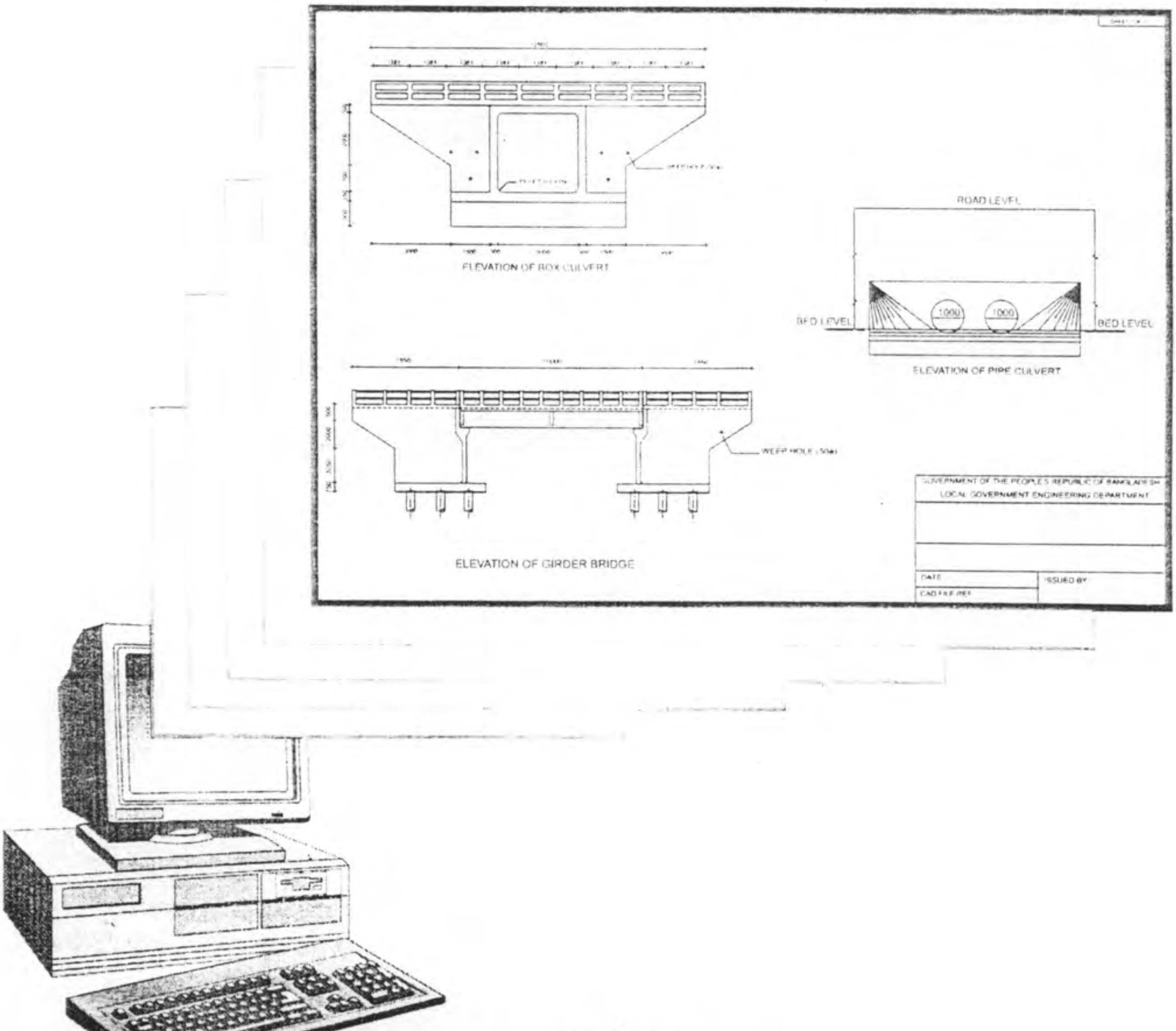
GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF LOCAL GOVERNMENT, RURAL DEVELOPMENT & COOPERATIVES
LOCAL GOVERNMENT ENGINEERING DEPARTMENT

ROAD STRUCTURES MANUAL

PART A
DESIGN CRITERIA, GUIDELINES AND SELECTION OF STRUCTURES



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ABBREVIATION

AASHTO	American Association of State Highway and Transportation Officials
ABUT	Abutment
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
B.M.	Bench Mark
BDS	Bangladesh Standards
BNBC	Bangladesh National Building Code
BPDB	Bangladesh Power Development Board
BRDG	Bridge
BS	British Standard
BSI	Bangladesh Standard Institute
BWDB	Bangladesh Water Development Board
BIWTA	Bangladesh Inland Water Transport Authority
CULV	Culvert
DCC	Dhaka City Corporation
DFL	Design Flood Level
FRB	Feeder Road - B
GDP	Gross Domestic Product
GSB	Geological Survey of Bangladesh
GWT	Ground Water Table
HBRI	Housing and Building Research Institute
HFL	High Flood Level
IEB	Institute of Engineers, Bangladesh
LGED	Local Government Engineering Department
LL	Liquid Limit
LWL	Low Water Level
MDD	Maximum Dry Density
NMC	Natural Moisture Content
OFL	Ordinary Flood Level
PHED	Public Health Engineering Department
PL	Plastic Limit
PWD	Public Works Department
R.L	Reduced Level
RHD	Roads and Highways Department
RRI	River Research Institute
RSM	Road Structures Manual
SCPT	Static Cone Penetrometer Test
SOB	Survey of Bangladesh
SRC	Structure Reference Code
STP	Standard Penetration Test
TRRL	Transport and Road Research Laboratory
USAID	United States Aid for International Development
USCS	Unified Soil Classification System
WASA	Water and Sewerage Authority

CONVERSION TABLES

Basic length conversion factors:

1 mm	= 0.03937 in		1 in	=	25.4 mm
1 m	= 3.281 ft / 1.094 yd		1 ft	=	0.3048 in
1 km	= 0.6214 mi		1 yd	=	0.9144 m
1 mm^2	= 0.00155 in^2		1 mi	=	1.609 km
1 m^2	= 10.76 ft^2		1 in^2	=	645.2 mm^2
1 m^2	= 1.196 yd^2		1 ft^2	=	0.0929 m^2
1 hectare	= 2.471 acres		1 yd^2	=	0.8361 m^2
1 mm^3	= 35.31ft3 / 1.308 yd^3		1 acre	=	0.4047 hectare
1 m^3	= 35.31 ft3 / 1.308 yd^3		1 in^3	=	16390 mm^3
			1 ft^3	=	0.02832 m^3
			1 yd^3	=	0.7646 m^3

Force:

1 N	= 0.2248 lbf	= 0.1020 kgf	
1 lbf	= 0.4536 kgf	= 4.448 N	
1 kgf	= 2.205 lbf	= 9.807 N	
1kN	= 0.1004 tonf	= 102.0 kgf	= 0.1020 tonne f
1 tonf	= 9.964 kN	= 1016 kgf	= 1.016 tonne f
1 tonne f	= 9.807 kN	= 1000 kgf	= 0.9842 tonf

Force per unit length:

1 N/m	= 0.06852 lbf/ft	= 0.1020 kgf/m
1 lbf/ft	= 14.59 N/m	= 1.488 kgf/m
1 kgf/m	= 9.807 N/m	= 0.672 lbf/ft
1 kN/m	= 0.0306 tonf/ft	= 0.1020 tonne f/m
1 tonf	= 32.69 kN/m	= 3.333 tonne f/m
1 tonne f/m	= 9.807 kN/m	= 0.3000 tonf/ft

Force per unit area:

1 N/mm^2	= 145.0 lbf/in^2	= 10.20 kgf/cm^2
1 lbf/in^2	= 0.006895 N/mm^2	= 0.0703 kgf/cm^2
1 kgf/cm^2	= 0.09807 N/mm^2	= 14.22 lbf/in^2
1 N/mm^2	= 0.06475 ton/in^2	= 10.20 kgf/cm^2
1 lbf/ft^2	= 47.88 N/m^2	= 4.882 kgf/m^2
1 kg/f^2	= 9.807 N/m^2	= 0.2048 lbf/ft^2
1 N/mm^2	= 0.06475 $tonf/in^2$	= 1020 kgf/cm^2
1 $tonf/in^2$	= 15.44 N/mm^2	= 157.5 kgf/cm^2
1 kgf/cm^2	= 0.09807 N/mm^2	= 0.006350 $tonf/in^2$
1 N/mm^2	= 9.324 $tonf/ft^2$	= 10.20 kgf/cm^2
1 $tonf/ft^2$	= 0.1073 N/mm^2	= 1.094 kgf/cm^2
1 kgf/cm^2	= 0.09807 N/mm^2	= 0.9144 $tonf/ft^2$

CONVERSION TABLES (Continued)

Force per unit volume:

1 N/m^3	=	0.006366 lbf/ft^3	=	0.102 kgf/m^3
1 lbf/ft^3	=	157.1 N/m^3	=	16.02 kgf/m^3
1 kgf/m^3	=	9.807 N/m^3	=	0.0624 lbf/ft^3
1 kN/m^3	=	0.002842 $tonf/ft^3$	=	0.1020 tonne f/m^3
1 $tonf/ft^3$	=	351.9 kN/m^3	=	35.88 tonne $f/^3$
1 tonne f/m^3	=	9.807 kN/m^3	=	0.02787 $tonf/ft^3$
1 kN/m^3	=	0.003684 lbf/in^3	=	0.1020 $tonnef/m^3$
1 lbf/in^3	=	271.4 kN/m^3	=	27.68 tonne f/m^3
1 tonne f/m^3	=	9.807 kN/m^3	=	0.03613 lbf/in^3

Moment:

1 N m	=	8.851 lbf in	=	0.7176 lbf ft	=	0.1020 kgf m
1 kgf m	=	9.807 N m	=	86.80 lbf in	=	7.233 lbf ft
1 lbf in	=	0.1130 N m	=	0.08333 lbf ft	=	0.01152 kgf m
1 lbf ft	=	1.356 N m	=	12 lbf in	=	0.1383 kgf m

Fluid capacity:

1 liter	=	0.22 imp. gal.	=	0.2642 US gal.
1 imp. gal.	=	4.446 liter	=	1.201 US gal.
1 US gal.	=	3.785 liter	=	0.8327 imp. gal.

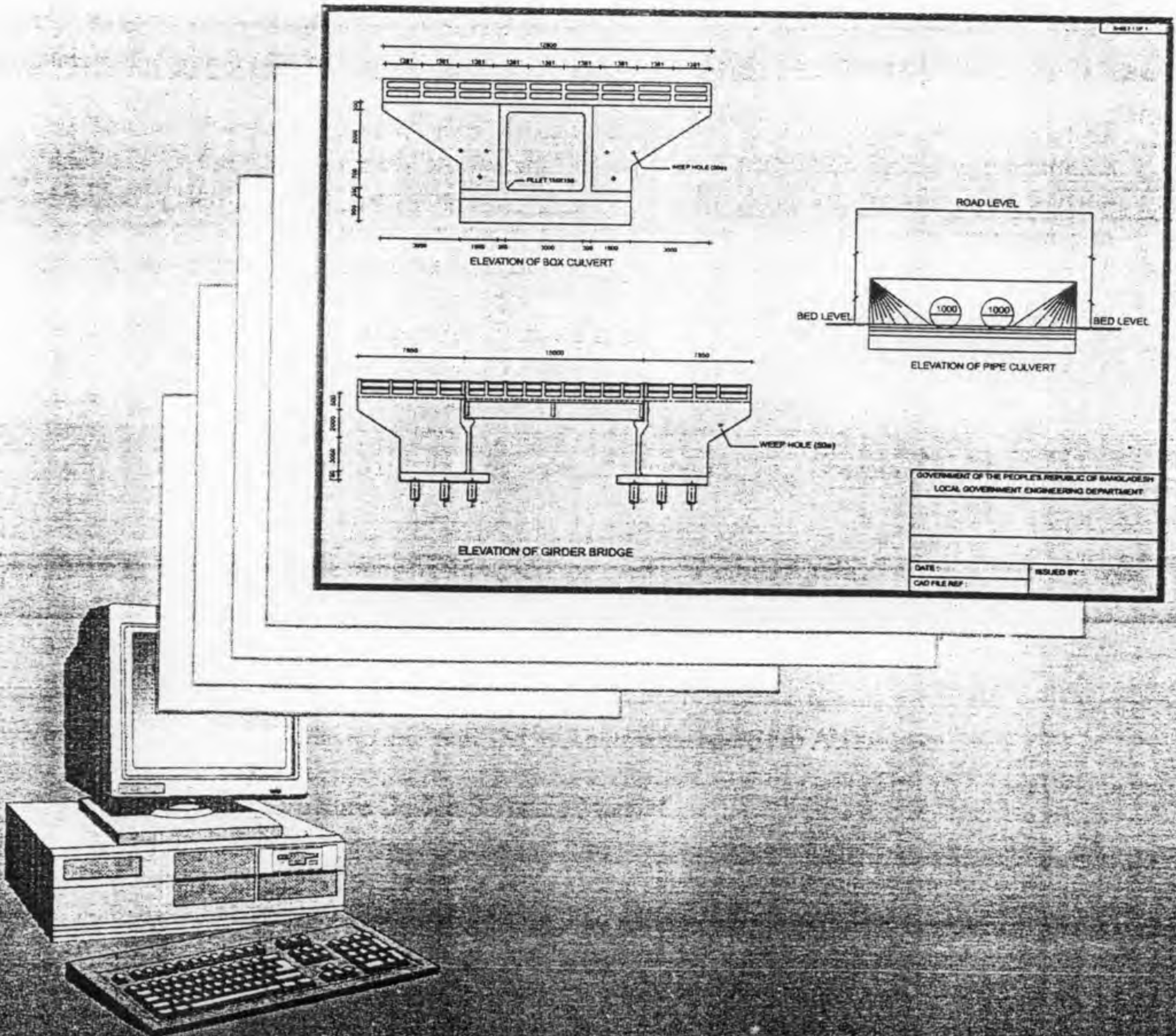
Other useful data:

1000 kg/m^3 = 62.4 lb/ft^3 (density of water)
23 kN/m^3 = 2400 kg/m^3 = 150 lb/ft^3 (nominal weight of reinforced concrete)
14 kN/mm^2 = 140 X 10^3 kg/cm^2 = 2 X 106 lb/in^2 (nominal elastic modulus of concrete)
10 X 10^{-6} per °C = 5.5 X 10^{-6} per °F (nominal coefficient of linear expansion of concrete)

SECTION I PRELIMINARIES

CHAPTER 1: Introduction

CHAPTER 2: Planning and Site Selection



Introduction

The Local Government Engineering Department (LGED) is constructing various types of rural roads in Bangladesh. A large number of structures are being built for channel crossing and for cross drainage purpose on these roads. This development effort is aimed at improving the rural communication network which will ultimately improve the socio-economic conditions of the rural people.

This Manual is prepared with the aim to make cost effective designs, appropriate for rural roads and to modify the existing Road Structure Manual (RSM) prepared in 1989.

This Manual offers a large number of standard designs for small bridges and culverts with guidelines for the whole process of planning, design and construction including preparation of bill of quantities. The guidance is supplemented with standard drawings and a user-friendly software that can select a suitable design and other details from the library of pre-designed structures presented in the Manual.

The Manual is presented in three parts - Parts A, B and C.

Part A	Design Criteria, Guidelines and Selection of Structures
Part B	Contains standard drawings of road structures. This is again divided into three volumes:-
Volume I	Standard Designs of Box and Pipe Culverts and U-Drain
Volume II	Standard Designs of Slab Culverts, Girder Bridges, Brick and R.C.C. Open Abutments.
Volume III	Standard designs of Full Depth Abutments and Stub Abutments
Part C	Road Structure Design Software Manual.

CHAPTER 1
Introduction

1.1 GENERAL

The Local Government Engineering Department (LGED) under the Ministry of Local Government, Rural Development and Cooperatives is responsible for planning, design, construction and maintenance of Feeder Roads Type 'B' and Rural Roads in Bangladesh. A large number of road structures are built annually by the Department all over the country. A great majority of these road structures represents small bridges and culverts. LGED with assistance from its development partners like CARE and USAID implements projects aimed at improving rural road networks to decrease rural transportation costs through development of continuous alignments. Thus in turn, will decrease the cost of agriculture inputs, ensure quick flow of agricultural information, marketing agriculture products and thereby facilitate increase in agriculture production and enhance socio-economic conditions of the rural people. Therefore, it was felt necessary to ensure a sufficient and appropriate cost-effective standard road structure designs are available to the projects in order a maximum number of rural roads are improved.

This Manual is prepared on the joint undertaking of CARE, USAID and LGED to make available cost-effective designs appropriate for rural roads and thereby modify the existing Road Structures Manual (RSM) prepared in 1989.

This Manual offers engineers a large number of standard designs of small bridges and culverts with a comprehensive set of guidelines to assist and simplify the process of design and construction. The structures presented in this Manual are an essential part of each and every rural roads. They are far more common than large bridges and are simpler to design and construct.

In addition to the presentation of standard designs of bridges and culverts this Manual covers the entire design process from the planning stage through site investigations and hydraulic design to the final preparation of drawings and Bill of Quantities. The Manual is aimed at direct use by design and field engineers of LGED and its partner organizations. It gives as much guidance as possible in the form of standard drawings and design guidelines plus a user-friendly software that can select a suitable design and other details from the library of pre-designed structures presented in the Manual.

1.2 ROAD STRUCTURES COVERED IN THE MANUAL

There are a wide variety of road structures which are constructed to pass roads over channels and to allow cross-drainage of surface water. The structures frequently constructed on LGED roads and covered in this Manual are as follows:

1) Reinforced Concrete Girder Bridges

Standard designs of single span simply supported reinforced concrete girder bridges including design guidelines have been provided in the Manual for three types of live loadings - AASHTO H20, H15 and H10 (see Article 8.2.3). For H20 and H15 loading a carriageway width of 3.66m and for H10 loading that of 2.44m have been adopted. Span lengths of bridges vary from 6.0m to 20.0m at 0.5m interval. For bridges with spans above 12.0m pedestrian footpaths on both sides have been provided.

Both reinforced concrete and brick masonry abutments (fig 9.1) have been proposed for the bridges. Three types of reinforced concrete abutments namely (1) Open RC Abutment (2) Full Depth Abutment (with pile foundation) and (3) Stub Abutment have been designed. The height of an Open Abutment varies from 3.0m to 6.0m and that for a Full Depth Abutment varies from 3.0m to 8.0m at 0.3m intervals in both cases. Stub Abutments have been designed in such a way that these abutments can be used for a variable height of 2.0m to 3.0m simply by adjusting the projected pile length. Gravity type brick masonry abutments have been designed for heights from 3.0m to 6.0m at 0.5m intervals.

Wing walls have been provided to all abutments. For reinforced concrete abutments return type wing walls and for brick masonry abutments splayed type wing walls have been adopted.

2) Reinforced Concrete Slab Culverts

Designs of reinforced concrete slab culverts have been provided with spans from 1.5m to 5.5m with 0.5m intervals. The carriageway widths of the slab culverts are 3.66m for H20 & H15 loading and 2.44m for H10 loading. The same abutments as proposed for the reinforced concrete girder bridges are to be used for the slab culverts.

3) Reinforced Concrete Box Culverts

Standard designs of single, double and triple vent reinforced concrete box culverts have been included in the Manual with spans varying from 1.0m to 4.3m. The heights of boxes also vary from 1.0m to 4.3m. The height to span and span to height ratios have been adopted in the range from 1:1 to 1:1.5. Box culverts have been designed for 3.66m and 2.44m carriageway width using H20, H15 and H10 AASHTO live loading. For single vent box culverts with heights 2.3m and less , splayed type wing walls have been provided. For all other box culverts return type wing walls have been provided

U-type irrigation drain of sizes 0.375m x 0.450m and 0.625m x 0.6m with precast top slab have also been provided.

4) Pipe Culverts

Standard designs of pipe culverts (fig 10.1) have been prepared with non-pressure type heavy and light duty reinforced concrete pipes. Three sizes of pipes with diameters 0.3m, 0.6m and 0.9m have been included in the design. Using these diameters single, double, triple and quadruple vent pipe culverts are provided in the Manual.

Cement concrete bedding has been recommended for laying pipes. Brick masonry head walls at a distance of 1.0m, 1.5m and 2.0m from the toe of embankment have been provided on both ends of pipes for 0.3m 0.6m and 0.9m size pipe respectively.

1.3 STRUCTURE OF THE MANUAL

The Manual has been divided into three parts – Part A, Part B and Part C. Part A contains the following sections and chapters:

Section I	**Preliminaries**
Chapter 1	Introduction
Chapter 2	Planning and Site Selection
Section II	**Survey and Investigation**
Chapter 3	Topographical Survey
Chapter 4	Sub-Soil Investigation
Chapter 5	Hydraulic Consideration
Section III	**Design Parameters and Criteria**
Chapter 6	Design Parameters
Chapter 7	Design Criteria
Section IV	**Design Considerations and Procedures**
Chapter 8	Superstructures for Bridges and Slab Culverts
Chapter 9	Substructures and Foundations for Bridges and Slab Culverts
Chapter 10	Pipe Culverts
Chapter 11	Box Culverts
Section V	**Selection of a Standard Design**
Chapter 12	Selection of a Standard Design for Bridges and Slab Culverts

Chapter 13 Selection of a Standard Design for Pipe Culverts

Chapter 14 Selection of a Standard Design for Box Culverts

Section VI **Material Specifications and Bill of Quantities**

Chapter 15 Material Specifications

Chapter 16 Bill of Quantities

Section VII **Annexures**

Part B of the Manual contains standard drawings. This part has been structured in three volumes as follows :

VOLUME I Standard designs of Box Culvert, Pipe Culvert and U-Drain

VOLUME II Standard designs of Slab Culverts, Girder Bridges, Brick Abutments and R.C.C. Open Abutments

VOLUME III Standard designs of Full Depth Abutments and Stub Abutments

Part C of the manual contains the Road Structures Design Software Manual

Planning and Site Selection

In planning a road structure, the engineer has to make surveys and investigations. The investigations involved reconnaissance survey, preparation of a map of the site showing the topographic detail and catchment area, channel cross section, pertinent information on flood levels and flow and other information deemed essential.

