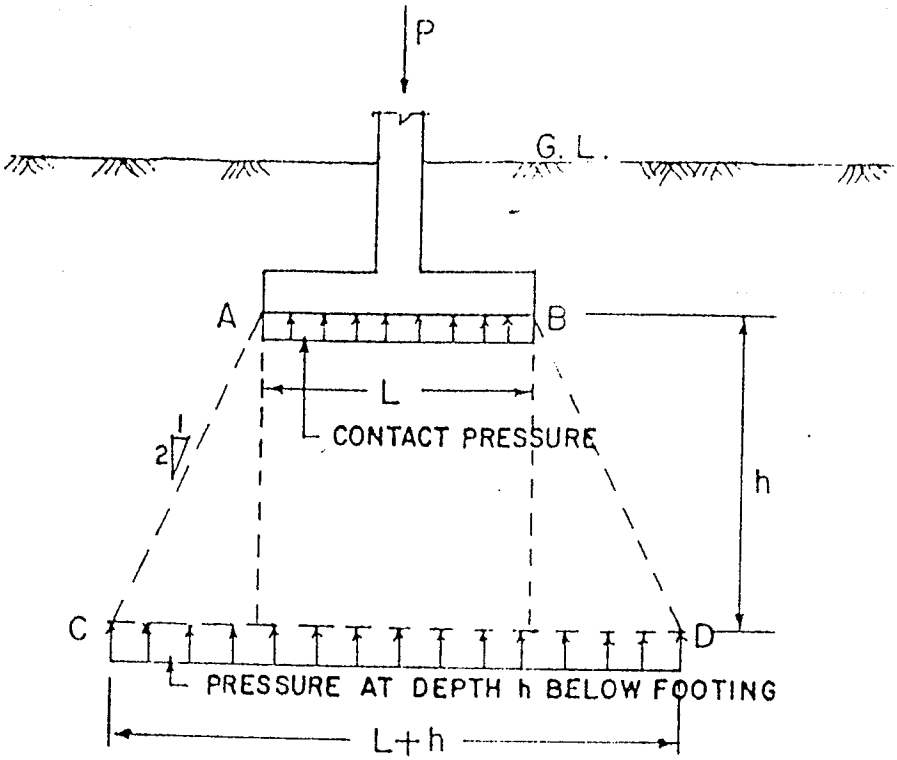


Fig. 1: Approximate Distribution of Pressure within Soil Mass under Footing

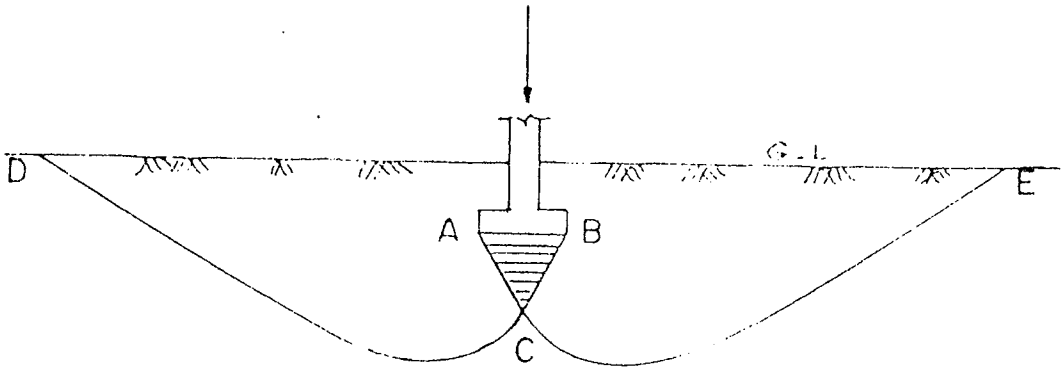


Fig. 2: Shear Failure of Soil under a Footing

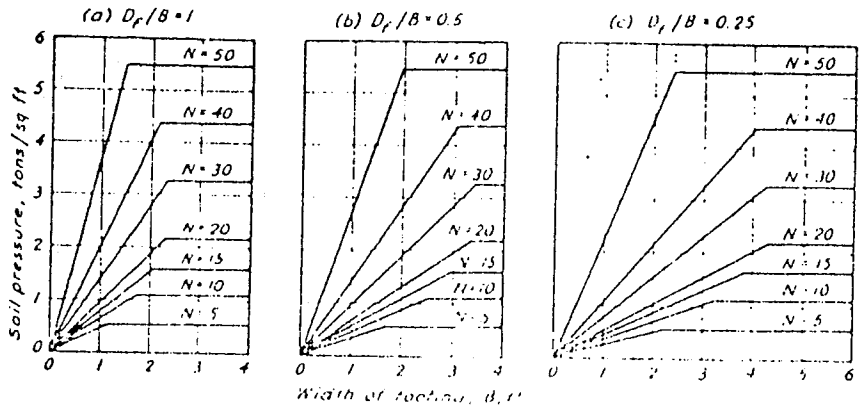


Fig. 5: Allowable Bearing Pressure of Footing on Sand from Settlement Consideration

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.