In selecting the site, the engineer should give weightage to hydraulic condition of the channel and stability of the bank, economy of construction, good foundation conditions and straight approach

Once a site has been selected, hydrologic and soil investigation should be made. Help may be taken from BWDB gauge data and local field investigation to fix design flood level and flow. Navigation clearance, if any, may be considered to fix the level of the structure.

Type of vehicles expected to ply over the structure should be assessed. Provision for pedestrian footpath should be made if the bridge is long enough and located close to town or a bazar area.

CHAPTER 2

Planning and Site Selection

2.1 GENERAL

Planning for the construction of a road structure is the initial stage of design, during which the road/bridge engineer identifies a preferred location for the structure, justifies the necessity of the structure, decides on the type, size and capacity of the structure and recommends on the further detailed surveys and investigations. A reconnaissance of the site and an investigation of pertinent project characteristics, are essential for proper planning.

2.2 RECONNAISSANCE SURVEY AND PREPARATION OF LOCATION MAP

The objective of the reconnaissance survey is to

i) collect information to help to determine the viability of the construction from technical as well as socio-economic aspects.

ii) establish the general characteristics of the proposed bridge site including the surrounding area.

iii) identify a suitable location for the structure

iv) identify further detailed investigations required and to prepare a site location map.

Before start of the reconnaissance survey a tentative site map is to be prepared using available union/thana/district maps. For this, important details of the area are to be traced on tracing paper showing the proposed tentative location of the structure and the existing roads and waterways. With this map in hand the reconnaissance party will visit the site. They will locate, examine and check different objects and features, include new objects, if any, on the map, examine possibilities of alternate sites and finally select the best possible site for the structure. Three cross-sections of the channel, one along the centre line of the proposed structure, one at 25m upstream and one at 25m downstream should be taken during reconnaissance survey. All information obtained at this stage will be recorded in a Field Data Sheet as in Table 2.1. On the basis of the survey a reconnaissance survey report is to be prepared containing the following information :

1) A location map at a suitable scale of the proposed site, showing all roads, structures, waterways, important objects and features of the surrounding area.

2) A brief justification of the proposed construction and of the selected location of the structure.

3) The maximum and normal high flood levels, normal low water level, flow width during high and low water levels, approximate velocity of flow, vertical and horizontal navigational clearances required, information about soils, scour, erosion and siltation of the present waterway etc. Type of river traffic with their capacity, frequency etc.

4) Probable span arrangement and required length of the proposed structure.

5) Probable approach road alignment.

6) Length, span arrangement and condition of the existing bridge/culvert, if any, at the proposed site and at upstream and down stream locations.

7) Nature of the waterway and soil.

8) Locally available construction materials.

9) Recommendations regarding detailed topographical survey, sub-soil investigation, hydrological analysis and materials investigation.

10) Type of vehicles expected to ply over the proposed structure with their loadings, frequency etc.

11) Any other information deemed necessary for consideration during future planning and design.

2.3 SITE SELECTION

The location of the majority of the proposed small bridges/culverts on rural roads will be fixed. In some cases, where the structure crosses an existing defined channel the engineer has to reach a compromise between the easiest channel crossing and the shortest road alignment. The choice of location then becomes an economic decision. The cheapest structure site and the one that potentially has the longest service life is on a location that :

- is on a straight reach of the channel
- is beyond the disturbing influence of larger tributaries
- has well-defined/stable banks
- has reasonably straight approach roads
- permits as square a crossing as possible
- has good foundation conditions

The site should allow the maximum gradient of the approach roads to be appropriate to the types of vehicle likely to travel on the road as well as offering vertical curves and sight distances suitable for the maximum speed of vehicles.

A structure aligned at right angles to the channel results in the shortest length. A skew bridge requires more material and is more complicated to design and construct and therefore, should be avoided.

2.4 SITE CONDITIONS

Once a site for the structure has been identified, it is necessary to obtain field information on the local terrain and channel conditions including information on soil and hydraulic data. The key points of field information relate to :

- if exist, the catchment area of the channel
- water levels
- navigational clearance requirements
- sub-soil characteristics

The extent of the channel catchment area determines the area to be considered to estimate flow volumes. If maps are available the limits of the catchment area may be marked on them. Otherwise the size of the catchment area and its average gradient should be estimated by means of observation at the site.

Information is needed on the maximum high flood level, the normal flood level and the low water level at the proposed site. The maximum HFL should be determined by local observation and by making local enquires. It may also be obtained if there is a gauge station nearby established by BWDB, IWTA or any other organisations. The silt marks that high floods generally leave on tree trunks and buildings remain visible for several years and may be used for this purpose. It is usually helpful to ask people who have been living in the area for a long time about their recollections of high floods, but it should be kept in mind that this source of information is variable in its reliability. It is better to make such inquiries by talking to people individually rather than in groups. The normal HFL is the level to which the channel water normally rises during monsoon season. The low water level (LWL) is the level prevailing in the channel during dry season. If there is no water in the channel in dry season, the period during which the river bed remains dry should be noted.

The deck level is to allow for the passage of any regular or occasional river craft as well as the clearance of floating debris at times of flood. The clearance required for this is called the navigational clearance. Experience from other structures on the same river/khal, together with local enquiries may be used to determine the required navigational clearance.

2.5 TRAFFIC

The composition and volume of the vehicular traffic likely to use the road structure throughout its design life should be estimated. The volume of current traffic, if any, can be determined from a simple traffic count or from local information. The growth rate should be calculated taking into account the local factors, such as agricultural or industrial development and the general increase in gross domestic product (GDP).

2.6 PEDESTRIAN FOOTPATH

A pedestrian footpath is required for long bridges or for any structure located near a town or Bazar area. This should be recorded by the engineer during the reconnaissance survey.

Table 2.1: Proforma for Reconnaissance Survey for Road Structure

1.	Date of Survey		:	
2.	Name of Project		:	
3.	(a) Name of Road		:	(b) Category of Road :
4.	Location of Structure			
	(1)	At chainage .... from	:	
	(2)	Village	:	
	(3)	Union	:	
	(4)	Thana	:	
	(5)	District		
5.	At least 3 photograph covering the full view of the proposed site		:	
6.	Sketch map (sketch map showing existing roads, waterway/catchment area, direction of flow, location of proposed and existing structures, etc. to be attached as attachment 'A')		:	See Attachment 'A'
7.	Site Characteristics			
	(1)	Type of Road	:	Earthen/HBB/Bituminous/Concrete
	(2)	Cross-section of the road/embankment near the structure	:	
	(3)	Cross-section of the proposed approach road/embankment	:	
8.	Waterway Characteristics			
	(sketch of cross-section to be attatched as attachment 'B')			
	(A)	For Defined channel	:	
	(1)*	Cross-section along C.L. of proposed structure	:	See Attachment 'B'
	(2)*	Cross-section at 25m upstream	:	See Attachment 'B'
	(3)*	Cross-section at 25m downstream	:	See Attachment 'B'
	(4)	Condition of Bank	:	Stable/Unstable
	(5)	Bank erosion/scour	:	High/Medium/Low
	(6)	Bed Scour		High/Medium/Low

Table 2.1 (continued)

	(7)	Navigational requirement (with type of river traffic)	:	...........m
		(i) Type	:	Launch/cargo/engine boat etc.
		(ii) Minimum vertical clearance required	:	...........m
		(iii) Minimum horizontal clearance required	:	.........m
	(8)	Source of water flow	:	Rain/Stream/Tidal
	(9)	Quality of water	:	Normal/Saline
	(10)	Period of low water/no water	:	.... months from ...... to ....
	(11)	Width of water surface at HFL	:	...........m
	(12)	Maximum depth of channel at deepest point from HFL	:	...........m
	(13)	Average depth of channel bed from HFL	:	...........m
	(14)	Approximate velocity of flow during high flood	:	........m/sec
	(15)	Area of channel cross-section at HFL	:	...........sq.m
	(16)	Discharge through the channel at HFL	:	........m3/sec
	(17)	Approx. catchment area	:	...........sq.km
	B.	For ill defined channel. (In addition to information taken in case of Defined Channel)		
	(1)	Approximate catchment area (area from which water flows to the site) during flood	:	..........sq.km
	(2)	Total cross-sectional area of all other structures feeding the catchment area	:	...........sq.m
	(3)	Total cross-sectional area of all other structures drawing from the catchment area	:	...........sq.m
9.	Existing Structure			
	(1)	Structure at site	:	Yes/No
	(2)	Structure at upstream	:	Yes/No; at ....m upstream
	(3)	Structure at downstream	:	Yes/No; at ....m downstream
	(4)	Length of existing structure(s)	:	...........m
	(5)	Waterway opening of existing structure(s)	:	...........m
	(6)	Navigational clearance of the existing structure(s)		...........m
	(7)	Condition of the existing structure	:	Good/Fair/Poor*

Table 2.1 (Continued)

10.	Type of soil at the site	:	Cohesive/Non-cohesive/Organic
11.	Type of vehicle expected to ply over the structure		
	(1) Truck	:	Yes/No
	(2) Car/Jeep/Microbus	:	Yes/No
	(3) Bus/Minibus	:	Yes/No
	(4) Bullock Cart	:	Yes/No
	(5) Rickshaw/Van	:	Yes/No
12.	Construction materials available in the area	:	Stone/Sand/Brick/Steel/ Cement/Timber
13.	Type and size of structure required	:	
	(1) Type of structure	:	
	(2) Length/Span	:	
14.	Requirement of footpath	:	Yes/No
15.	Additional information if any		

* If poor, state condition of abutment, pier, etc. including scour at bank/bed.

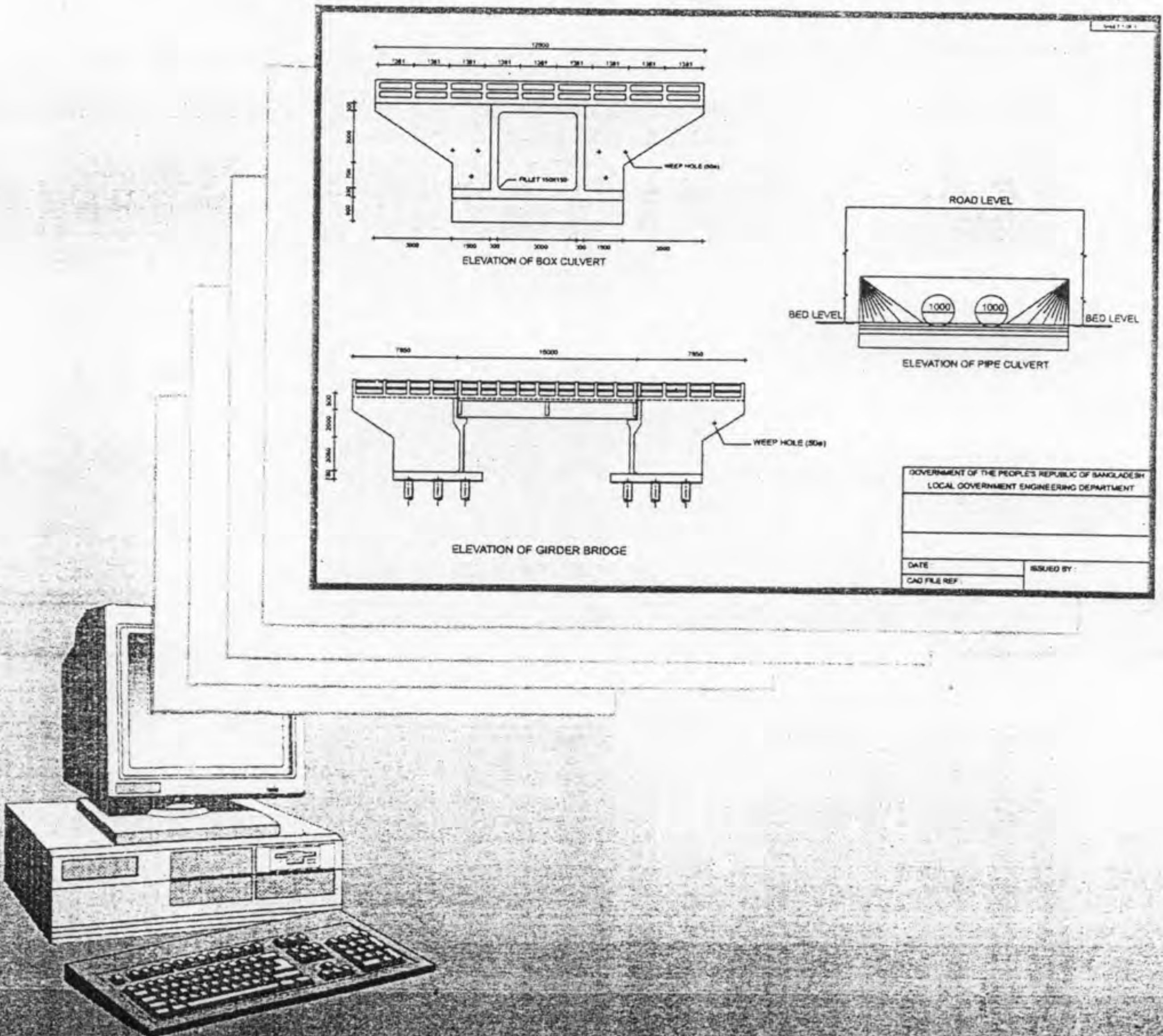
Note : Maximum HFL, normal HFL, normal LWL etc. are to be shown on the cross-sections all with reference to a particular point or an arbitrary bench mark. Sources of information about the water levels are also to be mentioned.

SECTION II SURVEY AND INVESTIGATION

CHAPTER 3: Topographical Survey

CHAPTER 4: Sub-Soil Investigation

CHAPTER 5: Hydraulic Consideration



Topographic Survey

Topographic survey of the structure site is essential for determining the exact location of the structure and geometrical features. A map to a suitable scale is prepared showing the approach road, channel section and the surrounding features. Cross section along the center line of the bridge/culverts and two sections 50m apart upstream and two sections downstream are needed.

Based on the survey, final location of the structure is determined. The survey may be done either by upon traversing, stadia surveying or plane table surveying. The suitability of the method depends on the site condition. However, plane table survey is more simple and suitable in most cases.

Taking cross sections are necessary part of the survey. If there is water, cross-sections may be taken with the help of sounding at close interval to define the bed profile. For structures in this Manual, five cross sections are recommended, one at the centre line of the structure and two upstream and two downstream at 50m interval.

Fixation of TBM and connecting it to the national BM established by Survey of Bangladesh are important steps for future references.

The process of cross section taking, levelling and writing the data should be done with great care.

CHAPTER 3
Topographical Survey

3.1 GENERAL

A topographical survey of the road structure site is necessary to obtain detailed information regarding the physical site characteristics and geometrical features around the site. As an output of the topographical survey a plan of the site and cross sections of the channel and approach roads are prepared. The plan is prepared to a suitable scale preferably to 1:1000. The cross sections are drawn to a suitable scale preferably both horizontal and vertical being the same. The cross sections should be taken one along the centerline of the proposed bridge/culvert, two upstream and two downstream from the centre line. Three additional cross sections on each of the approach roads are to be taken at 50m centres. Considering the limited length of road structures covered by this Manual it is proposed that an area of at least 100m radius from the centre point of the structure should be covered by the topographical plan. A typical Site Plan has been presented in fig 3.2.

3.2 PREPARATION OF DETAILED SITE PLAN

After the tentative location of the structure has been identified during the reconnaissance survey the detailed site plan may be prepared. One survey party headed by a Junior Engineer with a team of one surveyor, two survey assistants and one labour is required for undertaking this survey. The detailed site plan is required to ascertain the final location and span/length of the structure to be built. Information on the terrain and other features in the vicinity of the proposed site is marked on the plan, as well the direction of river flow and location of cross sections. The alignment of the road on which the structure is to be built is taken as a reference line. The bearing of the reference line with respect to the north-south line should be recorded on the site plan.

The detailed site plan may be prepared by following any of the three methods namely (1) open traversing, (2) stadia surveying and (3) plane table surveying. The type of surveying to be selected for the preparation of site plan depends on the type of instrument available and the field conditions. Open traverse surveying is possible only when all the features to be located in the map are easily accessible and the reference lines as well as the perpendicular distances between the objects and the reference lines can be directly measured by chain and tape. In stadia and plane table surveying, direct measurement of distances except the measurement of base or reference line can be avoided. Considering simplicity, versatility and easy availability of instruments plane table survey is recommended for preparation of site plan for the structures covered in this Manual. In plane table surveying, the observations and plottings are carried out simultaneously. The instruments required are (a) a plane table with tripod stand, (b) an alidade, (c) a trough compass, (d) a plumb bob, and (e) a spirit level (f) a scale rule (g) a steel tape measure. These are shown in Fig. 3.1.

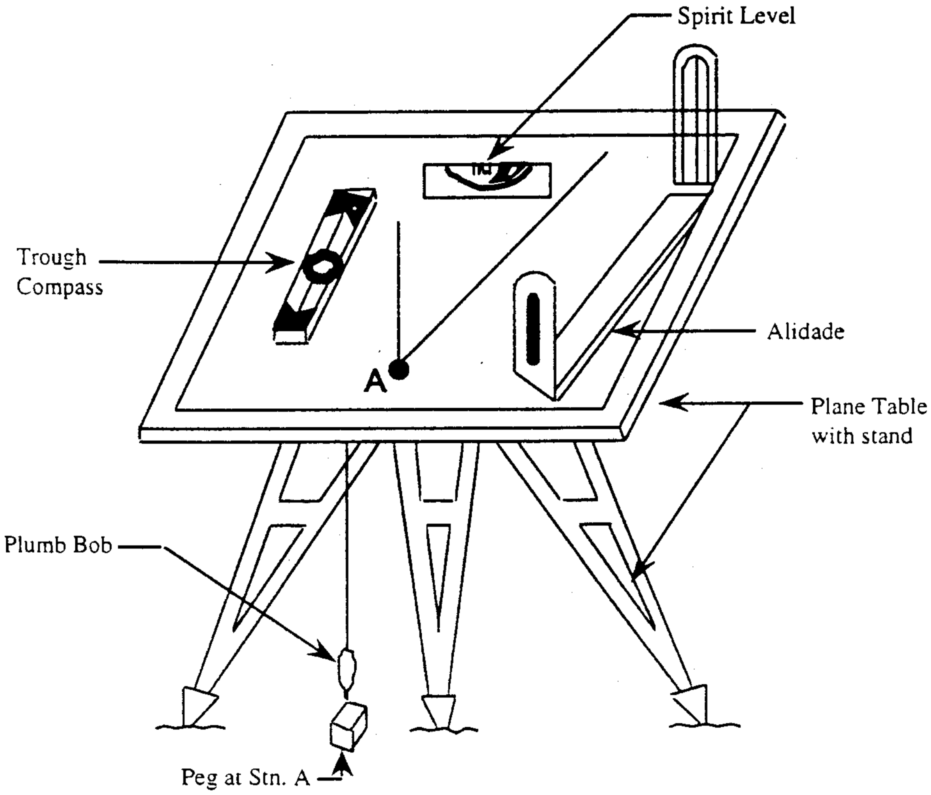


Fig. 3.1 Instruments for Plane Table Surveying

Either the radial or the intersection method can be used for plane tabling of the site of a road structure. The procedure of plane tabling is described below :

1) Select a station on the ground and fix it with a wooden or bamboo peg and set the plane table on the station. The table should be set on the station with the help of a plumb bob in such a way that the point representing the station on the table is directly above the peg. The station should be located on level ground and in a position where the maximum number of objects and features are visible from it. Fix-up a drawing sheet on the table and mark the station on it.

2) Level the table with the help of a spirit level and draw the North line with the help of a trough compass.

3) Select important points at the edges and corners of visible features and sight them through an alidade keeping its edge on the station and draw rays along the edge of the alidade.

4) Measure the distances between the station and the features and plot them to an appropriate scale (preferably 1:1000) along the rays drawn sighting

these features. For features whose distances are large or inaccessible the method of intersection for locating them on the plan may be used.

5) Fix another station from which the distant features sighted from this station are also visible and draw a ray towards that station. Measure the distance between the stations and plot the new station on the table.

6) Shift the plane table to the second station, set the table on this station and level it. Orient the table by rotating until the trough compass placed along the north line indicates north. Check the orientation by back sighting towards the previous station.

7) Draw rays sighting the distant objects already sighted from the previous station and plot the features at the point of intersection.

8) Sight new features from this station and draw rays to these features and where possible plot them on the table measuring distances from this station.

9) Shift the table to a third station and so on. Repeat the same procedure to cover all the features required to be plotted.

The sight plan prepared in the field by pencil may be traced on a tracing paper and inked by a draftsman in the office. A typical sight plan prepared through a plane table survey is presented in Fig. 3.2.

3.3 TAKING CROSS SECTIONS

Cross sections are necessary to determine the configuration of the channel bed at different locations and also the levels of the approach road and surrounding area. If the channel is dry or contains shallow water during survey work, cross sections of the channel may be taken by levelling only. In case the channel contains deep water, sounding is required for the deep water portion of the channel.

For the structures covered by this Manual it is recommended that 5 cross sections should be taken in the channel. These are – one along the centre line of the proposed structure and two each on both upstream and downstream sides at a distance of 25m and 50m from the centre line. Additional three cross sections are recommended to be taken on each of the approach roads – one at the structure end and two at a distance of 50m and 100m from the structure end.

3.3.1 Fixation of Bench Mark

A Bench Mark (B.M) is a point with known elevation. Elevation or Reduced Level (R.L) is the rise or fall of any point with respect to the mean sea level. Before undertaking levelling work at any site, first the B.M. at the site has to be fixed. This may be done by flying the R.L. of the B.M. from any permanent B.M. established by the Department of Survey. If such permanent B.M. is not available at adjacent areas an arbitrary B.M. may be used. This B.M. is to be fixed at the site at some permanent

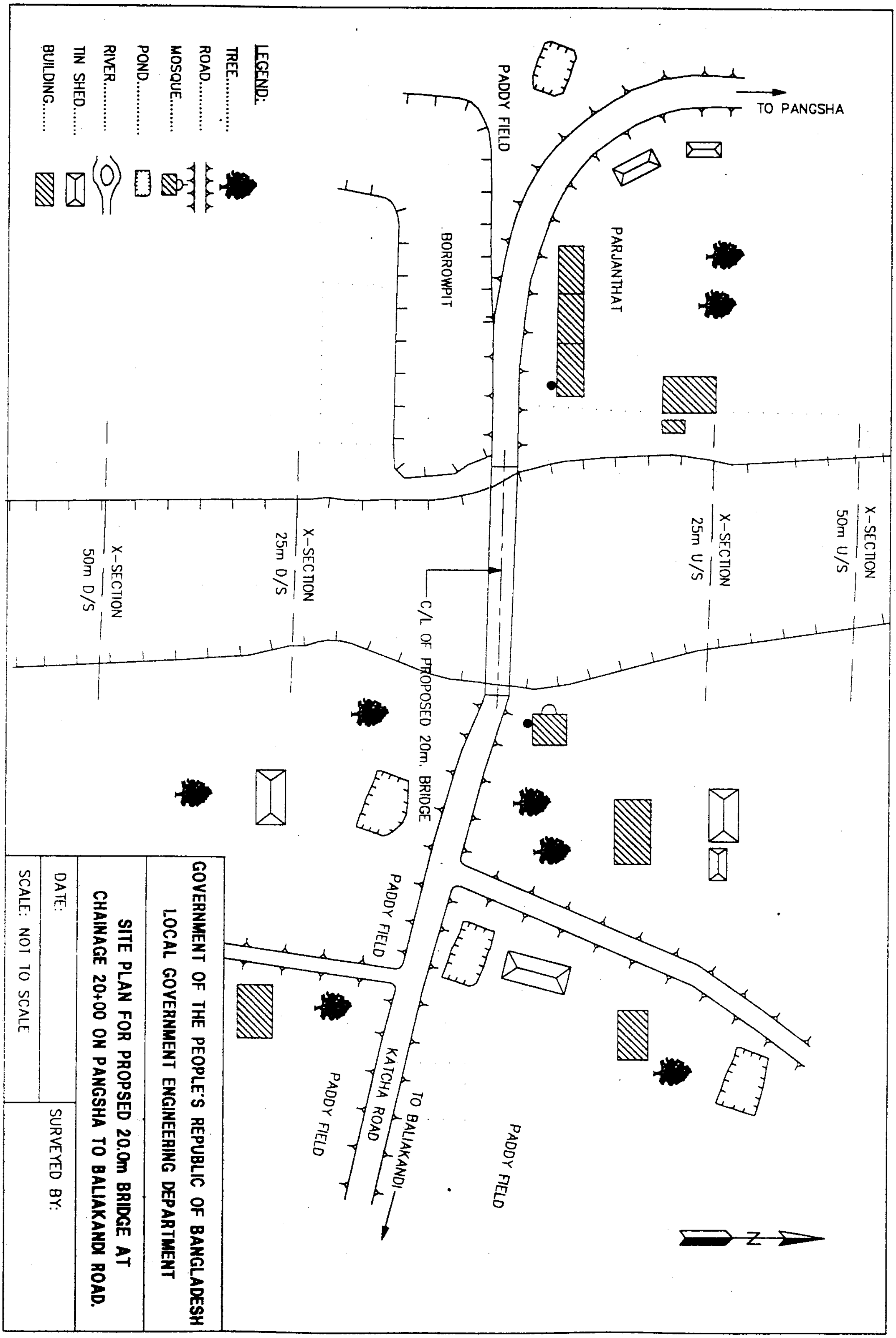


Fig. 3.2 A Typical Site Plan

object like existing buildings, bridges or trees etc. such that it remains visible and undisturbed up to the start of the actual construction work. Temporary B.M. may also be established by making reinforced concrete or brick masonry pillars at a suitable location and writing the R.L. on top of the pillar.

3.3.2 Cross Sectional Levelling

For undertaking cross sectional levelling the locations of the cross sections as stated above are to be marked first. Then using a level machine and a levelling staff elevations of different points on the cross sectional line are determined by adding or subtracting the difference of readings taken through the level machine on the staff on that point and on the B.M. The readings obtained during levelling operations are entered in a Level Book as illustrated in Table 3.1 below.

Table 3.1 Level Book Writing for a Channel Cross Section

Staff Station	Distance (m)	Staff Reading			Rise	Fall	Ht. of Inst.	Reduced Level	Remarks
		Back	Inter	Fore					
6	0.0	0.750	-	-	-	-	10.000	9.250	B.M.
7	1.5	-	1.465	-	-	0.715	-	8.535	
8	3.0	0.150	-	3.200	-	1.735	6.950	6.800	
9	4.5	-	1.095	-	-	0.945	-	5.855	
10	6.0	-	2.805	-	-	1.710	-	4.145	
11	7.5	-	3.505	-	-	0.700	-	3.445	
12	9.0	-	3.615	-	-	0.110	-	3.335	
13	10.5	-	3.415	-	0.200	-	-	3.535	
14	12.0	-	1.905	-	1.510	-	-	5.045	
15	13.5	3.295	-	0.245	1.660	-	10.000	6.705	
16	15.0	-	1.250	-	2.045	-	-	8.750	
17	16.5	-	0.730	-	0.520	-	-	9.270	
18	18.0	-	-	0.760	-	0.030	-	9.240	

From the data obtained in the Level Book, the cross sections of channel can be drawn as follows (Fig. 3.3).

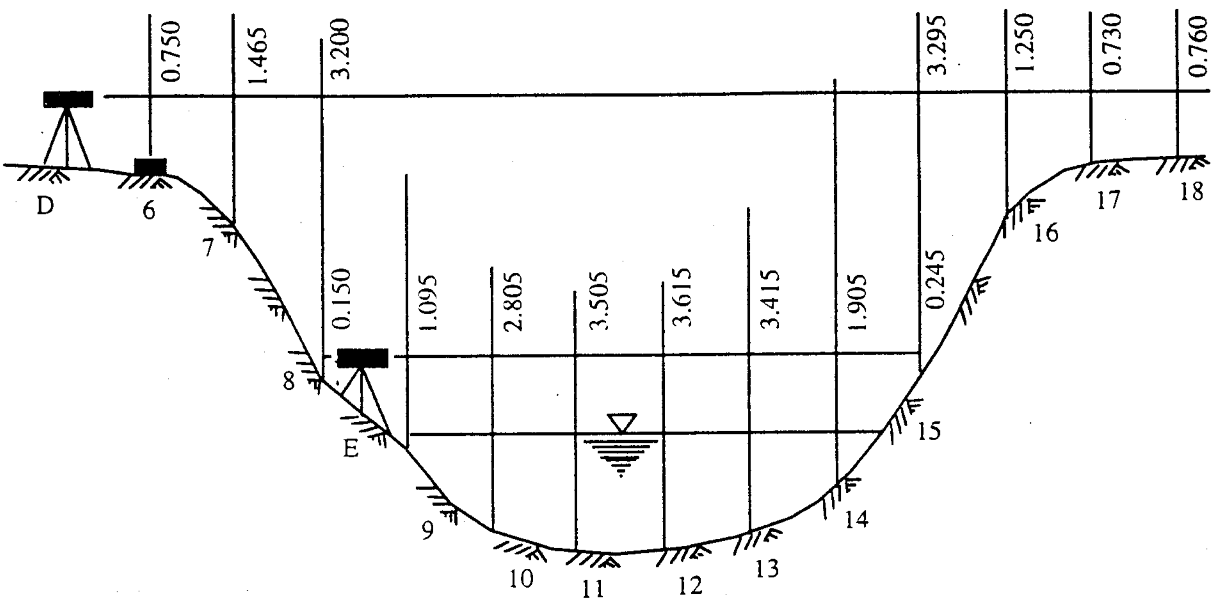


Fig. 3.3 Cross Sectioning of a Channel

3.3.3 Sounding

The measurement of depth under water is known as sounding. This is necessary for taking cross sections of a channel containing water. Modern scientific methods using echo sounder are now available to undertake sounding in deep channels. For structures on rural roads covered by this Manual, it is recommended that in lieu of the echo sounder, the following simple method can be used :

i) Drive two poles on opposite banks of the channel (Fig. 3.4).

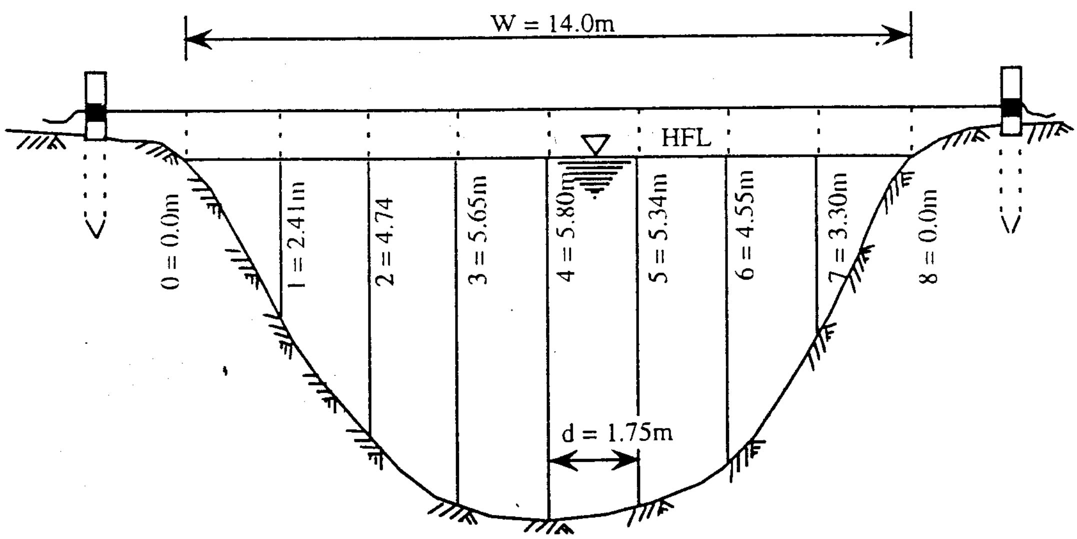


Fig. 3.4 Sounding along a Cross Section

ii) Tie the ends of a string securely to both the poles in such a way that the tie is horizontal without having any sag. The horizontality of the string can be checked by using a level machine or maintaining the string parallel to water surface.

iii) Divide the width of the channel into equal divisions by marking on the string. Smaller divisions in highly irregular bed will give more accurate bed profile.

iv) Measure the water depths at the marks with the help of levelling staff, graduated pole or string with plumb bob at the end and record the depths $0_1, 0_2, 0_3$ .... etc. on a rough sketch of the channel.

v) If the desired highest flood level does not exist during measurement, level may be measured with respect to the string and difference in level between the string and highest flood may be added or subtracted to obtain the actual ordinates at HFL.

vi) Using the R.L. of bed as obtained above and R.L. of water surface, the channel cross section may be drawn.

Sub-Soil Investigation

Sub-soil investigation is essential to determine, within practical limits, the stratification, ground water condition and engineering properties of the soils underlying the sites of the proposed culverts/bridges. The information is necessary for safe and economic design of the foundations of the structures.

The field and laboratory investigations, which will be required to obtain the above information of the construction sites have been described in Chapter 4 which mainly deals with the following:

- The uses and limitations of some common field investigation methods, namely test pit, hand auger boring, rod penetration, standard penetration test (SPT), static cone penetration test (SCPT), field vane shear test, pocket penetrometer test, etc. have been described.

- Different laboratory tests, which should be done on the disturbed and undisturbed soil samples obtained during field investigations, have been listed with relevant ASTM/BS specifications.

- Methods for assessment of soil types by Unified Soil Classification System (USCS) have been demonstrated by charts and tables.

- An approximate but quick method of assessment of uniformity in soil condition has been discussed.

- Procedures for assessment of bearing capacity of soil layers by different field and laboratory tests, using available correlation charts, tables and empirical equations have been outlined. Methods of determining bearing capacity of shallow foundation by Terzayhi's and Meyerhof's bearing capacity equations have been described.

- Procedures for assessment of settlement characteristics of soils occurring below the foundation base using the results of SPT and SCPT carried out in-situ and natural moisture content, plasticity and consolidation tests carried out in the laboratory have been outlined.

- Procedures for estimation of capacity of driven piles based on standard penetration test (SPT), static cone penetration test (SCPT) and laboratory shear test have been discussed.

Chapter 4
Sub-Soil Investigation

4.1 GENERAL

4.1.1 Purpose of Sub-Soil Investigation

Before an engineer can design the foundation of a structure, he must have a reasonably accurate conception of the engineering properties and the arrangement of the underlying materials. The field and laboratory investigations required to obtain this essential information is called the sub-soil investigation. Because of the complexity of natural deposits, no one method of sub-soil investigation is best for all situations. Engineering judgement has to be exercised for selection of the elements of a sub-soil investigation depending on the type of the structure to be constructed.

The main purpose of the sub-soil investigation is to determine, within practical limits, the stratification, ground water table and engineering properties of the soils underlying the site of the proposed culverts and bridges considered in this Manual. The principal properties of interest will be the strength and settlement characteristics of the underlying soils. The sub-soil investigation program should be planned so that the maximum amount of information can be obtained at a minimum cost. The investigation cost should not generally exceed 0.50 to 1.0 percent of the project cost.

4.1.2 Phases of Sub-Soil Investigation

Sub-soil investigation normally falls into three phases; reconnaissance, desk study and ground investigation, although phases may be overlapped, merged or omitted depending on site conditions and the requirements of a particular project.

4.1.2.1 Reconnaissance

It involves visiting the site and its surroundings and noting the salient features of the area. Useful information can be obtained simply by visiting the site and noting such features as topography, drainage, soil types, rock outcrops, vegetation, land use, and the condition of existing roads, culverts, bridges and buildings. Details of former use of the site and nearby structures or proposed developments which may also affect, or be affected by the project, should be considered. Pertinent information such as cracks, noticeable sags, and possibly sticking doors and windows of nearby structure and its foundation treatments should be taken.

4.1.2.2 Desk Study

It includes a review of available information from geological/agricultural soil survey maps and records of sub-soil investigation carried earlier out in the vicinity, if any.

As much as possible information should be obtained from existing sources. The desk study is typically carried out at about the same time as the initial site visit. Ideally, information from the desk study should be available before visiting site but this is often not possible. For some site the desk study may yield a great deal of information but for others, almost nothing. Typical sources of information in Bangladesh are :

1. Bangladesh Water Development Board (BWDB)
2. River Research Institute(RRI), Faridpur
3. Roads and Highways Department (RHD)
4. Geological Survey of Bangladesh (GSB)
5. Housing and Building Research Institute (HBRI)
6. The District Councils
7. Dhaka City Corporation (DCC) and District Municipal Offices
8. Public Health Engineering Department (PHED)
9. Dhaka and Chittagong WASA
10. Local Government Engineering Department (LGED)
11. Bangladesh Power Development Board (BPDB)
12. Public Works Department (PWD)
13 Private Organisations
14 Consultants

Information obtained through these sources may be quite useful for economic assessment of soil conditions of the concerned sites/projects.

4.1.2.3 Ground Investigation

Ground investigation includes sinking test pits and borings, field tests and observations, and laboratory testings.

There are many methods of ground investigation. The choice of methods depends on the depth to be investigated, the type of sampling required, the strata likely to be encountered and the resources available. The most widely used method of subsurface investigation for compact sites and most extended sites is boring holes into the ground from which samples may be collected for either visual inspection or laboratory testing. Several procedures are commonly used to make the holes to obtain the soil samples.

The procedures are briefly described in Article 4.2.

4.1.3 Execution of Soil Exploration

Soil exploration has the following three major components:

(1) Planning

Method(s) of subsurface exploration and soil sampling, location, number and spacing between the test points, estimated depth and type of field and laboratory tests required for a particular culvert/bridge should be listed in the light of the information gathered during reconnaissance and desk study. Engineering judgement must be used in selecting the appropriate method(s) of field and laboratory investigations for each circumstance.

(2) Execution

Different field and laboratory investigations as per planning will be carried out to obtain the required data.

(3) Report Writing

Field and laboratory test results along with other information as detailed in Article 4.1.3.2 will be presented in the report.

All the three components are equally important for the satisfactory solution of the problem. However, the execution of the soil exploration program works as a bridge between planning and report writing and therefore occupies an important place. No amount of planning would help writing a good report, if the execution of field works and laboratory testings are not done with diligence and care. So, it is essential that the execution part must always be entrusted to well qualified, dependable and resourceful geotechnical consultants who will also be responsible for report writing. A specialist organisation offering comprehensive facilities for boring, sampling, field and laboratory testing, and soil mechanics analysis may undertake the whole investigation; and this has been much preferred to the system whereby one organization does the borings, another the testing and yet another the analysis. Only a single organisation can maintain the essential continuity and close relationship between field, laboratory, and office work.

It is advantageous for the single organisation to permit the boring and testing program to be readily modified in the light of information obtained as the boring work proceeds and additional samples can be obtained, if felt necessary, from soil layers shown by laboratory tests. Whatever procedure is adopted for carrying out the site exploration work it is essential that the individuals or organization undertaking the work should be conscientious and completely dependable (Tomlinson, 1986).

4.1.3.1 Record of Field Works

A detailed record of boring operations and other tests carried out in the field is a very essential part of the sub-soil investigation. The borehole log should be made during the boring operation. The soil is classified based on visual examination and touch of the collected disturbed samples. The borehole log should include the difficulties faced during the boring operations and should contain information on the occurrence of sand boils and the presence of artesian water conditions, if any, etc. A typical example of a borehole log is presented in Fig. 4.1.

4.1.3.2 Report Preparation

The report should normally include the followings :

- A general description of the nature of the project and its importance
- A general description of the topographical features and hydraulic conditions of the site
- A brief description of the various field works and laboratory testings carried out and difficulty faced
- Analysis and discussion of the test results
- Recommendations
- Borehole logs, Tables and Figures containing Field and Laboratory test results
- Drawings containing an index plan, a site plan, etc. showing bore-hole locations with R.L.

RECORD OF BORING AND TESTING

Project Name :

Client :

Borehole No. :
Method of Boring :
Boring Dia : cm
Depth of Boring : m

Existing Ground Level : m(RL)
Ground Water Level : m (EGL)
Date Starting : Time :
Date Completion : Time :

LEGEND
SAND
SILT
CLAY
GRAVEL
MICA
SPT+ DISTURBED SAMPLE
UNDISTURBED SAMPLE

Location of Boring :

STRATIFICATION

Depth below G.L. (m)	Thickness (m)	DESCRIPTION OF SOIL STRATA
0 – 7.0	7.0	Grey/brown medium stiff/stiff SILT, trace fine sand, high plastic (MH)
7.0 – 8.6	1.6	Brown very stiff SILT, trace fine sand, low plastic (ML)
8.6 – 13.0	4.4	Brown dense/very dense fine SAND, some silt, non-plastic (SP)
13.0 – 17.0	4.0	Reddish brown/brown medium dense/dense medium to fine SAND, little silt, non-plastic (SP)
17.0 – 22.0	5.0	Grey/brown medium dense/dense SILT, little fine sand, non-plastic (ML)

STANDARD PENETRATION TEST (SPT)

Sample No.	SPT Intervals (meter)	BLOWS ON SPOON PER 15cm PENETRATION: 15 cm	15 cm	15 cm	N - value
D 1	1.5	1	3	3	6
D 2	3	2	3	4	7
U 1					
D 3	4.5	4	5	6	11
D 4	6	3	5	7	12
D 5	7.5	5	10	13	23
D 6	9	16	20	30	50
D 7	10.5	11	17	26	38
D 8	12	9	16	25	41
D 9	13.5	11	17	26	43
D 10	15	5	7	7	14
D 11	16.5	7	9	16	25
D 12	18	6	10	19	29
D 13	19.5	6	11	20	31
D 14	21	8	13	22	35
D 15	22.5				

GRAPHICAL REPRESENTATION OF N - VALUES

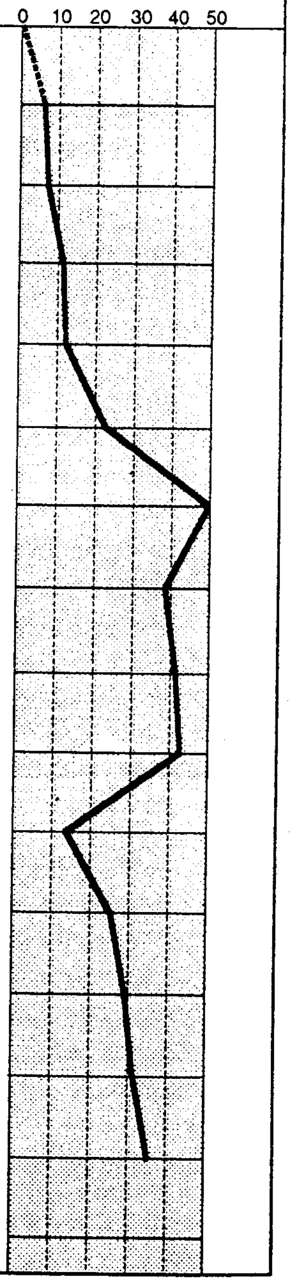


Figure -

Fig. 4.1 Sample of Boring and Testing Log

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4.2 METHODS OF SUB-SOIL INVESTIGATION

There are many diferent methods for soil exploration. The following methods, which are commonly used, are outlined in this section.