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

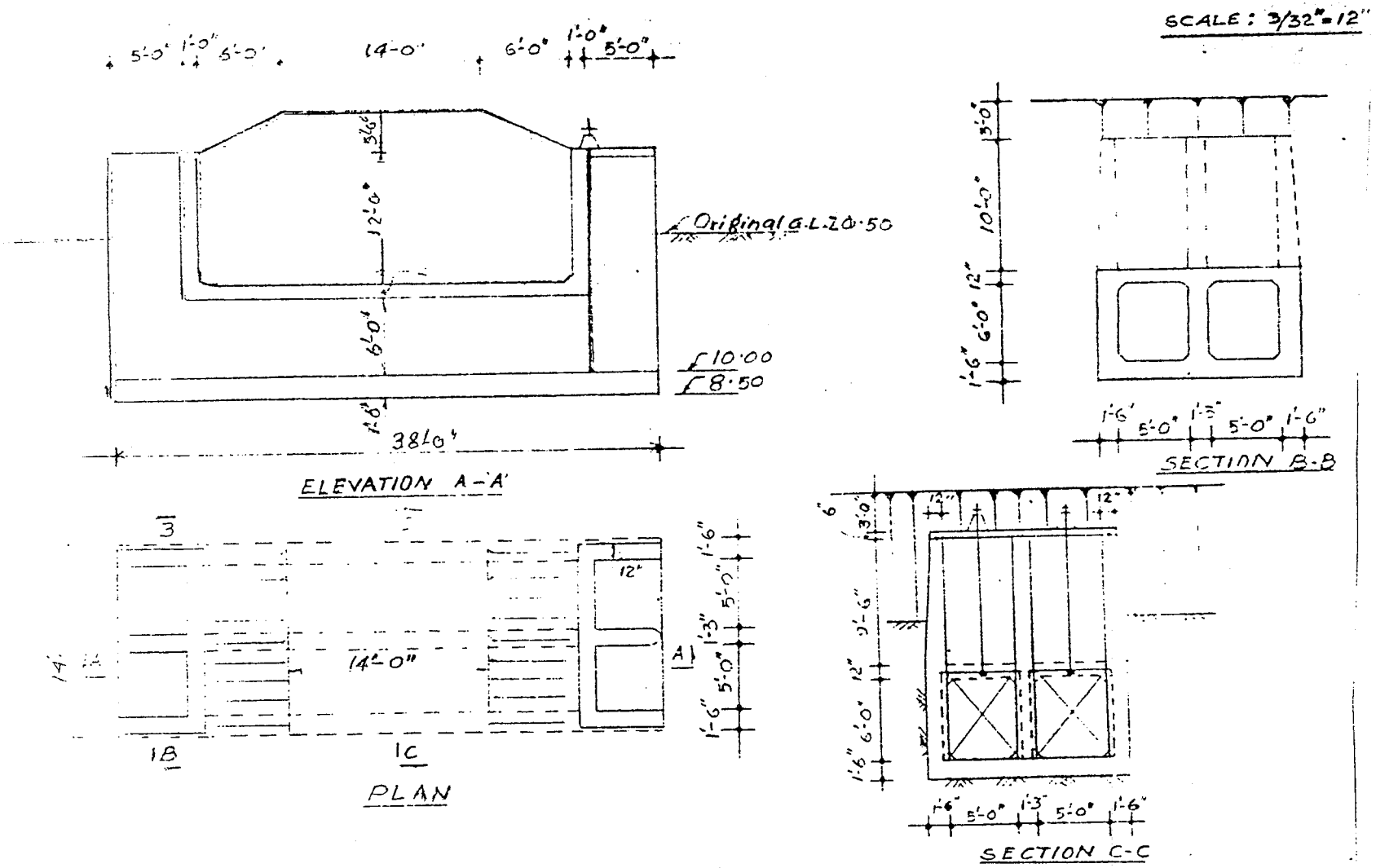


Fig. 6: Main Part of