- Test pits
- Hand auger boring
- Rod penetration
- Standard penetration test
- Rotary drilling
- Cable percussion boring

All these methods are described in details in the relevant BS/ASTM specifications. In this Manual the description of the methods are brief and intended only to remind the geotechnical engineer about the uses and limitations of the tests.

4.2.1 Test Pits

Test pit is the cheapest method of soil exploration to shallow depth. A pit is normally dug by hand using any local labour in Bangladesh. A mechanical excavator, where available, may also be used to remove the bulk of the material before the sides and bottom are squared and cleaned for examination. A pit should be at least 1m square at the bottom with adequate side slopes or support of the slopes, if there is a risk of collapse. Before a man goes into the pit for work, the possibility of poisonous or asphyxiating gases should also be kept in mind, and gas detection device provided as deemed necessary. The maximum practical depth to which a pit can be excavated is about 3m; below a depth of about 1.2 to 1.5m the sides of the pit will require support or will need to be excavated at a safe slope. The test pits should be limited to ground water table.

Test pit method of soil investigation provides a clear picture of the stratification of soils and the presence of any lenses or pockets of weaker material. This method enables hand-cut samples of soil, giving the minimum disturbance to be taken. Test pits are particularly valuable in investigating the nature of fill materials when voids, loosely deposited layers, or deleterious material can readily be recognized. Test pits or trenches are, in fact, the only ready means of obtaining adequate information on filled ground or highly variable natural deposits. However, in water-bearing soils, particularly in sands, there may be difficulty in digging below the water table and trial pits may be more costly than borings in such conditions (Tomlinson, 1986).

In-situ shear strength can be determined in the cohesive soil layers encountered in the test pit by pocket penetrometer and vane shear tester as described in Article 4.5.1 under field tests. Approximate strength of cohesive soil may also be estimated by rod penetration test in the test pits as described in Articles 4.2.3 and 4.8.1.1.

Disturbed and practically undisturbed samples can be obtained from the pits. Hand-trimmed samples can be obtained. Careful hand-trimmed sample yields the least disturbed sample than any other method. Since the undisturbed samples obtained above the water table are not fully saturated, the test results obtained from such samples have to be analysed with caution.

4.2.2 Hand Auger Boring

The hand auger boring method uses light hand-operated equipment. The auger and drill rods are normally lifted out of the borehole without the aid of tripod. and no borehole casing is used. Auger borings often provide the simplest method of soil investigation and sampling. They may be used for any purpose where disturbed samples can be used and are valuable in connection with ground water level determination and indication of changes in strata. Hand auger borings are also cheap means of subsurface exploration in favorable types of soil, as the soils must have sufficient cohesion to stand unsupported in an unlined borehole and there must not be large cobbles, boulders, or other obstructions which would prevent rotation of the auger. Hand augers (also known as post hole augers) are usually of sizes from 38mm through 203mm (1.5 through 8 inches) and are provided with extension rods. Holes can be sunk to depths of about 5-6m in soft to firm (medium stiff) clays/sands possessing some cohesion. The auger holes can be used, by special arrangement, for collection of undisturbed samples from cohesive soil strata. Sketch of a auger is shown in Fig. 4.2. The hand augers can be locally made by supplying technical specifications.

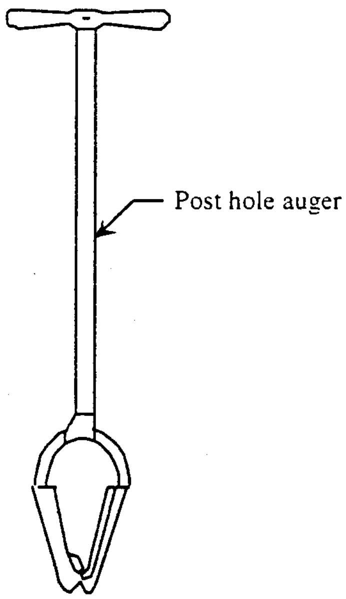


Fig. 4.2 Hand Auger

4.2.3 Rod Penetration Test

In this method an initial excavation up to the depth of the foundation base is required. At this depth, a steel rod 10mm to 16mm, (3/8 to 5/8 inch diameter) is forced into the ground by applying a known static weight (usually the weight of the person performing the test) at several points within the proposed foundation area. Uniformity in penetration values at all locations will indicate uniformity in soil condition. Depth of penetration can be correlated with the soil strength as indicated in Fig. 4.13. The method is suitable for cohesive soil only.

4.2.4 Standard Penetration Test

Granular, cohesionless soils are especially difficult to sample without disturbance. For this reason the engineering properties are usually determined by taking disturbed samples and measuring the in-place relative density using a penetration resistance test. The disturbed soil samples of cohesionless and cohesive strata obtained during penetration resistance test are useful for plasticity and grain size analysis. The penetration tests are made at frequent intervals with at least one test in each soil stratum. The normally accepted vertical interval between tests in one hole is 1.5m (5 feet), although it may sometimes be desirable to test continuously in one or more holes at each time. Several dynamic and static penetration tests have been developed in different countries of the world and different tools and procedures are used. One widely accepted dynamic method is the standard penetration test.

The standard penetration test (ASTM D1586) consists of driving a 50.8 mm (2-inch) outside diameter split spoon soil sampler (Fig. 4.3) into undisturbed, in-place soil strata. The soil sampler is known as a split spoon. The driving effort utilizes a 63.5kg (140-lb) drop hammer freely falling an average vertical distance of 762mm (30 inches). The method is most suitable under a wide variety of conditions of drilling holes into the ground and extracting samples for identification and, in some instances, for detailed testing in the laboratory. Several procedures are in common use for drilling the holes. Likewise, a variety of methods is available for obtaining the samples, the choice depends on the nature of the materials and on the purpose of the exploratory program. Any type of well drilling equipment with adequate depth capacity can be used to advance the borehole. The two most common methods are wash boring and augering. The hand augering has limitations as discussed earlier.

4.2.4.1 Wash Boring

Wash boring is very common in use for advancing the boreholes into the soils of Bangladesh for collection of disturbed samples simultaneously with standard penetration test in the boreholes at required depths.

In the beginning of wash boring, the borehole is advanced a short depth by auger of suitable diameter (generally 100mm diameter) and then a casing pipe is pushed into the ground to prevent the sides from caving in. The borehole is then continued by the use of a chopping bit fixed at the end of a string of hollow drill rods of standard size. A stream of water or drilling mud under pressure is forced through the hollow rod (wash-pipe) and the bit into the borehole which loosens the soil. The wash-pipe is pulled up and down or rotated by hand in the borehole. The water or mud flow carries the loosened soil up the annular space between the wash-pipe and the side of the borehole and it overflows at ground surface, where soil in suspension in water/drilling mud is allowed to settle in a tank or pond and the top fluid from the settling tank or pond is re-circulated or discarded to waste as required. Soils settled in the tank/pond can be visually inspected in-situ for identification and classification, but this procedure is often unreliable as the cuttings are mixed as they flow up the borehole and in the settling tank, and the structure of the soil damaged. However, accurate identification is possible from the split spoon samples and undisturbed sample tubes taken from the boreholes at regular intervals. Wash boring has the advantage that the structure or density of the soil below the bottom of the borehole is not disturbed by blows of the boring tools, but the method can not be used in large gravels, shales or soil containing boulders. It is best suited to uniform sands or clays and that is why the wash boring method is popularly used in Bangladesh by the soil boring organisations. A variety of tools are used for fitting to the end of the wash pipe for different soil types. The use of mud instead of water allows the hole to remain uncased. The motive power for wash boring is either mechanical or man power. Man power is the most popular motive power in Bangladesh. Sketch of a wash boring set is shown in Fig. 4.3.

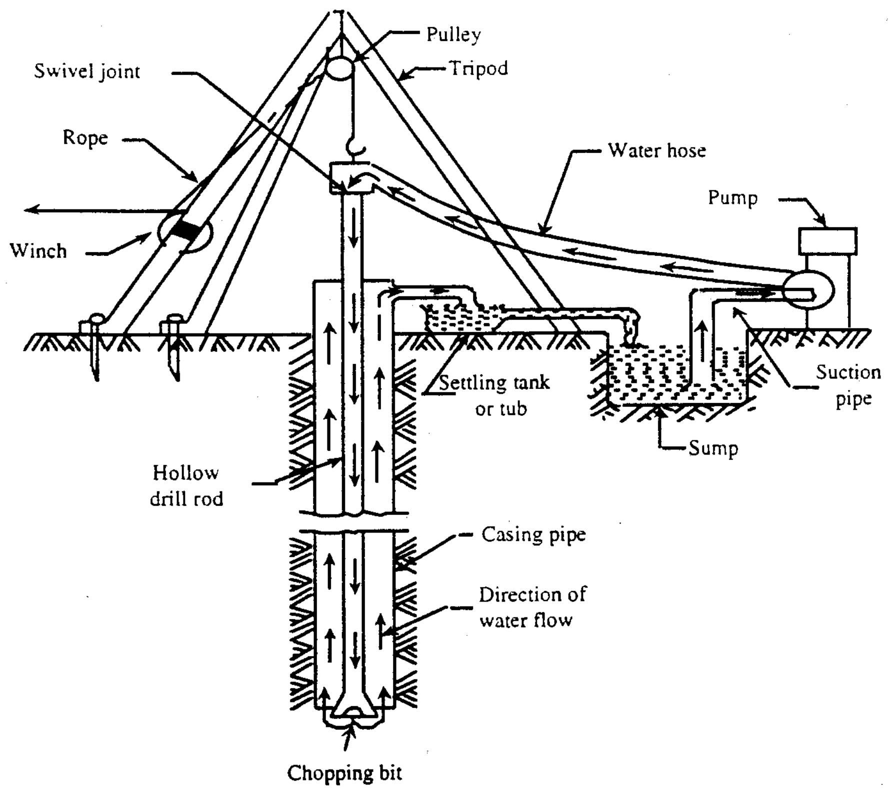


Fig. 4.3 Sketch of Wash Boring Set

4.2.5 Rotary Drilling

Rotary drilling is also a method of wash boring normally used for rocky soil strata. In this method a cutter bit or a core barrel with a coring bit attached to the end of a string of drill rods is rotated by a power rig. The core barrel is used primarily in rocky strata to get rock samples. Diamond coring bits are the most versatile of all the coring bits (Murthy, 1993). For investigation of small bridge sites, rotary drilling may not be necessary in the very common soil deposits of Bangladesh.

4.2.6 Cable Percussion Boring

Cable percussion boring, which is another method of making borehole, can be carried out in all types of soil since the boreholes can be lined where required with steel casing tubes, and a wide variety of tools are used for different soil and rock types. A cable percussion rig employs a friction winch to raise and lower the boring tools. The machines can be provided with a hydraulic power take-off to operate a rotary core drill attachment for coring in rock to a limited depth of penetration (Tomlinson, 1986). As soils of Bangladesh are composed mostly of alluvial clays, silts and medium to fine sands, cable percussion boring is not popular in Bangladesh and wash boring is adopted in most cases of boring, sampling and penetration test.

4.3 EXTENT OF SUB-SOIL INVESTIGATION

The site investigation may range in scope from a simple examination of the surface soils with or without a few shallow test pits, to a detailed study of the soil and ground water conditions to a considerable depth below the surface by means of boreholes and in-place and laboratory tests on soils encountered. The extent of the investigation work depends on the importance and foundation arrangement of the structure, the complexity of the soil conditions, and the information which would be available on the behaviour of existing foundations on similar soils during site visit and desk study as discussed earlier.

4.3.1 Depth of Boreholes

The depth up to which boreholes should be sunk is governed by the depth of soil affected by the foundation bearing pressures, which depth is called the significant depth, or the zone of influence of the structure. The usual practice is to sink the borehole to at least 1.5 times the least width of the foundation from the base level of the foundation as this depth is generally considered as the zone of influence of the structural load. However, the boreholes may have to be taken to greater depths depending on the type of the structural foundation and strata encountered during the progress of the borehole.

In this respect AASHTO standard specifications for highway bridges (Fifth edition, 1992) states "Where substructure units will be supported on spread footings, the minimum depth of the subsurface exploration shall extend below the anticipated bearing level a minimum of two footing widths for isolated, individual footings where L < 2B, and four footing widths for footings where L > 5B. For intermediate footing lengths, the minimum depth of exploration may be estimated by linear interpolation as a function of L between depths of 2B and 5B below the bearing level. Greater depths may be required where warranted by local conditions.

Where substructure units will be supported on deep foundations, the depth of the subsurface exploration shall extend a minimum of 6m (20 feet) below the anticipated pile or shaft tip elevation. Where pile or shaft groups will be used, the subsurface exploration shall extend at least two times the maximum pile group dimension below the anticipated tip elevation, unless the foundations will be end bearing on or in rock".

Therefore, in case of pile or pier foundations the subsoil should be explored upto the depths required to cover the soil lying even below the tips of piles (or pile groups) and piers which are affected by the loads transmitted to the deeper layers.

However, the following depths of boreholes are recommended :

(1) For culverts: 6 - 10m

(2) For girder bridges: ≤ 20m

4.3.2 Number of Boreholes

Adequate number of boreholes is needed to

- provide a reasonably accurate determination of the contours of the proposed bearing stratum.
- locate any soft pockets in the supporting soil which would adversely affect the safety and performance of the proposed design.

The number of boreholes which need to be sunk on any particular site is a difficult problem which is closely linked up with the relative cost of the investigation and the project for which it is undertaken. When the soil is homogenous over the whole area, the number of boreholes could be limited, but if the soil condition is erratic limiting the number would be counter productive. According to AASHTO specification for highway bridges (1992) "a minimum of one soil boring shall be made for each substructure unit. For substructure units over 33m (100 feet) in width a minimum of two borings shall be required. A substructure is any structural, load supporting component generally referred to by the terms abutment, pier, retaining wall, foundation or other similar terminology."

However, the following is the recommended number of borehole(s) of 100mm diameter for different types of culverts and bridges :

(1) For box culvert of single cell - one borehole at the centre
(2) For two-vent culverts and bridges - one borehole at the centre
(3) For bridges of three or more vents - two boreholes, one at each abutment.

If static cone penetration test (SCPT) is adopted, which can be done comparatively at lesser cost, at least one SCPT at each substructure unit should be done.

4.4 SOIL SAMPLING

Two types of soil samples are extracted from the augerholes, boreholes and test pits. They are :

- Disturbed samples, and
- Undisturbed samples.

4.4.1 Collection of Disturbed Samples

The augerhole samples, the hand samples dug from test pits, and the soils of the spilt spoon sampler in the standard penetration test are disturbed samples and are tested to identify and classify soils. The structure of the natural soil may be disturbed to a great extent by the action of the boring tools or excavation equipment. The samples are placed in airtight jars or bags and labelled to identify location, test pit or borehole number, depth of sample, and date of sampling.

4.4.2 Collection of Undisturbed Samples

The undisturbed samples are collected either by the use of (a) thin-walled steel tube (Shelby tube) sampler, and/or (b) a piston sampler.

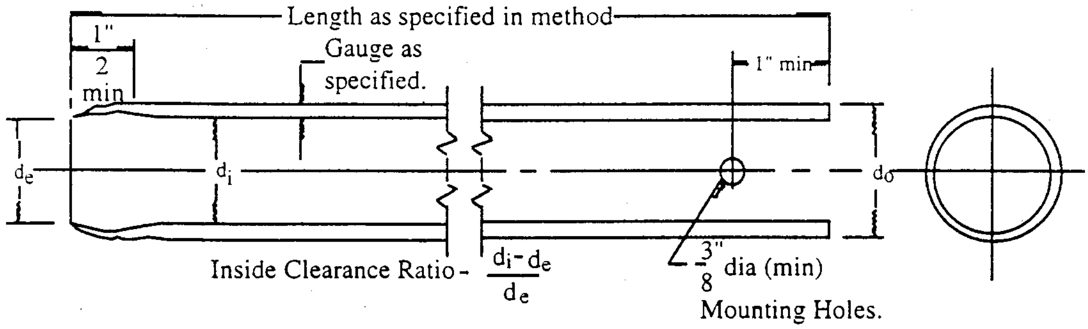
4.4.2.1 Thin-Walled Tube Sampling

Thin-walled tube sampler is an open drive sampler and is popularly known as Shelby tube. It is a thin-walled seamless steel tube sampler with a hard beveled cutting edge and connected to sampler head. The sampler head contains a ball check valve and ports which permit the easy escape of water or air from the sample tube as the sample enters the tube. The sampling method is described in ASTM D1587. Sketch of a thin-walled seamless steel tube is shown in Fig. 4.4. The thickness of tube wall is governed by the area ratio, A_r, which is defined as

$$A_r = \frac{d_o^2 - d_i^2}{d_i^2}$$

where d_i = inside diameter, and

d_o = outside diameter.



NOTE :
- Minimum of 2 mounting holes on opposite sides for 2 to $3\frac{1}{2}$ in. sampler.
- Minimum of 4 mounting holes spaced at 90 deg for samplers 4 in. and larger. Tube held with hardened screws.
- In order to obtain samples without much friction, it is necessary to reduce the friction between the soil core and the inside. This is accomplished by crimling the cutting edge so that its inside diameter, d_e is slightly smaller than the inside dimaeter, d of the tube. The degre of sampling disturbance is also affected by the Inside Clearance Ratio:

  $C_r\ (\%) = 100\ \frac{d_i - d_e}{d_e}$
- For undisturbed samples of high quality the inside clearance ratio should not exceed 1 per cent.

Fig. 4.4 Thin - Walled Tube Soil Sampler

The area ratio varies from 10 to 20 percent depending upon the type of soil to be sampled. A higher ratio is required in very stiff to hard clay strata, and smaller ratio in soft clay. The procedure of sampling involves attaching a string of drill rods to the sampler tube adopter and lowering the sampler on to the bottom of the borehole. The sampler is either pushed or driven the required depth, and then sheared off by giving a twist to the drill rod at the top. The sampling tube is then taken out of the hole, and the tube is taken out of the sampler head. The top and bottom of the sample is suitably sealed with molten wax so that natural moisture of the sample is not lost ; and the tube is identified with labels, one pasted on the tube and one tied in the mounting hole. The labels should contain particulars, namely project, borehole number, sample number, depth, etc. The tubes with samples should be transported in the wooden box and the tubes should be cushioned in the box with saw-dust or straw so that the tube samples are not disturbed by jerking during transportation to the laboratory. The lid of the wooden box should be screwed at site so that the lid can be removed at the laboratory by unscrewing it with a screw driver. The lid must not be closed with nails as hammering the nails disturbs the tube samples in the box. Necessary identification, strength and compressibility tests should done on the undisturbed samples after carefully ejecting the samples from the tubes by a suitable ejector at the laboratory.

The undisturbed sampling tubes may be of different diameters and lengths ranging from about 50mm to 100mm diameter and 450mm to 600mm length. If only unconfined compressive strength of the cohesive soil is required then 50mm diameter tube sample is taken for economy of sampling cost. If unconfined compressive strength test or triaxial shear test and consolidation test are to be done on the cohesive sample, then 75mm tube sample is collected. If it is desired to test three specimens of 38mm diameter and 80mm length from the same depth of cohesive soil, then a 100mm tube sample is collected. If the cohesive soil layer contains larger particles like gravels, then also 100mm diameter tube sample is collected for the above tests. If direct shear test on 60mm x 60mm specimens is required to be done, then also tube sample with diameter of 90mm or more is required. In Bangladesh such cohesive soil formation with large particles is generally not encountered and undisturbed tube sample larger than 75mm diameter is not taken. In fine-grained soils of Bangladesh 75mm outside diameter thin-walled steel tubes of 1.24 to 1.65mm wall thickness are generally used for sampling.

Ground Water Table

After completion of SPT and soil sampling, the initial level of ground water within the borehole is determined with the help of a measuring tape. The borehole is preserved with a cover to prevent entry of water from outside into the hole and left for 24 hours. After 24 hours the steady water table of the area as indicated in the borehole is recorded on the log of the borehole.

4.4.2.2 *Piston Sampler*

A piston sampler (Fig. 4.5) is used to improve the quality of samples and to increase the recovery of soft or slightly cohesive soils. The piston sampler is particularly used for sampling in cohesive soils. The piston sampler employs a fixed piston which can be held at any desired level by a rigid rod extending to ground level. Thus the sampler with its end closed by the piston, can be pushed down below the soil at the bottom of the borehole which is disturbed by the boring operations, and on reaching the depth where the ground is considered to be undisturbed the piston can be held in position and the tube pushed down. In fact in soft soils it is possible to push the sampler down through the soil to any desired depth. The sampling tube is pushed into the soil under hydraulic pressure by keeping the piston stationary. The presence of the piston prevents the soft soils from squeezing rapidly into the tube and thus eliminates most of the distortion of the samples. Piston samplers should never be driven down by hammer but should be pushed down by hydraulic jack.

In Bangladesh piston sampler is not commonly used as it makes the boring and sampling more expensive. The open drive sampler is locally made but piston sampler is not yet made locally.

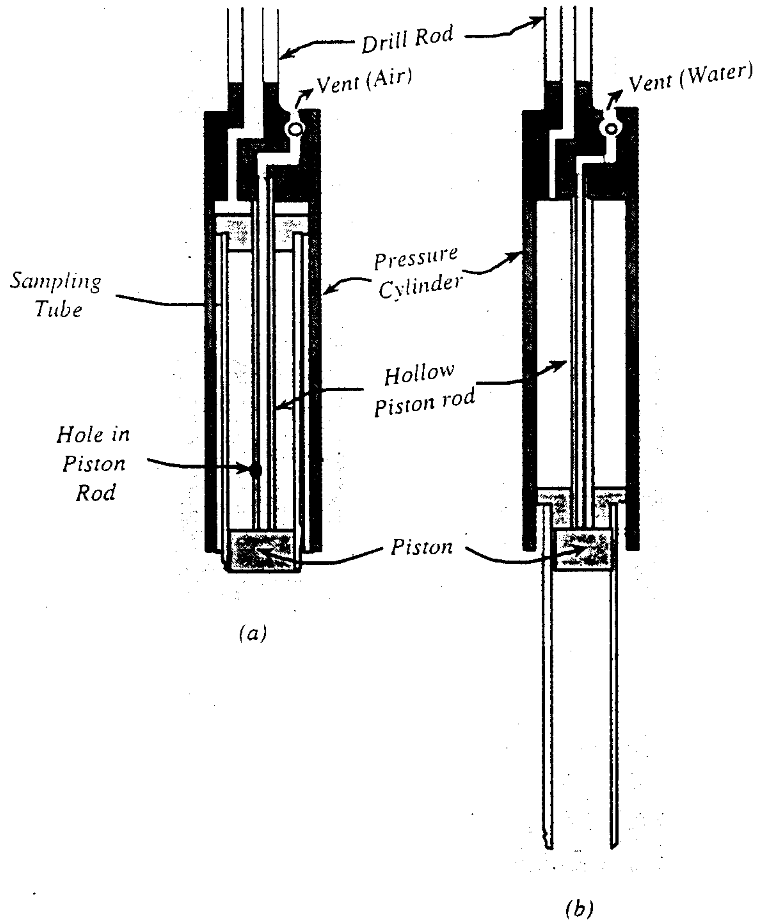


Fig. 4.5 Piston Sampler of Hydraulically Operated Type. (A) Lowered to Bottom of Drill Hole, Drill Rod Clamped in Fixed Position at Ground Surface. (B) Sampling Tube after being Forced into Soil.

4.4.2.3 Test Pit Undisturbed Sampling

Undisturbed samples collected from test pits are likely to be the least disturbed samples compared to all other types of samples.

The basic procedure consists of trimming out a column of soil to a size slightly smaller than the container to be used for transportation, sliding the container over the sample and surrounding the sample with wax. Tight, stiff containers that can be sealed, and are not readily disturbed, should be used. Sketch for undisturbed sampling in test pit is shown in Fig. 4.6.

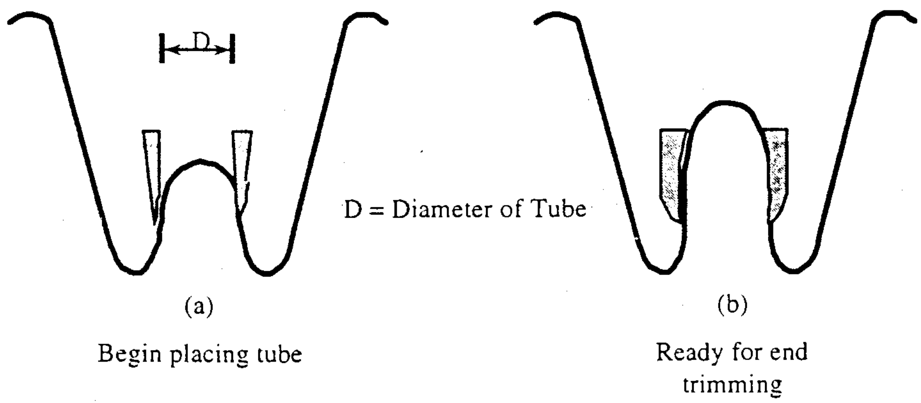


Fig. 4.6 Sketch of Undisturbed Sampling in Test Pit

The undisturbed soil samples obtained by the above mentioned techniques will be sufficiently intact to enable the ground structure within the sample to be examined. However, the quality of these undisturbed samples can vary considerably, depending on the sampling technique and soil condition; and most soil samples will show some degree of disturbance.

Table 4.1 indicates the quantity of sample required for identification purposes, namely Atterburg tests, moisture content, sieve analysis and hydrometer analysis.

Table 4.1 Mass of Soil Sample Required for Identification Purposes

Soil Type	Mass Required, kg
Clay, silt, sand	2
Fine and medium gravel	5
Coarse gravel	30

4.5 SOIL TESTING

4.5.1 Field Tests

Field test may include the following :

- Density measurements
- Shear vane tests
- Penetration tests
  1) Standard penetration test
  2) Static cone penetration test
  3) Pocket penetrometer test
  4) Rod penetration test
- Plate bearing test

These tests are done according to the relevant BSI and ASTM standards and are described very briefly here.

4.5.1.1 Density Measurements

Determination of the dry density of soil on the site can be done by sand replacement method and core cutter method as per Test 15 (A), 15 (B), 15 (C) and 15 (D) of BS 1377:1975 or ASTM D1556-82 and ASTM D2937-83. Sand replacement and core cutter methods are commonly used in Bangladesh. In-place dry density of earth embankments becomes necessary to assess the relative compaction of earth embankment and subgrade of the old existing road and of the newly compacted embankment fill and subgrade with respect to laboratory standard or modified maximum dry density (MDD).

4.5.1.2 Shear Vane Tests

Shear vane tests are usually applicable to uniform cohesive, fully saturated soils. The presence of even small amounts of coarse particles, rootlets or thin laminations of sand leads to unreliable results.

The vane shear test apparatus was developed to measure the shear strength of very soft and sensitive clays. The standard equipment and test procedure are described in British Standard 1377:1975 (Test 18). The vane test is performed by rotating the four bladed vane in the soil below the bottom of a borehole or by pushing down and rotating the vane rod independently of boring. Thus the test is done in soil unaffected by boring disturbance. The undrained shear strength of a clay as measured by the vane test can differ quite appreciably from the actual field strength as measured from the behaviour of full-scale earthworks (Tomlinson, 1986).

In recent years variety of vane shear testers have been developed by different manufacturers for determining shear strength of uniform cohesive soils within the test pits of excavation. One of such apparatus has been in use in Bangladesh by Development Design Consultants Ltd in test pits for shallow foundation. The apparatus developed by English Drilling Equipment Co. Ltd (EDECO) PILCON is a Hand Vane Tester for quick and accurate determination of in situ shear strength of cohesive soils, either on site or on undisturbed samples in the laboratory. The instrument comprises a torque head with a direct reading scale which is turned by hand. A no-return pointer indicates the reading. Vanes either 19mm or 33mm diameter, with optional extension rods are screwed into the rear of the torque head, and pushed at least 200mm into undisturbed clay. Extra 300mm or 1000mm extension rods can be used for greater penetration and for gaining access to difficult or dangerous locations. The unit has recently adapted to show on the dial a conversion factor to B.S. 1377 Vane Test results. The principal components of the apparatus are shown on Fig. 4.7.

4.5.1.3 Penetration Tests

4.5.1.3.1 Standard Penetration Test (SPT)

The standard penetration test is described in ASTM D1586-84 and in Test 19 of BS 1377 :1975. The test is made in boreholes by means of the standard 50.8mm outside diameter split spoon sampler. It is very useful means of determining the approximate in-situ density (relative density) of cohesionless soils and consistency of cohesive soils.

The split sampler of 50.8mm outside diameter and 35mm inner diameter is driven to penetrate 450mm (18 inch) into the undisturbed soil in the borehole by drops of a hammer weighing 63.5 kg (140 lbs) falling freely from a height of 762mm (30 inch). The blows required for penetration of each 150 mm (6 inch) is recorded. The number of blows for the last 300mm (12 inch) penetration is taken as the Standard Penetration Test Value (SPT N or simply N). These are recorded in the borehole log as shown in Fig. 4.1. Soil sample in the split spoon is collected and preserved in airtight jars or bags with proper identification labels for laboratory testings.

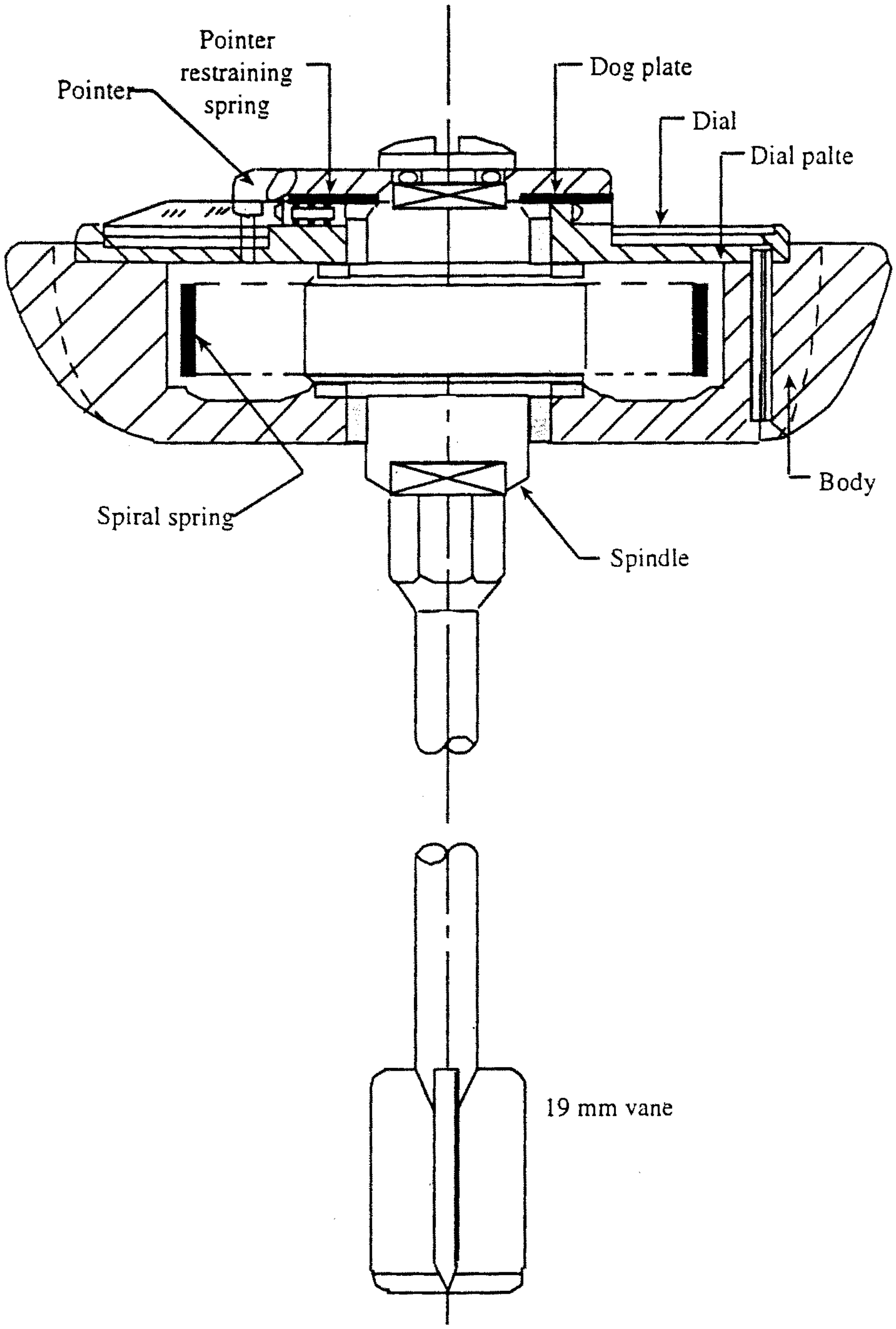


Fig. 4.7 Principal Components of PILCON Hand Vane Tester

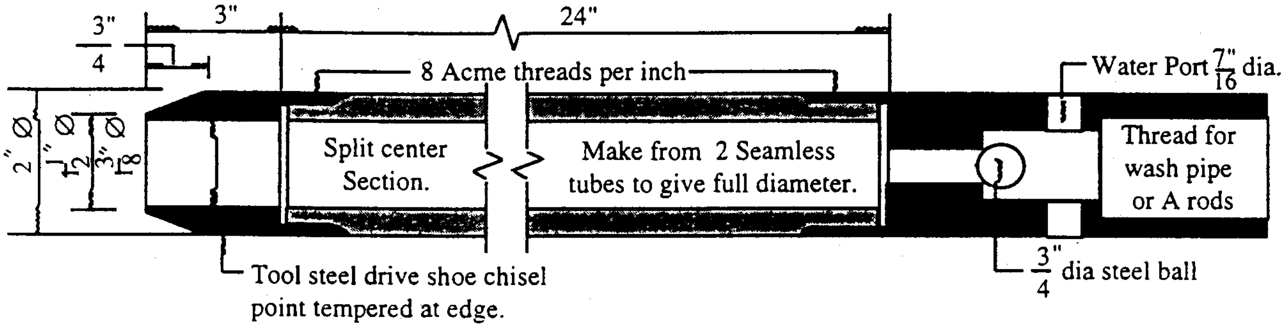


Fig. 4.8 Split Barrel Sampler for Standard Penetration Test

(i) The Validity of SPT

The validity of the SPT which is widely used in different countries and considered as the most economic test method, has been the subject of study and research by many authors/researchers since the test was introduced by Karl Terzaghi in the year 1927. The basic conclusion is that the test results are difficult to reproduce (Bowles, 1988 and Murthy, 1993). Some of the factors that affect the reproducibility are:

1. Variation in the height of free fall of drop weight during the test
2. Interference in the free fall of drop weight by the guide rod which can be out of plumb during the test
3. Diameter and condition (rusty, clean, etc.) of the drum of hand operated winch or cathead
4. The number of turns of rope around cathead or drum of the hand operated winch
5. The condition of Manila rope used for the test, whether new or old etc.
6. Use of badly damaged drive shoe
7. Improper seating of the sampler on the bottom of the hole
8. Effect of isolated stones met during driving,
9. Effect of overburden pressure
10. Carelessness in conducting the test, such as allowing a quick condition in the hole bottom by too rapid withdrawal of auger, or bit plug, or from a differential water level between GWT and in hole.
11. Length and diameter of drill rod.

Possibly there may be many more factors which affect reproducibility and which can not be accounted for. The number of turns around the drum affects the frictional resistance offered for the free fall of weight hammer. It appears that nominal two turns of rope around the drum is the optimum and is widely used. A new rope reduces the frictional resistance whereas an old one increases the resistance.

Of late some research centres and organisations have developed automatic free-fall hammer which eliminates the first five irritants of SPT method mentioned above. Some studies indicate that if SPT is conducted at depths greater than about 60m, the SPT N-value will be greater than the actual value. This discrepancy between the actual and the apparent has been attributed to the weight and flexibility of drill rods. Some other studies indicate that stiffness and weight of the drill rods does not affect the SPT N-values. Inspite of some shortcomings, the SPT is valid and widely used all over the world because of its operational simplicity and economy.

(ii) Standardisation of SPT

It has been suggested by the different authors/researches that if an automatic free-fall weight hammer is not used which is normally the case with most of the organizations, then the first five factors mentioned earlier affect the driving energy. The energy imparted to the sampler depends upon the velocity of the drop weight at the impact level which can be compared to the theoretical velocity or energy. The studies by the researchers indicate that the energy delivered to the sampler decreases with the increase in the number of turns of rope around the cathead. The energy ratio, Re for 1, 2 and 3 turns are given in Table 4.2.

Table 4.2 Energy Ratio for a given number of Turns of Rope around Cathead

No. of Turns	Impact Velocity in cm/sec	Re %
1	318	67
2	312	65
3	254	43

Generally speaking, the use of one or two turns of rope around cathead results in about 66 percent of kinetic energy delivered as compared to about 40 percent for three turns. It has been indicated that the number of turns around cathead should be limited to two turns only. These are only approximate values and the other factors mentioned earlier also affect the energy transfer. By merely changing some of the SPT conditions, wide variations in delivered energy could occur. Large values of energy ratios Re, decrease the blow count N nearly linearly that is, if N=32 for Re 50%, the blow count N for Re = 80% is

$$N = \frac{50}{80} X 32 = 20$$

It has been however, suggested that the impact velocity or energy standard be a criteria for standard penetration test. The selected velocity or energy should be that which least disturbs the existing correlations between N-values and engineering properties, performance of foundations, etc.

It has already been stated earlier that SPT is a very popular in-situ test all over the world. There are variations from country to country in the equipment used and methodology adopted for the test. This has led to a wide scatter of energy ratio R_e and the resulting blow counts N, therefore vary for the same type of strata. It is the opinion of some authors/researchers that the drill system dependent R_e be referenced to a standard energy ratio, R_{es}. In this way a drill rig with, say $R_e = 50$ percent would, on adjustment to the standard energy ratio R_{es}, compute the same N count as from a drill rig with $R_e = 70$ percent. Standard energy ratios R_{es}, suggested during 1983 - 1986 by different authors range from about 55 to 70. Bowles (1988) has opined that he would use R_{es} equal to 70 since the more recent data using current drilling equipment with a safety or an automatic drop hammer and driller attention to ASTM D 1586 details indicate this is close to the actual energy ratio R_e obtained in North American practice.

(iii) SPT Equipment commonly used in Bangladesh

In Bangladesh most drilling organisations use SPT and sampling equipment manufactured locally in private workshops, based principally on relevant ASTM specifications. Again, winch is not generally used for raising and dropping the 63.5 kg (140 lb) driving hammer for measuring SPT N-values, and instead a crew of 3 or 4 laborers pulls the rope over the pulley to raise the driving hammer to an average height of 760mm (30 inch) and suddenly releases the rope to allow the driving hammer a "free fall" from an average height of 760mm (30 inch). So, the local system of measuring SPT N-values may or may not be any closer to the automatic "free fall hammer system" of ASTM D1586.

No research or study for the standardisation of the locally made drilling rigs and energy ratios has been made so far to find out the range of actual hammer energy to the sampler and energy ratio with respect to standard imput energy. Therefore, the N-values as measured by the Drill Rigs made locally claimed to be as per ASTM D1586 details are used in the design purposes without any adjustment for energy ratio as discussed earlier.

(iv) Correction to observed SPT N-values in Cohesionless soils

Two types of corrections were earlier applied to the observed SPT N-values in cohesionless soils. They are:

1. Correction due to dilatancy
2. Correction due to overburden pressure

1. Correction due to dilatancy

In saturated fine or silty dense or very dense sand deposits, the N value may be greater than the actual value because of the tendency of such materials to dilate during shear under undrained conditions. Hence, in such soils, the results of SPT should be interpreted conservatively. For such sand deposits, Terzaghi and Peck (1948) in their 'Soil Mechanics in Engineering Practice' recommended that if the observed N-value is greater than 15, it should be corrected for dilatancy effect as

$$N' = 15 + 0.5(N - 15) \quad \text{.....(4.1)}$$

where, N' = Corrected SPT blow counts

N = Measured SPT blow counts

Bowles (1988) has suggested that this correction is not necessary in view of current research works; and Terzaghi and Peck in their book of 1967 edition did not include the above dilatancy correction probably due to research findings after 1948. So, measured SPT N-values may be used without correction for dilatancy.

2. *Correction due to Overburden Pressure*

The effect of overburden pressure in cohesionless soils has been recognized for long by different researchers/authors (Gibbs and Holtz, 1957; Bazara, 1967; Peck et al, 1974; and Liao and Whitman, 1986). The corrected N-values by the suggested methods of the above authors differ widely in some cases. However, Peck et al (1974) has suggested the following equation for correcting the measured SPT blow counts N:

$$N' = 0.77 \log \frac{2000}{P'_o} N = C_N N \quad \text{.....(4.2)}$$

where, N' = SPT value corrected for overburden pressure

P'_o = effective overburden pressure in kPa

C_N = correction factor

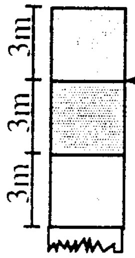
The equation (4.2) has the following characteristics

1. The value of N' approaches infinity as the value of P'_o approaches zero.
2. The value of N' becomes approximately equal to N as the value of P'_o becomes equal to 100 kPa.
3. The value of N' decreases as the value of P'_o increases beyond 100 kPa.

4. The equation is valid for the values of P'_o greater than or equal to 25 kPa.

The method of calculation of effective overburden pressure, P'_o say at a depth of 9m from the ground level, is shown by the accompanying sketch below :

Ground level (GL)



Unit weight of moist sand layer, $\gamma_{moist} = 18$ kN/m^3

Ground water level (GWL)

Unit weight of saturated clay layer, $\gamma_{sat} = 17$ kN/m^3

Unit weight of saturated sand layer, $\gamma_{sat} = 20$ kN/m^3

Unit weight of water, $\gamma_w = 9.81$ kN/m3

Calculation :

(1) Effective overburden pressure due to the top 3 m of moist sand layer above the ground water level = 3 x 18 = 54 kN/m^2

(2) Effective overburden pressure due to the saturated clay layer from 3 to 6 m depth = 3 x (17-9.81) = 21.57 kN/m^2

(3) Effective overburden pressure due to the saturated sand layer from 6 to 9 m depth = 3 x (20-9.81) = 30.57 kN/m^2

Total effective overburden pressure, P'_o at 9m depth of the soil column = 54+21.57+30.57 = 106.14 kN/m^2

A chart for estimating correction factor C_N is shown in Fig. 4.9.