a Typical 2 Vents (5' - 0" x 6' - 0") Sluice

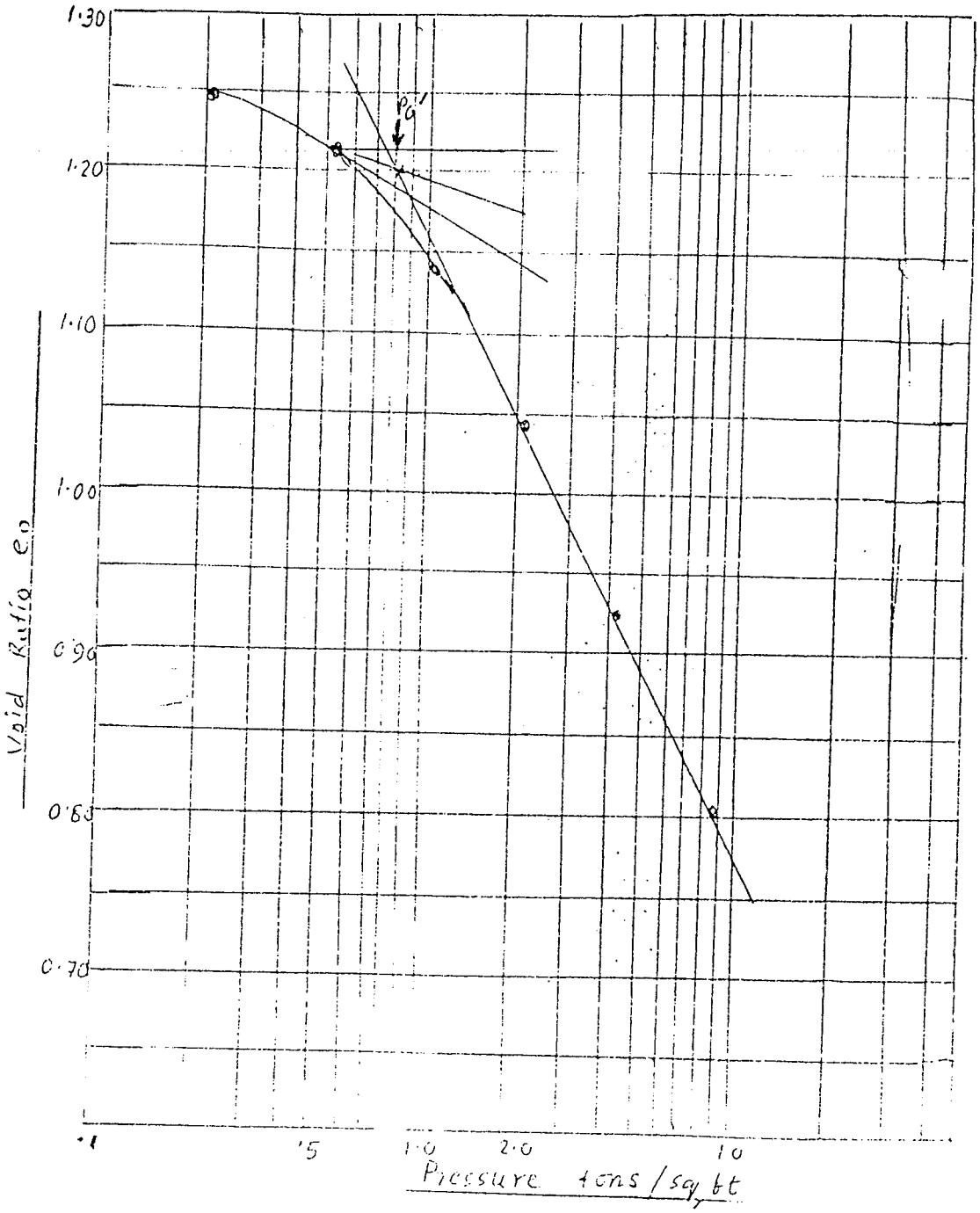
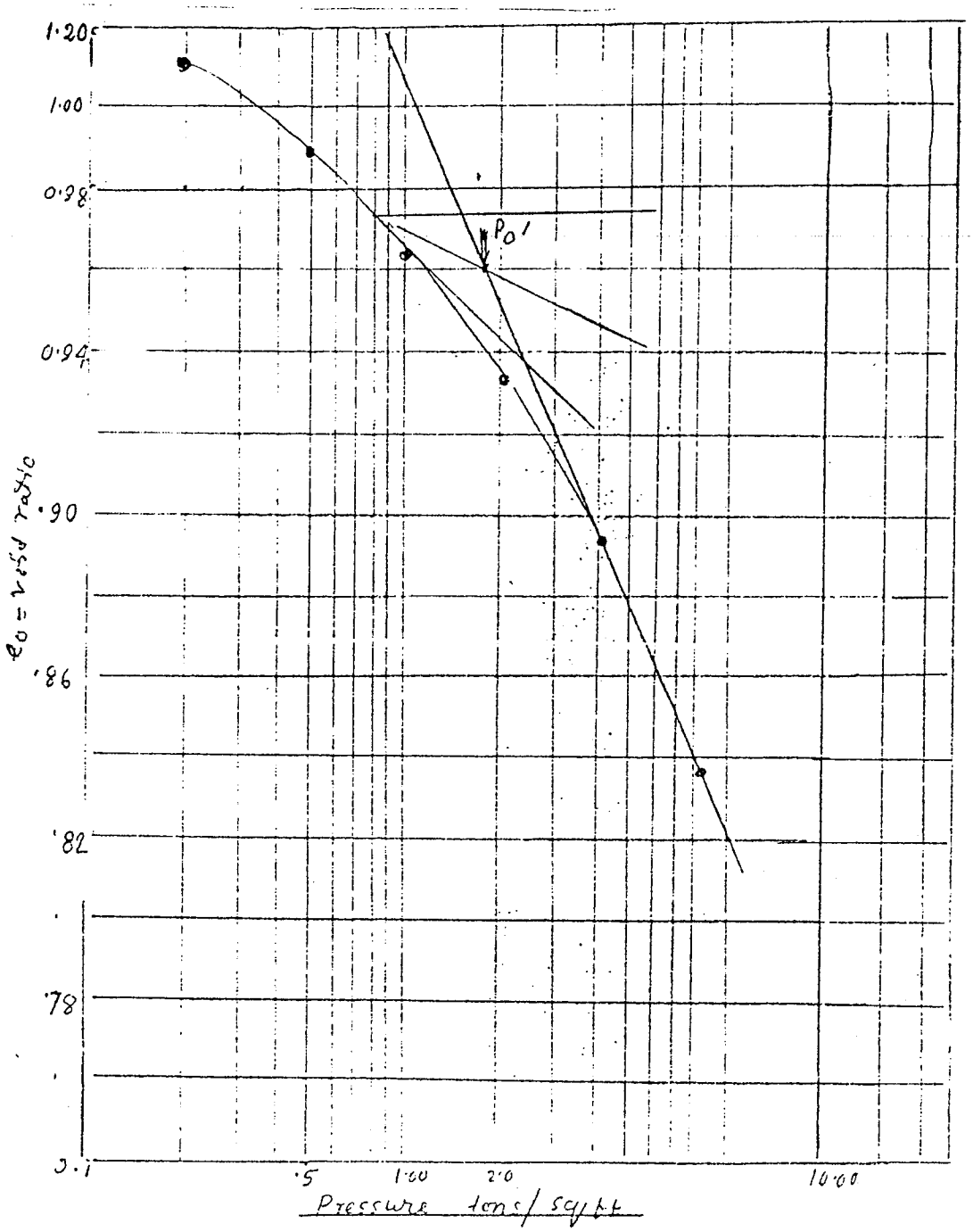


Fig. 7

Po' = 0·80 ton/sqft

Cc = 0·32

e0 = 1·28



$P_0' = 1{\cdot}80$ ton/sq ft

$C_c = 0{\cdot}188$.

$e_0 = 1{\cdot}05$.

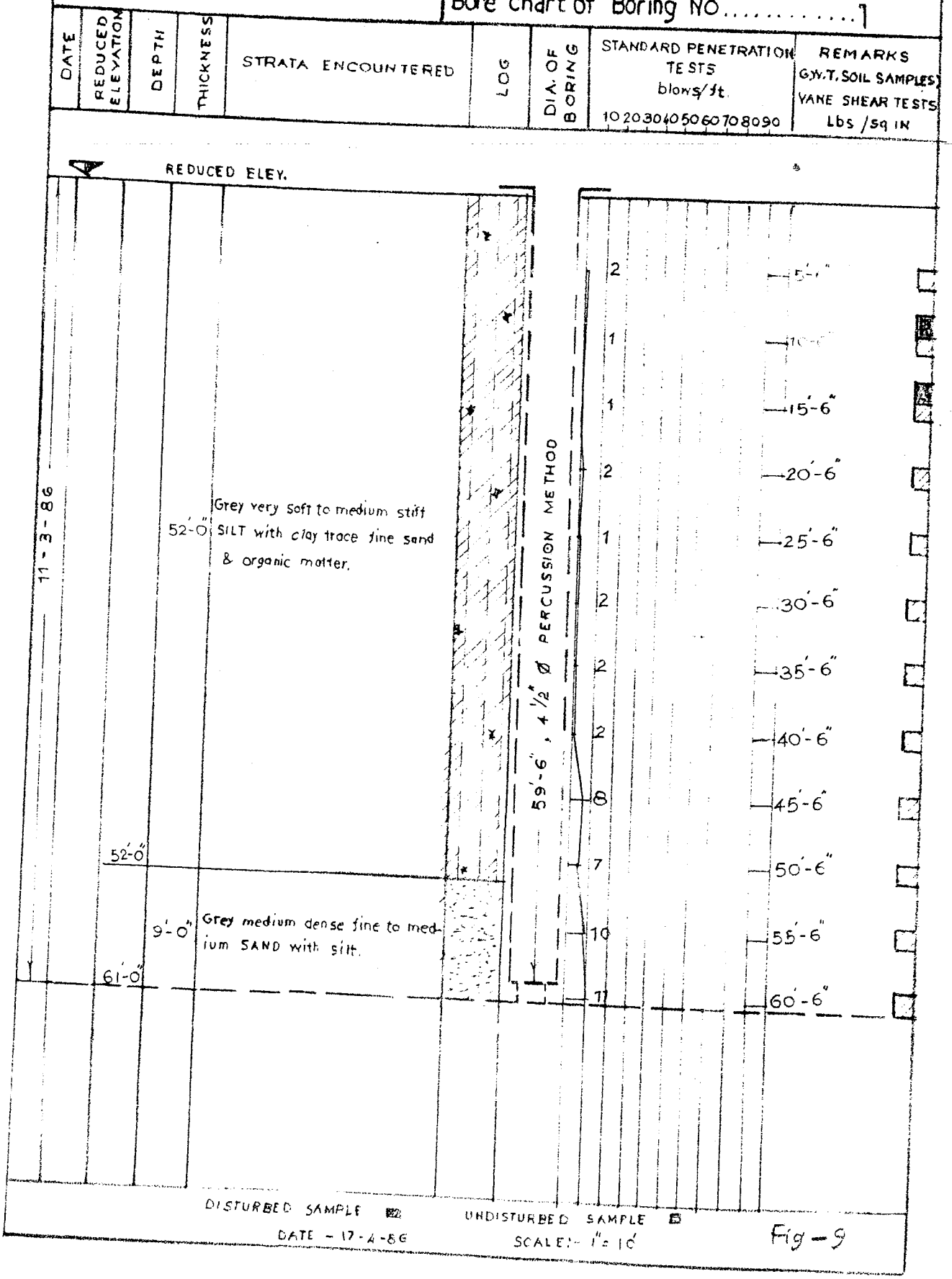
Fig. 8

SOILTECH
INTERNATIONAL LIMITED
DHAKA

Client:- W D B O & M DIV. KAPTAI

Site:- KHANDAKIA (RIVER BED) HATHAZARI UPA-ZILLA, CHITTAGONG

Bore chart of Boring NO.....1



DISTURBED SAMPLE ▨ UNDISTURBED SAMPLE ▥

DATE - 17-4-86 SCALE:- 1" = 10'

Fig - 9

BANGLADESH WATER DEVELOPMENT BOARD
GROUND WATER DIVISION - 1
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 bank AT HRS OF DATE
TOTAL DEPTH 82'

NO OF SAMPLE	TYPE OF SAMPLE	SCALE	DESCRIPTION OF MATERIALS	PERMEABILITY	DENSITY	COLOUR	MOISTURE	DILATANCY	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		VERY SOFT	G	M	DILATANCY	0 0 1 1			
		8'										
D2		10	FINE SAND tracey of silt		LOOSE	R	O	NON PLASTIC	2 3 4 5			
D3									2 3 4 5			
D4		20							2 4 4 6			
D5									3 4 4 7			Bottom of caisson
D6		30							3 4 5 7			
D7					MODERATE				6 8 10 17			
D8		40					I		7 9 12 16			
			43'			E						
D9			CLAY little silt		VERY SOFT			PLASTIC	1 1 1 2			
D10		50							1 1 1 2			
D11					SOFT		S		1 2 2 3			
D12		60							2 2 2 4			
D13					MOSTIFF	Y	T		2 3 3 5			
D14		70	CLAY little silt						2 3 3 6			
D15									3 3 3 8			
D16		80							3 3 4 10			
			82'									

INDEX
DISTURBED SAMPLES
UNDISTURD SAMPLES

Fig - 10

RIVER RESEARCH INSTITUTE, BWDB. BORING LOG

PROJECT :- Bridge over Changkhali Khal
LOCATION:- Changkhali
DRILLED BY:- GWD - 1