Liao and Whitman (1986) have proposed the following equation for overburden pressure correction of measured blow counts, N :

$$N' = N\sqrt{\frac{P}{P'_o}} = C_N N \quad \ldots\ldots (4.3)$$

where, P_o = standard overburden pressure = 95.75 kPa

C_N values computed by making use of equations by Peck et al (1974), and Liao & Whitman (1986) are given in Table 4.3 for ready use where required.

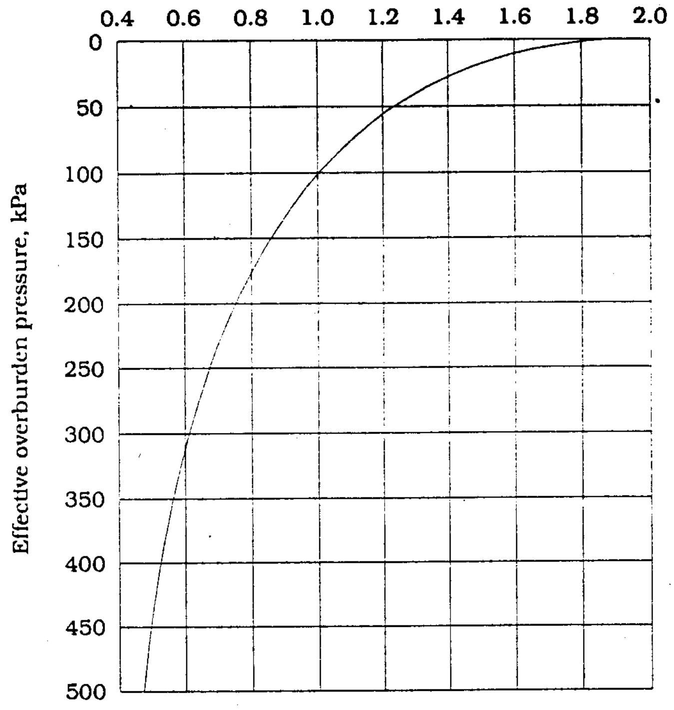


Fig. 4.9 Chart for Correction of N-values in Sand for Influence of Overburden Pressure (After Peck et al 1974)

Table 4.3 Correction Factors C_N

P'_o kPa	Values of C_N as per eqs. by	
	Peck et al (1974)	Liao & Whitman (1986)
0	∞	∞
25	1.46	1.96
50	1.23	1.38
100	1.00	0.98
150	0.96	0.80
200	0.77	0.69
400	0.45	0.49

Any one of the two methods may be used as no significant difference is there in values.

4.5.1.3.2 Static Cone Penetration Test (SCPT)

The Static Cone Penetrometer Test (SCPT) first developed in Holland is popularly known as the Dutch Cone Penetrometer Test. It is a soil sounding device and consists of a cone shaped penetrometer, which is pressed into the ground using a hydraulic jack. The penetrometer is attached to an uncased rod and the penetrometer resistance is read from a hydraulic dynamometer.

Penetration resistance is measured in kg/cm^2 (generally, the same as $ton/ft.^2$)

The spacing between soundings must be based on rational considerations and never on conventions or standardization. Wide pacing intervals may be used for preliminary soundings with additional soundings intermediately placed as needed to definitely identify the soil profile penetration characteristics.

Sounding with SCPT is limited, frequently, to a depth of about 18 to 30m in alluvial and loose to medium dense soils.

The primary use for SCPT data is for design of pile resistance although some information is available which proposes the use of penetrometer data for determining allowable bearing capacity for spread or continuous footings.

One of the greatest value of the SCPT consists of its function as a small scale model pile test. Empirical correlations established over many years permit the calculations of the pile bearing capacity directly from the SCPT results without the use of conventional soil parameters (Murthy, 1993)

Penetrometer of a variety of shapes and sizes are used in different countries. However, the one which is standard in most of the countries is the cone with an apex angle of 60° and a base area of $10cm^2$. The sleeve (jacket) has become a standard item on the penetrometer for most applications. Sketch of the mechanical friction-cone penetrometer is shown on Fig. 4.10.

The cone is pushed into the soil at a rate of 10 to 20mm/sec by hydraulic pressure applied to drill rods extending from the cone to the ground surface. The penetration resistance q_c is determined by dividing the measured force by the $10cm^2$ cone area.

The Begemann friction cone (Begemann 1965) has a friction sleeve mounted on the rods behind the cone. A system of inner and outer rods permits advancing either the cone alone or the cone and sleeve together. Thus both a cone bearing capacity q_c and soil-sleeve friction f_s are measured.

In addition to q_C and f_s, the friction ratio Rf equal to $\frac{f_s}{q_c}(\%)$ is also calculated.

The parameters q_C, f_s and R_f have been correlated with soil characteristics in several ways. The values of q_c and R_f can be used for an approximate identification of the soil type from Fig. 4.11 (Dunn, Anderson and Kiefer, 1980).

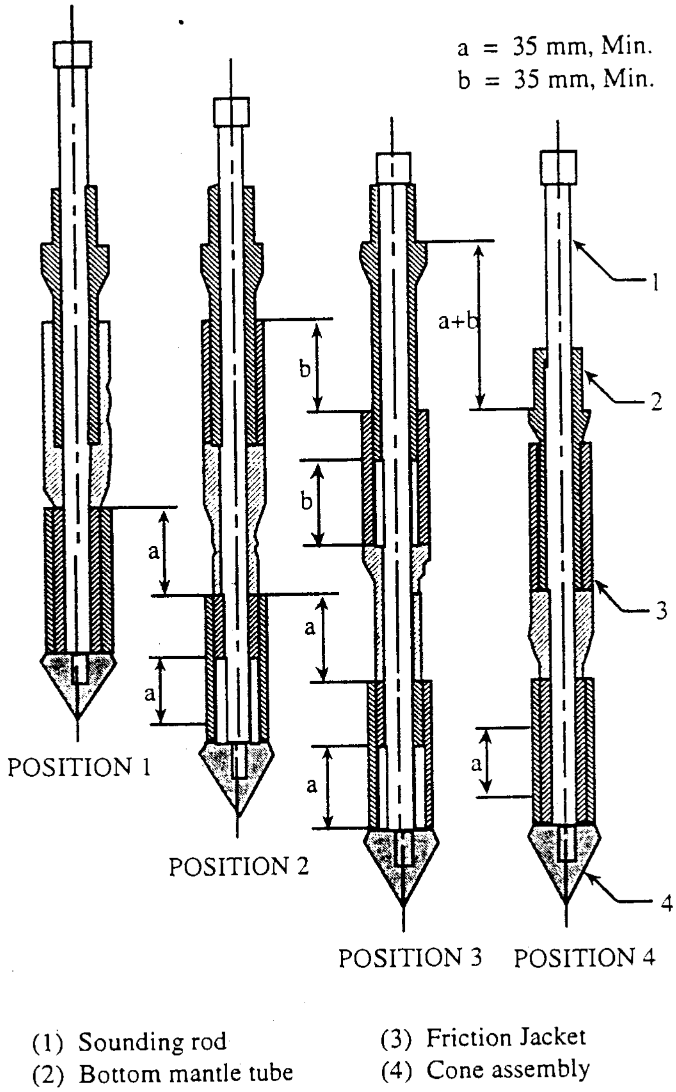


Fig. 4.10 Four Positions of Mechanical Friction Cone Penetrometer

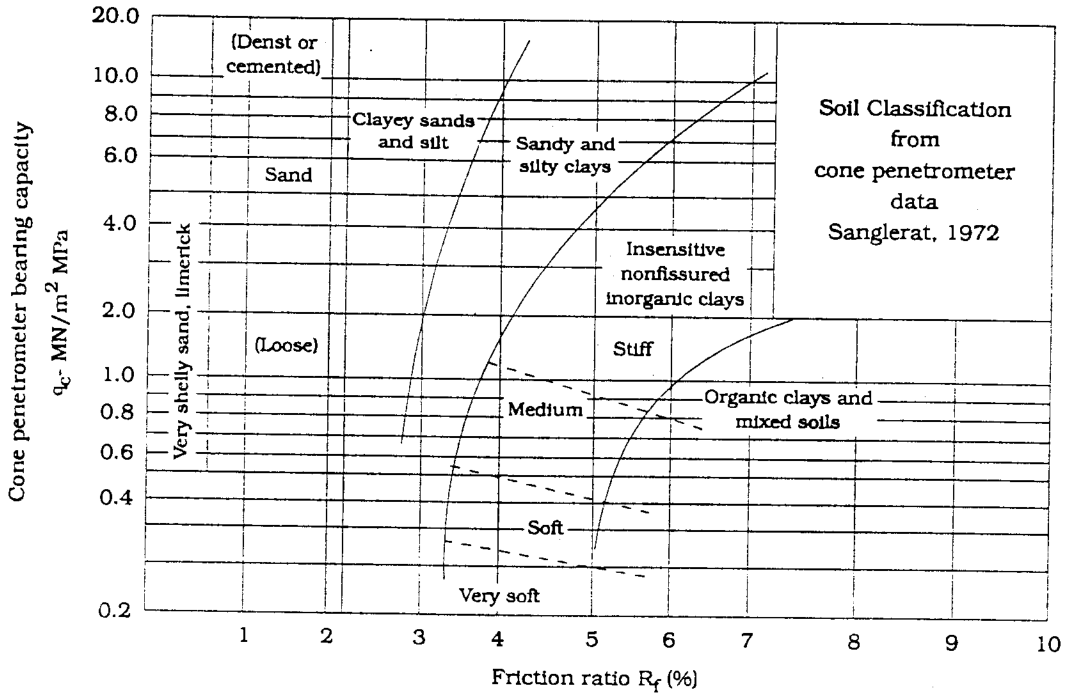


Fig. 4.11 Cone Penetrometer Bearing Capacity

4.5.1.3.3 Pocket Penetrometer Test

Pocket penetrometer can also be used for determining undrained shear strength and unconfined compressive strength in the cohesive soil strata. The pocket penetrometer is a simple, compact and reliable unit for determining an approximate value of the unconfined compressive strength and shear strength of soils. However, this can not replace the laboratory testing of unconfined specimens, due mainly to the small testing area covered by the penetrometer. The pocket penetrometer should be regarded as a simple tool to aid the engineer in exploration and in checking and comparing similar types of soil. The penetrometer scale is calibrated in kg/cm^2 and lb/ft^2 to give a direct reading of unconfined compressive strength.

4.5.1.3.4 Rod Penetration Test

The method is described in Article 4.2.3

4.5.1.4 Plate Bearing Test

There are a number of methods for determining the bearing capacity of soils and weak rocks by the use of a steel plate to which either a continuous load or a constant rate of penetration is applied. Plate bearing tests are made by excavating a pit to the predetermined foundation level or other suitable depth below ground level, and then applying a static load to a plate set at the bottom of the pit. The load is applied in successive increments until failure of the ground in shear is attained or more usually, until the bearing pressure on the plate reaches some multiple, say, two or three times the bearing pressure proposed for the full-scale foundations. The magnitude and rate of settlement under each increment of load is measured. After the maximum load is reached the pressure on the plate is reduced in successive decrements and the recovery of the plate is recorded at each stage of unloading. This method is known as the maintained load test and is used to obtain the deformation characteristics of the ground. Alternatively, the load can be applied at a continuous and controlled rate to give a penetration of the plate of 2.5mm/min. This is known as constant rate of penetration test and is applicable to soils where the failure of the ground in undrained shears is required, as defined by gross settlement of plate; or where there is no clear indication of failure with increasing load, the ultimate bearing capacity is defined by the load causing a settlement of 15 percent of the plate diameter. Tomlinson (1986) mentions that such tests appear to answer all the requirements of foundation design, the method is subject to serious limitations and in certain cases the information given by the tests can be widely misleading. Requirements of apparatus, test procedure and limitations of the plate bearing test have also been described in the Standard Test Method for Bearing Capacity of Soils for Static Load and Spread Footing (ASTM D1194-72, reapproved 1987).

4.5.2 Laboratory Testings

All the disturbed samples collected from the test pits and boreholes should be visually inspected by an experienced geotechnician and representative samples of different soil strata of a borehole should be selected for different classification and index property tests. As per the laboratory tests, the classification of the untested similar disturbed samples should be made. Field classification should be modified as per laboratory classification and field logs should be corrected, where necessary. All the undisturbed samples collected from the cohesive soil strata shall be subjected to classification and identification tests when the samples are tested for shear strength and consolidation settlement parameters.

The tests listed in Table 4.4 should be done in a good geotechnical laboratory to meet the purpose of the Manual.

Table 4.4 List of Soil Tests

Soil Sample Type	Type of Test	ASTM Method	BS Method 1377 : 1975	Remarks
Disturbed soil samples (from test pits and boreholes)	Natural Moisture Content	D 2216	Test 1	For cohesive soils only
	Liquid Limit, Plastic Limit and Plasticity Index	D 4318	Test 2, Test 3, & Test 4	For cohesive soils only
	Grain Size Analysis (Sieve and Hydrometer Method)	D 421 D 422	Test 7	For cohesionless and cohesive soils
	Specific Gravity	D 854	Test 6	For selected cohesionless and cohesive soils
Undisturbed cohesive soil samples (from test pits and boreholes)	Natural Moisture Content	D 2216	Test 1	
	Liquid Limit, Plastic Limit and Plasticity Index	D 4318	Test 2, Test 3 & Test 4	
	Grain Size Analysis (Sieve and Hydrometer Method)	D 421 D 422	Test 7	
	Specific Gravity	D 854	Test 6	
	Unconfined Compression Test	D 2166	Test 20	
	One Dimensional Consolidation Test	D 2466	Test 17	
Bulk samples (disturbed samples) from excavated pits for road component only	Moisture - Unit Weight Relationship (Compaction Test) in addition to the other tests mentioned for disturbed samples (from test pits and boreholes).	D 698 (Standard) D 1557 (Modified)	Test 12 (Standard) Test 13 (Modified)	For bulk samples of soils and aggregates to be used in construction of road/approach road
	California Bearing Ratio Test	D 1883	Test 16	

The above tests of ASTM and BS specifications have been described in details in different laboratory soil testing manuals (Lambe, 1951; Bowles, 1992; Akroyd, 1957; Head, 1984)

4.6 ASSESSMENT OF SOIL TYPES

It is necessary for the foundation engineer to classify the site soils for use as a foundation material. There are several systems of classification of soils based on granular composition and plasticity properties.

The Unified Soil Classification System (USCS) is commonly used in Bangladesh as also in USA for foundation work. It is recommended that the Unified Soil Classification System should be followed under LGED. The general scheme of the classification as shown in Table 4.5 & Fig. 4.12 may be followed.

The term C_u in Table 4.2 is the coefficient of uniformity and C_c is the coefficient of curvature, defined as:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{.....(4.4)}$$

$$C_U = \frac{(D_{60})^2}{D_{60}D_{10}} \quad \text{.....(4.5)}$$

where D_{60}, D_{30} and D_{10} are particle sizes in mm corresponding to 60 percent, 30 percent and 10 percent respectively finer by weight read from the grain size distribution curve.

Natural moisture content (NMC), liquid limit (LL), plastic limit (PL) and plasticity index (PI) are important physical and index characteristics of the fine-grained soils. Plasticity is the ability of a soil to undergo irrecoverable deformation at constant volume without cracking or crumbling. The upper limits and lower limits of the range of water content over which a soil exhibits plastic behaviour are defined as the liquid limit and plastic limit respectively. The water content range is defined as the plasticity index, that is

$$PI = LL - PL \quad \text{.....(4.6)}$$

Procedures of determining LL, PL and PI have been referred to in section 4.5.2 under Laboratory Testings. Liquid limit and plasticity index data are used in the Plasticity Chart on Fig. 4.12 which is used for classification of fine-grained soils. The axes of plasticity chart are plasticity index and liquid limit. After these parameters are determined in the laboratory, the plasticity characteristics of a fine-grained soil can be represented by a point on the chart. Classification symbols (CL, ML, MH, OL etc.) are assigned to the soil according to the zone within which the point lies in the chart of Fig. 4.12. The boundary lines for different zones are given by A-line and the LL= 50 % line. Soils above A-line are clays (symbol C) and below A-line are silts (symbol M) or organic soils (symbol O). If LL is more than 50%, it is referred to as highly plastic (symbol H) and less than 50%, it is of low plasticity (symbol L). The UL (upper limit) in Fig. 4.12 means that no soil has so far been found with coordinates that lie above the upper limit or U line of the Plasticity Chart. The line is based on Corps of Engineers findings.

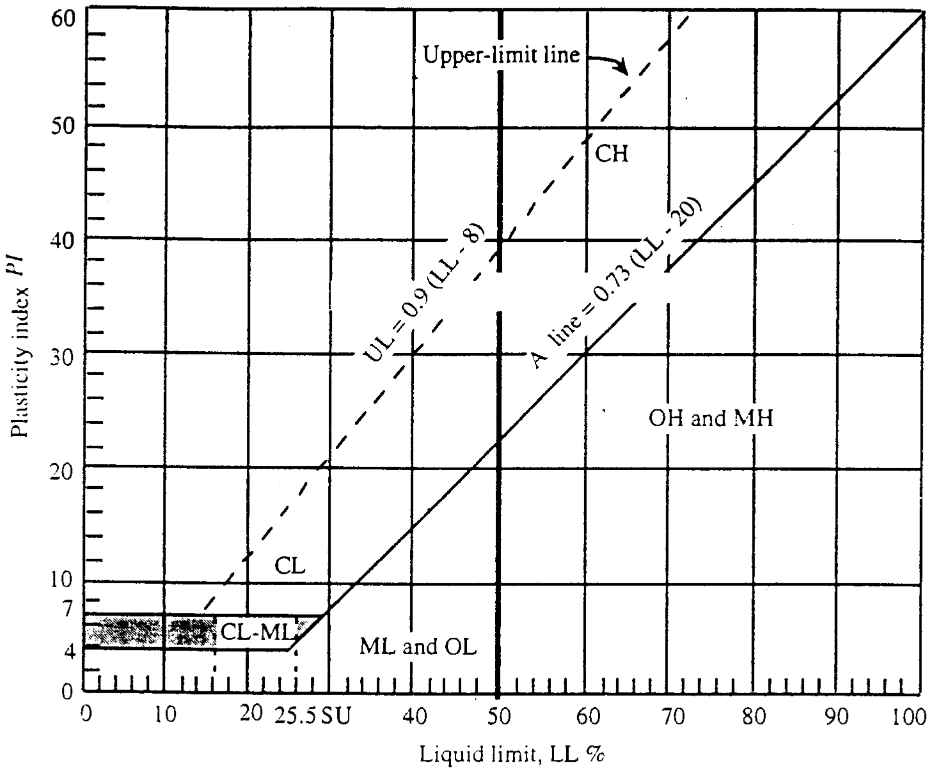


Fig. 4.12 Plasticity Chart (also called "Casagrande's A-chart") to use with the Unified Soil Classification System.

By Unified Soil Classification System the soil can be broadly identified into several groups of non-cohesive and cohesive soils based on grain size distribution and liquid and plastic limits determined in the laboratory :

Non-cohesive soils : GW, GP, GM, GC, SW, SP, SM, SC

Cohesive soils : ML, CL, OL, MH, CH, OH

Peat and other highly organic soils are assigned the symbol Pt.

Some clayey sand (SC) soils may behave as cohesive soil if the clay content is very high (i.e. greater than 20%).

Table 4.5 The Unified Soil Classification System

Major Divisions			Group Symbol	Typical Names	Classification Criteria for Coarse-grained Soils	
Coarse-grained Soils (more than half of material is larger than No. 200)	Gravels (more than half of coarse fraction is larger than No. 4 sieve size)	Clean Gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u \geq 4$ $1 \leq C_c \leq 3$	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW ($C_u < 4$ *or* $1 > C_c > 3$)	
		Gravels with fines (appreciable amount of fines)	GM $\frac{d}{u}$	Silty gravels, gravel-sand-sile mixtures	Atterberg limits below A line or $I_p < 4$	Above A line with $4 < I_p < 7$ are boderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixture	Atterberg limits above A line or $I_p > 7$	
	Sands (more than half of coarse fraction is smaller than No. 4 sieve size)	Clean Sands (little or no fines)	SW	Well-graded sands, gravelly-sands, little or no fines	$C_u \geq 6$ $1 \leq C_c \leq 3$	
			SP	Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW ($C_u < 6$ *or* $1 > C_c > 3$)	
		Sands with fines (appreciable amount of fines)	SM $\frac{d}{u}$	Silty sands, sand-silt mixtures	Atterberg limits below A line or $I_p < 4$	Limits plotting in hatched zone with $4 \leq I_p \leq 7$ are boderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Atterberg limits above A line or $I_p > 7$	
Fine-grained Soils (more than half of material is larger than No. 200)	Silts and Clay (liquid limit < 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	1. Determine percentages of sand and gravel from grain-size curve. 2. Depending on percentages of fines (fraction smaller the 200 sieve size), coarse-grained soils are classified as follows: Less than 5% - GW, GP, SW, SP More than 12% - GM, GC, SM, SC 5 to 12% - Borderline cases requiring dual symbols.	
			CL	Inorganic clay of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
	Silts and Clay (liquid limit > 50)		OL	Organic silts and organic silty clays of low plasticity	$C_u = \dfrac{D_{60}}{D_{10}}$ $C_c = \dfrac{D_{30}^2}{D_{10}\,D_{60}}$	
			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			CH	Inorganic clays or high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity, organic silts		
	Highly Organic Soils		Pt	Peat and other highly organic soils		

In the absence of facilities for performing grain size analysis and limit tests, the criteria shown in Table 4.6 may be used to determine whether a soil is cohesive or non-cohesive :

Table 4.6 Criteria for Determining Type of Soil

Type of soil	Characteristics
Non-cohesive	Some of the individual grains will be visible to naked eye, rough granular appearance, in dry condition when squeezed in hand and released will not form into a ball. In wet condition may form a ball but will crumble when lightly touched. Can not be riboned without crack
Cohesive	Particle too small to be seen; lumps difficult to break between fingers; sticks to hand when wet. In dry condition forms a stiff lump which can be freely handled without breaking. Can be formed into long thin ribbons and also worked into a compact ball.

Organic soils can be identified by pungent smell and the presence of decayed grass, plant remains and by a black or grey colour. This type of soil is unsuitable for foundation support and therefore should be removed and replaced by granular soils before placement of foundation. In case of the presence of this soil to a great depth special foundation treatments are required.

4.7 ASSESSMENT OF UNIFORMITY IN SOIL CONDITION

An approximate but quick method of checking uniformity of soil below any foundation is to perform rod penetration test at several points within the proposed foundation area. In case of equal or nearly equal penetration, the soil condition may be assumed uniform. In case of widely varying penetration (say of the order of 1m (3 ft) or more), the softer layers where penetration is high may require to be removed or special foundation treatment may become necessary.

4.8 ASSESSMENT OF BEARING CAPACITY

The first step in determining bearing capacity of a uniform soil layer is to determine the available shearing resistance parameters for the soil. As these parameters are different for cohesive and non cohesive soils, it is essential to establish first whether the soil is cohesive or noncohesive based on field and laboratory test results as discussed earlier.

4.8.1 Cohesive Soils

The shear strength of a cohesive soil may be estimated by any of the following methods :

1. By rod penetration test
2. By standard penetration test
3. By static cone penetration test
4. By laboratory unconfined compression test on undisturbed soil samples
5. By laboratory direct/triaxial shear tests on undisturbed soil samples

In all the above methods, the minimum unconfined compressive strength, q_u or minimum shear strength, S_u or cohesion of the soil is to be obtained at worst possible condition (i.e. at saturated submerged state). The first two methods provide an approximate value of S_u, and the third provides the value of S_u as indicated in equation 4.9 and as discussed in Article 4.5.1.3.2, while the fourth & 5th are improvements depending upon the quality of sampling. The tests under method 5 are not recommended for the small structures considered in this manual.

4.8.1.1 By Rod Penetration Test

In the rod penetration method, a plain steel rod of known diameter of 10 to16mm (3/8 to 5/8 inch) is pushed into the ground by a person of known weight and the extent of penetration is recorded. Chart shown in Fig. 4.13 may be used to estimate the shear strength S_u of the soil from the penetration value. For example, if a penetration of 400mm (16 inch) is obtained by a person weighing 55 kg (120 lbs) using a 5/8 inch diameter rod, then the approximate shear strength of the soil is 25 kN/m^2 (521 psf).

4.8.1.2 By Standard Penetration Test

For assessment of consistency of saturated cohesive soil from SPT results of cohesive soil strata Bowles (1988) has suggested (Table 4.7) a correlation of consistency against SPT value N'_{70}, where N'_{70} indicates that measured N which has been adjusted with respect to some energy ratio R_e and to standard energy ratio R_{es} as discussed earlier under Article 4.5.1.3.1 (Standardisation of SPT). But in Bangladesh this adjustment is not yet possible. So, it is general practice to use the measured N value.

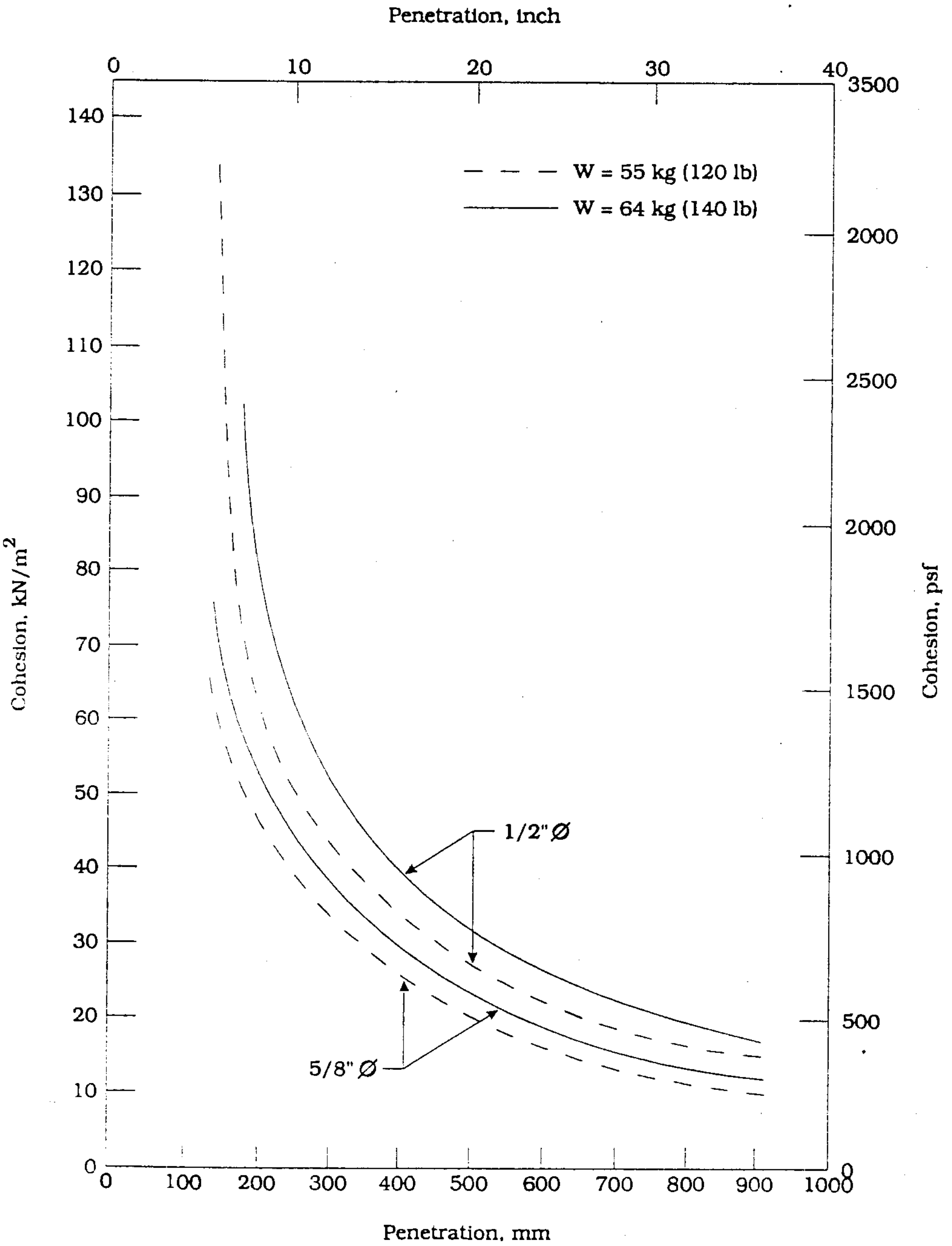


Fig. 4.13 Rod Penetration Chart

Table 4.7 Consistency of Saturated Cohesive Soils (Bowles, 1988)

Consistency		N'_{70}	Remarks
Very soft	* NC; Young clay	0 - 2	Squishes between fingers when squeezed
Soft		3 - 5	Very easily deformed by squeezing
Medium		6 - 9	
Stiff	Increasing **OCR; Aged/ cemented	10 - 16	Hard to deform by hand squeezing
Very stiff		17 - 30	Very hard to deform by hand
Hard		> 30	Nearly impossible to deform by hand

Note : Blow counts and overconsolidation ratio (OCR)division are for a guide—in clay "exceptions to the rule" are very common.

** NC - Normally Consolidated - A soil is said to be normally consolidated (NC) if the current stress state is the largest to which the soil mass has been subjected ; i.e. the effective present overburden pressure P'_0 is the maximum pressure on an element in the soil mass.*

*** OCR - The soil is said to be overconsolidated (or preconsolidated) if the compression behaviour is as if a column of soil larger than the present one has compressed the soil element at some time during the geological history of the mass. The concept of preconsolidation is of considerable importance for cohesive soils but is not given much importance for cohesionless soils. The overconsolidation ratio (OCR) is defined as the ratio of the preconsolidation pressure P_c to the effective present overburden pressure $P'o$:* $OCR = \dfrac{P_c}{P'_o}$

Terzaghi & Peck (1967) have provided for saturated cohesive soils, correlations between SPT N-value, consistency and unconfined compressive strength, q_u, as shown in Table 4.8. This correlation is quite useful but has to be used according to the soil conditions met in the field. It is mentioned that unconfined compressive strength determined in the laboratory on undisturbed soil samples taken from the cohesive soil strata is more reliable than the strength estimated from SPT N-values.

Table 4.8 : Correlation between SPT N-Value and Unconfined Compressive Strength, q_u

Consistency	SPT N-value	Unconfined Compressive Strength, qu (kPa)
Very soft	0 - 2	< 25
Soft	2 - 4	25 - 50
Medium stiff	4 - 8	50 - 100
Stiff	8 - 15	100 - 200
Very stiff	15 - 30	200 - 400
Hard	> 30	> 400

Ref : Terzaghi and Peck (1967), page 347. *(1 ton/sq.ft Å 100 kPa)*

Some authors in their recent publications of geotechnical engineering text books and manuals have suggested that the SPT N-value to be used in the above table should be the blow count normally corrected for standard energy ratio R_{es}, which has been discussed elsewhere. But in Bangladesh SPT N-value as measured is commonly used.

The present practice is to relate q_u with SPT N-value as follows :

$$q_u = \bar{\kappa}N \qquad (4.7)$$

$$\qquad (4.8)$$

where $\bar{k}$ is proportionality factor.

Values of $\bar{k}$ determined by different researchers of various countries/regions varied over a great range. Relations suggested by Terzaghi and Peck (1948) give a value of about 12.5 for $\bar{k}$ when q_u is expressed in kPa. Values of $\bar{k}$ suggested by several other authors of different countries range from about 6 to 25.

A plot of several hundred test results of q_u of Bangladesh clays and plastic silts against corresponding SPT N-values determined by local made SPT equipment and local procedure of pulling and releasing the drop weight by a crew of labourers indicate an average value of 16.8 for $\bar{k}$, when q_u is expressed in kPa (Serajuddin and Chowdhury, a paper under submission to IEB Convention of 1996). The authors have obtained also the following correlations for cohesive soils of various liquid limit ranges as below :

(a) $\bar{k} = 14.3$ for soil samples with liquid limit $\leq 35\%$,

(b) $\bar{k} = 16.9$ for soil samples with liquid limit >36 to 50 %

(c) $\bar{k} = 17.8$ for soil samples with liquid limit > 51%

4.8.1.3 By Static Cone Penetration Test

The undrained shear strength S_u of cohesive soils (plastic silt & clay) can be estimated from cone penetration data. Schmertmann (1975) suggested that $S_u = \dot{f}_s$ is lower limit for the undrained strength. The following equation has been suggested for S_u by Dunn, Anderson and Kiefer (1980) :

$$S_u = \frac{q_c - \gamma Z}{N_c} \qquad (4.9)$$

where γz is the total overburden pressure at the depth, z at which q_c is measured and γ is the unit weight of soil. The factor N_c has been shown to vary from 5 to 70; however, for recent clay deposits of low sensitivity, a plasticity index less than 10% and an overconsolidation ratio of less than 2, an N_c value of 16 has been recommended by Schmertmann, 1975 (as quoted by Dunn et al, 1980). It is suggested that as a better procedure, a few undrained shear tests to measure values of S_u should be used in equation 4.9 to better ascertain the correct value for N_c for Bangladesh clays and plastic silts.

It is mentioned that a comparative study between q_c and actual q_u (q_u determined on undisturbed cohesive silt-clay soils collected in Shelby tube by drill hole method adjacent to the location of SCPT) has indicated that average value of unconfined compressive strength q_u of cohesive soils of coastal embankment project area of Bangladesh is about 0.50 kg/cm^2 corresponding to a static cone resistance value q_c of 10 kg/cm^2. This means a reduction factor of 20 to estimate q_u from q_c and reduction factor of 40 to estimate cohesion, C (=S_u) from q_c are required (Serajuddin, 1969).

Dunn et al (1980) has also suggested that the cone penetrometer data is useful for identifying compressible layers that will settle excessively when loaded. Suggested criteria for identifying compressible layers is

$$q_c < 1\frac{MN}{m^2} \text{ (Sanglerat, 1972)} \qquad (4.10)$$

4.8.1.4 By Unconfined Compression Test

Bearing capacity of cohesive soil can also be calculated from the unconfined compressive strength determined by laboratory tests of undisturbed cohesive soil samples.

It has been shwon that for footings on clay and plastic silts (ϕ = 0 condition) allowable soil pressure q_a with safety factor of 3 is as under :

$$q_a = \frac{CN_c}{3} \qquad (4.11)$$

or

$$q_a = \frac{q_u N_c}{6} \qquad (4.12)$$

where q_u = unconfined compressive strength, and

N_c= bearing capacity factor (after Skempton, 1951)

The values N_C for square and circular footing and continuous footing may be taken from Fig. 4.14. Because of the variations that normally occur even in relatively uniform clay and plastic silt deposits, the value of q_u in equation 4.12 should represent the average for a depth B (width of the footing) below the base of the footing. If the cohesive soil layer is not fairly uniform but instead a soft layer is located within a depth B below the base of the footing, the strength of the soft layer is likely to determine the factor of safety of the footings (Peck et al, 1974).

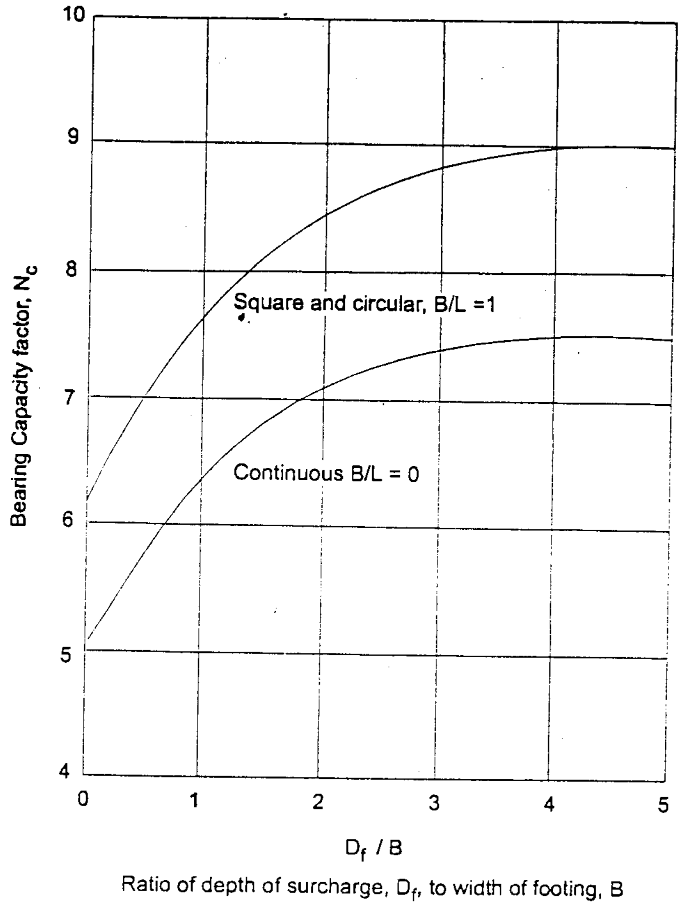


Fig. 4.14 Bearing Capacity Factors for Foundations on Clay under $\phi = 0$ Conditions (after Skempton, 1951)

4.8.2 Cohesionless Soils

4.8.2.1 By Standard Penetration Test

The SPT of cohesionless soil strata has been used in correlations for unit weight, relative density, D_r and angle of internal friction, ϕ by different researchers/authors. One such correlation suggested by Bowles (1988) is shown in Table 4.9. Because N'_{70} can not be determined in Bangladesh as discussed earlier, measured N may be used in place of N'_{70} in Table 4.9 for estimation of different soil parameters of the table.

Table 4.9 Empirical Values for ϕ, D_r, and Unit Weight γ of Granular Soils Based on the SPT at about 6 m Depth and Normally Consolidated

Description	Very loose	Loose	Medium	Dense	Very dense
Relative density D_r	0	0.15	0.35	0.65	0.85
SPT N'_{70}: fine	1 - 2	3 - 6	7- 15	16- 30	
medium	2 - 3	4 - 7	8 - 20	21 - 40	> 40
coarse	3 - 6	5 - 9	10 - 25	26 - 45	> 45
ϕ : fine	26 - 28	28 - 30	30 - 34	33 - 38	
medium	27 - 28	30 - 32	32 - 36	36 - 42	< 50
coarse	28 - 30	30 - 34	33 - 40	40 - 50	
γ_{wet}, pcf	70 - 100	90 - 115	110 - 130	110 - 140	130 - 150
(kN/m^3)	(11 - 16)	(14 - 18)	(17 - 20)	(17 - 22)	(20 - 23)

Note : Excavated soil or material dumped from a truck will weigh 11 to 14 kN/m^3 and must be quite dense to weigh much over 21 kN/m^3. No existing soil has a D_r = 0.00 nor a value of 1.00—common ranges are from 0.3 to 0.7.

Another correlation by Peck, Hanson and Thornburn (1974) is presented in Fig. 4.15 to estimate angle of shearing resistance ϕ and Terzaghi's bearing capacity factors N_q and N_γ which are required for the assessment of the bearing capacity of the soils by using different bearing capacity equations. We shall briefly describe here the bearing capacity equations of Terzaghi and those of Meyerhof.

It is mentioned that the soil must be capable of carrying the loads from any engineered structure placed upon it without a shear failure and with the resulting settlements being tolerable for that structure. Bearing capacity equations as are described below are concerned with the evaluation of the limiting shear resistance, or ultimate bearing capacity, q_{ult} of the soil to foundation load. To obtain allowable bearing capacity q_a with respect to shear failure the ultimate bearing capacity, q_{ult} of the soil will have to be divided with a safety factor (SF),

$$q_a = \frac{q_{ult}}{SF} \qquad (4.13)$$

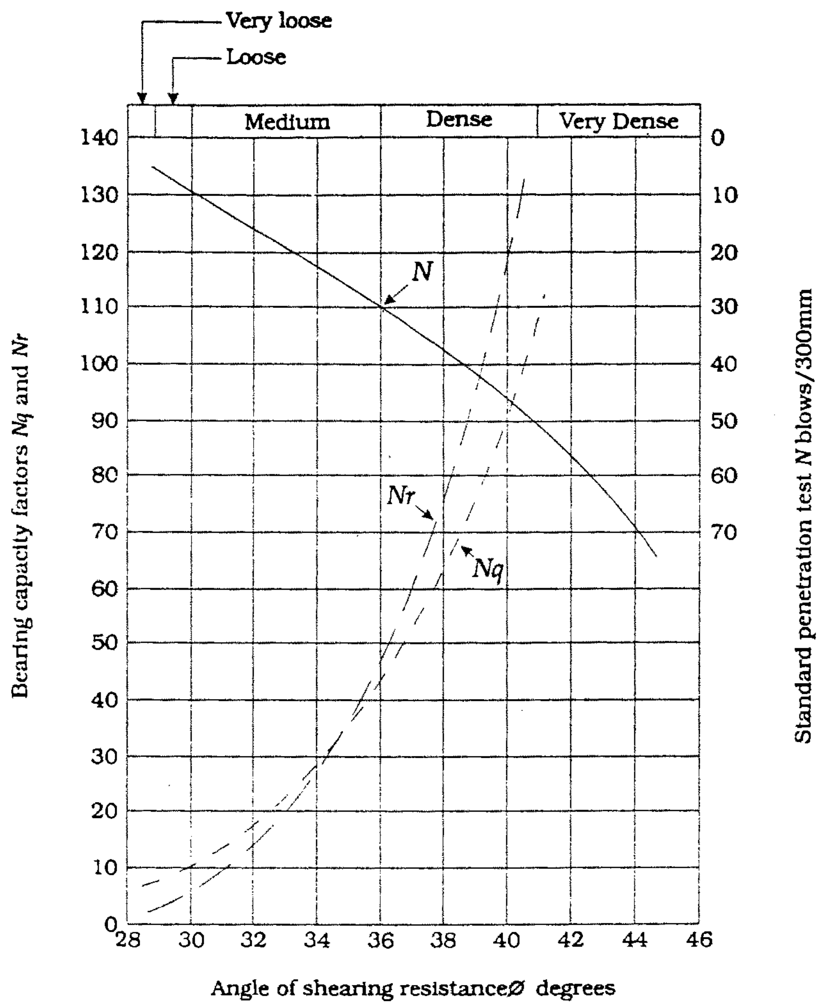


Fig. 4.15 Relationship between ϕ, Bearing Capacity Factor, and N-Values from the Standard Penetration Test (Peck, Hanson, and Thornburn, 1974)

The safety factor is based on the type of soil (cohesive or cohesionless), reliability of the soil parameters, structural information (importance, use, etc.) and experience and judgement of the geotechnical engineer/consultant. However, SF = 2.5 to 3 is generally suggested for footing foundations to compute allowable bearing capacity q_a against shear failure.