HOLE NO. # 5
GROUND LEVEL. N/S
GROUND WATER LEVEL River Boring

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG	DESCRIPTION OF MATERIALS	GENERAL: PERMEABILITY	DENSITY	COLOUR	MOISTURE	PLASTICITY	BLOWS ON SPOON PER 6" PENETRATION 0–6	6–12	12–18	18–24	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	REMARKS
		1		SILT,		V.			M.						
D1				trace fine sand,		S			C	0	0	0	1		
D2			10'			O			O	0	0	0	1		
				CLAY,		F	G	M	M						
D3				little fine sand,		T			F	Shelby tube 3"x24" Pushed 15'					
D3			20'	CLAY,		S			L	1	1	2	3		
D4				&		O	R	O	P						
				FINE SAND,		F			L	1	1	3	4		
D5			30'			M			A						
						S			S	2	3	3	6		
				SILT &		I	E	I	T						
D6				FINE SAND,		L			I						
						O			N	3	3	4	7		
				SILT,		S			P.						
D7			40'	little fine sand,		E			M	3	3	5	8		Bottom level of foundation
						M			C.						
						S	Y	S	M						
D8				CLAY,		O			P	1	1	1	2		
D9			50'	little fine sand		F			L						
						T		T	A	1	1	1	2		
									S						
D10									T	1	1	1	3		
									I						
			60'	FINE SAND,		M			C						
				trace silt		D.			M	5	6	6	8		
				trace mica					P.						
									H						
				CLAY,					E	2	2	2	3		
			70'			S			D						
				little fine sand		O			P	1	2	2	3		
						F			L						
D14						T			A	2	2	2	3		
									S						
			80'						T	2	2	2	5		
									I						
									C						

INDEX: DISTURBED SAMPLES; UNDISTURBED SAMPLE

82'

Fig – 11