4.8.2.1.1 Terzaghi's Bearing Capacity Equations

Karl Terzaghi as early as in 1943 suggested sets of bearing - capacity equation of the general form as indicated below for different types of foundation :

$$q_{ult} = CN_c s_c + \bar{k} N_q + 0.5\gamma B N_\gamma s_\gamma \quad \text{..(4.14)}$$

For strip foundation $s_c = 1.0$, $s_\gamma = 1.0$
For round foundation $s_c = 1.3$, $s_\gamma = 0.6$
For square foundation $s_c = 1.3$, $s_\gamma = 0.8$

where,

q_{ult} = ultimate bearing capacity of the supporting footing foundation,

C = cohesion of the soil at the base of the footing,

$\bar{q} = \gamma D$, in which γ is unit weight of soil and D is the depth of foundation from the ground surface to the base of the footing,

B = width or diameter of the footing,

s_c and s_γ are shape factors, and

N_c, N_q and N_γ are Terzaghi's bearing capacity factors.

The values of Terzaghi's bearing capacity factors for different values of angle of internal friction, ϕ are presented in Table 4.10 for direct estimation from SPT N-values in addition to the values of N_q and N_γ shown in Fig. 4.15.

Terzaghi's bearing capacity equations were intended for "shallow" foundations where D (the depth of foundation) would be less than or equal to B (the width of foundation).

Table 4.10 Bearing-Capacity Factors for the Terzaghi Equations

ϕ^o	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	1.5
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
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
50	347.5	415.1	1153.0

4.8.2.1.2 Meyerhof's Bearing Capacity Equations

Meyerhof (1951, 1963) proposed bearing capacity equations similar to those of Terzaghi but included a shape factor for the depth term N_q. He also included depth factors and inclination factors for cases where the footing load is inclined from the vertical. The following equations of the general form were suggested :

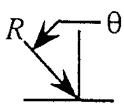
Meyerhof's Bearing Capacity Equations (including Shape, Depth and Inclination Factors)

Vertical load : $q_{ult} = CN_c\, s_c\, d_c + \bar{q}\, N_q\, s_q\, d_q + 0.5\gamma BN_\gamma\, s_\gamma\, d_\gamma$ ....(4.15)

Inclined load : $q_{ult} = CN_c\, d_c\, i_c + \bar{q}\, N_q\, d_q\, i_q + 0.5\gamma BN_\gamma\, d_\gamma\, i_\gamma$ ....(4.16)

The equations for calculation of the values of shape factors s_c, s_q & s_γ ; depth factors d_c, d_q & d_γ ; and inclination factors i_c, i_q & i_γ are presented in Table 4.11.

Table 4.11 Shape, Depth, and Inclination Factors for the Meyerhof Bearing-Capacity Equations (Bowles, 1986)

Factors	Value	For
Shape :	$s_c = 1 + 0.2K_p\frac{B}{L}$ $s_q = s_\gamma = 1 + 0.1K_p\frac{B}{L}$ $s_q = s_\gamma = 1$	Any ϕ $\phi > 10^o$ $\phi = 0$
Depth :	$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$ $d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B}$ $d_q = d_\gamma = 1$	Any ϕ $\phi > 10^o$ $\phi = 0$
Inclination : 	$i_c = i_q = \left(1 - \frac{\theta^o}{90^o}\right)^2$ $i_\gamma = \left(1 - \frac{\theta^o}{\phi^o}\right)^2$ $i_\gamma = 0$	Any ϕ $\phi > 0$ $\phi = 0$

where K_N = $\tan^2(45+\phi/2)$
θ = angle of resultant measured from vertical without a sign
B = width of foundation
L = length of foundation
D = depth of foundation
s = shear strength

There are several other bearing capacity equations suggested by other researchers. Terzaghi's equations being the first proposed have been very widely used. They are still used by some (may be many) because of the greater ease from not having to compute all the extra shape, depth, and other factors. They are only suitable for a concentrically loaded horizontal footing which is most often the case. They are not applicable for columns with moment or tilted base. There is some opinion that Terzaghi equations are overly conservative and probably are for soils with little cohesion and with D on the order of $\frac{B}{2}$ to 2B (Bowles, 1988).

The following general observations about the bearing capacity equations may be useful:

1. The cohesion term predominates in cohesive soil.
2. The depth term ($\bar{q}N_q$) predominates in cohesionless soils. Only a small D increases q_{ult} substantially.
3. The base width term $0.5\gamma\ BN_\gamma$ provides some increase in bearing capacity for both cohesive and cohesionless soils.
4. The footings should not be placed on a cohesionless soil mass with a relative density, D_r less than 0.5. If the soil is loose, it should be compacted in some manner to a higher density prior to placing footings in it.

4.8.2.1.3 Effect of Water Table on Bearing Capacity

The effective unit weight of the soil is used in the bearing capacity equations for computing the ultimate capacity.

The terms $\bar{q}\,N_q$ and $0.5\gamma\,BN_\gamma$ use the effective unit weight of the soil mass. So, the presence of a water table around a footing reduces the effective shear strength of a granular soil and, hence, its bearing capacity is also reduced. For a fully submersed footing the unit weight of soil used with the $\bar{q}N_q$ and $0.5\gamma\,BN_\gamma$ terms of the bearing capacity equation is the buoyant unit weight (saturated unit weight of soil - unit weight of water). Since the buoyant unit of soil is about one-half the moist unit weight, the bearing capacity of a submersed footing is about one-half that of the footing well above the water table. If the water table is a distance B below the footing, it is assumed to have no effect on bearing capacity. When the water table is at the base of the footing, the buoyant unit weight should be used only with the $0.5\gamma\,BN_\gamma$ term.

For water table positions intermediate between the ground surface and base of the footing or between the base of the footing and a distance B below the footing, adjustment of the respective unit weight should be made by linear interpolation (Peck et al, 1974, Dun et al, 1980, Bowles,1988 and Murthy 1993).

4.8.2.2 By Static Cone Penetration Test

Several methods are available for evaluating the friction angle, ϕ corresponding to the drained shear strength of sands from cone penetration resistance q_c.

Meyerhof (1956) established an average correlation between the standard penetration resistance, N and the static cone penetration resistance, q_c (kg/cm^2) for cohesionless soils as under :

$$q_c = 4N \quad \text{.........} \quad (4.17)$$

From the correlation Meyerhof (1956) also suggested the relation between relative density, penetration resistance and angle of internal friction of the cohesionless soils as shown in Table 4.12.

Table 4.12 Relationship between Relative Density, Penetration Resistance and Angle of Internal Friction of Cohesionless Soils

State of Packing	Relative Density, Dr	SPT N - Value (Blows per 300 mm Penetration)	Static Cone Resistance, q_c (kg/cm^2)	Angle of Internal Friction, ø (Degrees)
Very loose	< 0.2	< 4	< 20	< 30
Loose	0.2 - 0.4	4 - 10	20 - 40	30 - 35
Compact	0.4 - 0.6	10 - 30	40 - 120	35 - 40
Dense	0.6 - 0.8	30 - 50	120 - 200	40 - 45
Very Dense	> 0.8	> 50	> 200	> 45

Another such relationship suggested by Meyerhof (1974) is shown in Fig. 4.16 (Dunn et al, 1980). The value of q_c used is the limiting or maximum cone resistance measured at a critical depth below which the penetration pressure shows no increase with further penetration.

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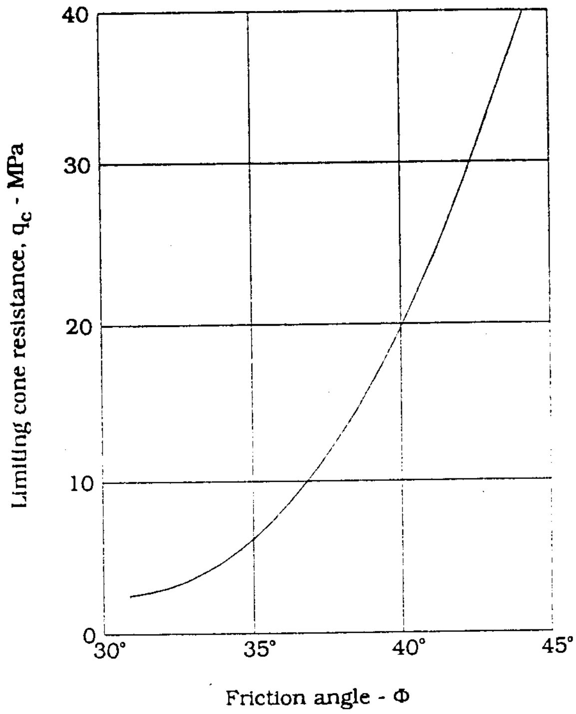

Fig. 4.16 Limiting Cone Resistance as a Function of Friction Angle
(After Meyerhof, 1974)

Meyerhof (1956) has also suggested simplified formulas for determining allowable bearing pressures to ensure that a settlement of 25mm is not exceeded. His formulas are applicable to pad or strip foundations of fairly small dimensions on dry sands as follows :

For square or strip foundations equal to or less than 1.2m wide, the allowable bearing pressure, q_a is given by

$$q_a = 3.6\, q_c \text{ kN/m}^2 \approx \frac{q_c}{30} \text{ kg/cm}^2 \quad \text{.......... (4.18)}$$

where q_c = cone resistance in kg/cm^2.

For square or rectangular foundations greater than 1.2m wide, the allowable bearing pressure is given by

$$q_a = 2.1\, q_c \left(1+\frac{1}{B}\right)^2 \text{ kN/m}^2 \approx \frac{q_c}{50} \left(1+ \frac{1}{B}\right)^2 \text{ kg/cm}^2 \quad \text{.......... (4.19)}$$

where B = width of the foundation.

An approximate formula to cover all foundations irrespective of width is

$$q_a = 2.7\, q_c\ \text{kN/m}^2 \approx \frac{q_c}{40}\ \text{kg/cm}^2 \qquad \text{.......... (4.20)}$$

Values of q_a calculated from formulas (4.18) to (4.20) above should be halved if the sand within the stressed zone is submersed. Meyerhof suggests that the allowable bearing pressures given by his formulas may be doubled for raft or pier foundations. It is seen that the settlements of the foundations in cohesionless soils have been taken care of in the above formulas of Meyerhof.

4.9 SETTLEMENT ANALYSIS

The allowable bearing pressure imposed by a foundation is a function of the characteristics of the ground, the depth and dimensions of the foundation and the degree of settlement which can be tolerated by the structure or its installations. There are two approaches to the estimation of allowable bearing pressures. First, from a knowledge of the shear strength characteristics of the soil, determined by field and laboratory tests described earlier, the ultimate bearing capacity, q_{ult} of the soil can be calculated for a foundation of given depth, D, and dimensions (length, L and width or diameter, B). An arbitrary safety factor, based on the judgement and experience of geotechnical engineer/consultant, can then be applied to the calculated ultimate bearing capacity to give the presumed bearing value. If it can be shown from experience or by calculation that the settlements given by a foundation pressure equal to the presumed bearing value are not excessive for the type and function of the structure, then the allowable bearing pressure (q_a) can be taken as equal to the presumed bearing value. If the settlement is excessive then a lower value will have to be taken for the allowable bearing pressure. The second approach is to determine the allowable bearing pressure from experience and a knowledge of the characteristics of the ground or by empirical methods based on the results of the certain types of in-situ test made on the soil.

4.9.1 Cohesive Soils

4.9.1.1 By Laboratory Test Results and Empirical Equations

Compressibility can be determined from the laboratory consolidation test parameters of undisturbed soil samples. In absence of laboratory test data, compression index, C_c required in the design for calculation of settlements of a structure overlying a compressible soil layer, can be estimated from the approximate correlation (Terzaghi and Peck, 1967) as shown below :

$$C_c \sim 1.30\, C'_c = 0.009\,(LL-10\,\%), \qquad (4.21)$$

where C'_c represents the compression index for the clay in remoulded state and has the following correlation with liquid limit (LL) :

$$C'_c = 0.007\,(LL-10\%) \qquad (4.22)$$

Serajuddin and Ahmed (1967) obtained the following correlation between C_c and LL for Bangladesh clays and plastic silts :

$$C_c = 0.0078\,(LL-14\,\%) \qquad (4.23)$$

If the C_c for a given layer of clay is known, either from laboratory consolidation test on the undisturbed cohesive soils or from the empirical correlations, the compression of the layer due to a surcharge Δp can be computed by means of the following equation:

$$S = H\frac{C_c}{1+e_o}\log_{10}\frac{P_o+\Delta P}{P_o} \qquad (4.24)$$

where S is the compression of a confined stratum of normally load ordinary clay, H is the thickness of the bed of clay under a pressure p_o and Δp is the difference of pressure for an increase of the pressure from p_o to $p_o + \Delta p$ and $\frac{C_c}{1+e_o}$ is termed the compression ratio. Knowing the natural moisture content of the saturated clay and plastic silts it is possible to estimate the compression ratio from the graphical relation between compression ratio and natural water content as found by R E Fadum (quoted by Terzaghi and Peck, 1948 and 1967) for soil of other countries and the graphical relation found by Serajuddin and Ahmed (1967) for clays and plastic silts of Bangladesh. The relationship is presented in Fig. 4.17.

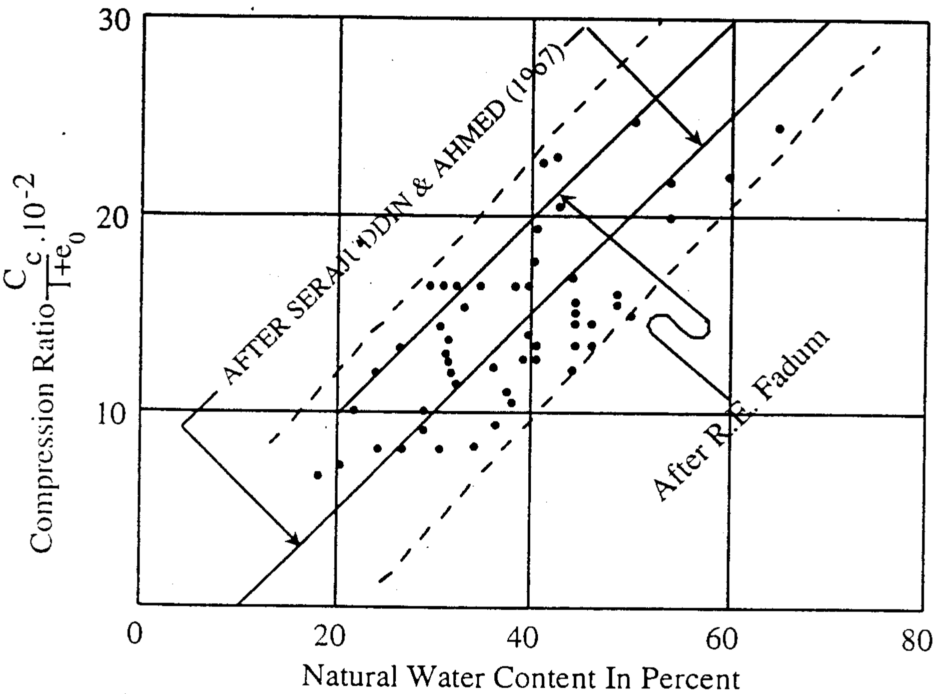

Fig. 4.17 Compression Ratio vs Natural Water (W_N)Content

Some overseas researchers established approximate relationship of natural moisture content, W_N of saturated cohesive soils with compression index, C_c for soils of their studies. Serajuddin (1969) made a correlation study between natural moisture content and compression index of a large number of undisturbed cohesive soils of coastal districts of Bangladesh and obtained the following approximate relationship.

$$C_c = 0.0135\ (W_N - 20) \quad \text{..........} (4.25)$$

Serajuddin (1987) made further study and plotted compression index, C_c values against natural water content of 130 cohesive samples selected at random from the large number of consolidation test results of soils covering all the districts of Bangladesh and obtained power, linear, logarithmic and exponential regression curves and all of which gave coeff. of correlation (r) from 0.76 to 0.79 and the linear relationship is as below :

$$C_c = 0.0102\ (W_N - 9.15), \quad \text{..........} (4.26)$$

with coeff. of correlation = 0.79

It is suggested that in absence of laboratory consolidation tests data, the above simple linear relationships can be used knowing only the natural moisture content of the saturated cohesive soils to estimate compression index, C_c and then settlements can be estimated using C_c and other parameters of the equation (4.24).

4.9.2 Cohesionless Soils

Settlements of cohesionless soils such as sands, gravels and granular fill materials, take place almost immediately as the foundation loading is imposed on them, while settlements in the cohesive soil layers continue over long period. Because of the difficulty of sampling cohesionless soils there is no practical laboratory test procedure for determining their consolidation characteristics. Therefore, the researchers have developed empirical correlation between the allowable bearing pressure and SPT N, and SCPT values. The allowable bearing pressure, q_a is defined as the maximum allowable net loading intensity of the ground in any given case, taking into consideration the bearing capacity, the estimated amount and rate of settlement that will occur, and the ability of the structure to accommodate settlement; it is therefore, a function both of the site and of the structural conditions.

4.9.2.1 By SPT N-Values

Terzaghi and Peck (1948) gave charts for estimation of safe bearing capacity of footing foundation from SPT values and their charts were applicable for a settlement of 25mm in dry sands. Peck et al (1974) modified the charts of Terzaghi and Peck (1948). These modified charts of Peck et al (1974) are presented in Fig. 4.18. These modified charts are developed for depth/width ratios of D/B=0.25, 0.5 and 1.0. Each line in the chart corresponds to a particular SPT N-value and indicates the soil pressure corresponding to a settlement of 25mm (1 inch). These charts were developed based on the results of tests on footings of different sizes in sand. The SPT N-value to be used in the charts should be the one, which is corrected for overburden pressure and the SPT N-value must represent the average condition of the soil below the footing upto a depth equal to the width of the footing.

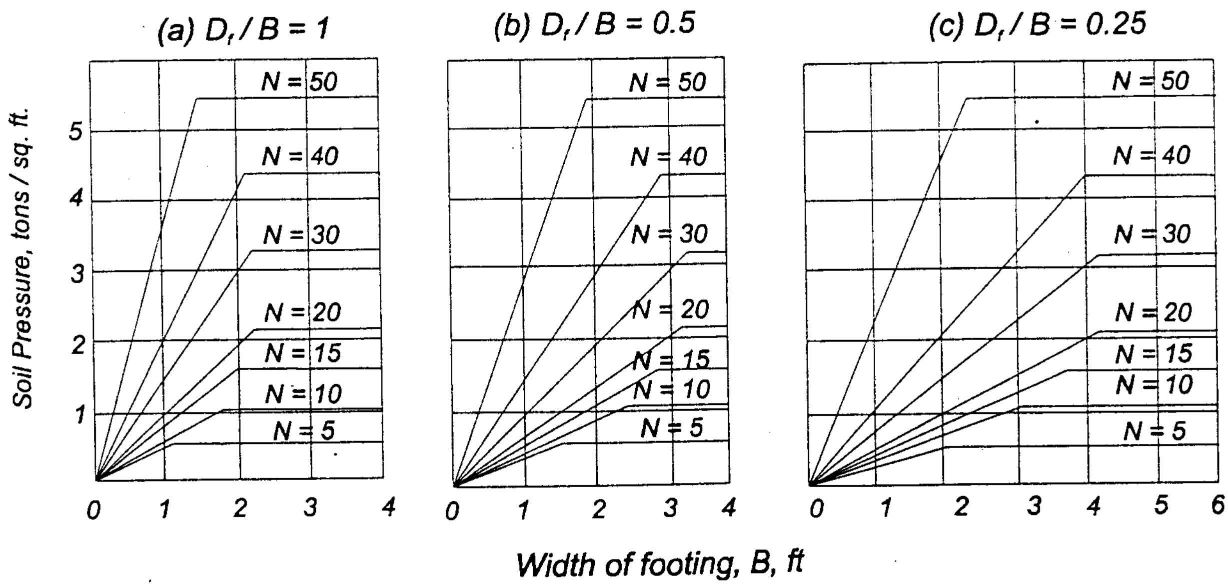

Fig. 4.18 Design Chart for Proportioning Shallow Footings on Sand (Peck et al, 1974).

The curves presented in Fig. 4.18 are for the condition that the water table exists at a depth greater than the width of the foundation. For other positions of the water table, necessary water table corrections are to be applied.

If the water table occurs and will remain at or below a depth D+B below the ground surface surrounding the footing, the footing should be proportioned for the soil pressure taken directly from the chart. If the water table is located at or may rise to the ground surface, the values from the chart should be multiplied by a correction factor $C_w = 0.5$. For a depth to ground water level equal to D_w, measured from the surface of the surcharge surrounding the footing, the correction factor may be obtained with sufficient accuracy by linear interpolation,

or

$$C_w = 0.5+0.5\,\frac{D_w}{D+B} \qquad \text{......} \qquad (4.27)$$

4.9.2.2 By SCPT Values

Meyerhof (1956) suggested a set of empirical equations based on Terzaghi and Peck curves (1948) for estimating safe bearing capacity for shallow foundations using SCPT values. The equations which considered both shear failure and settlement failure to calculate the allowable bearing pressure, q_a were mentioned earlier in equations (4.18), (4.19) and (4.20) under Article 4.8.2.2.

4.9.3 Safe Bearing Pressures for Raft Foundation on Sand

It is reasonable to permit larger safe soil pressures on raft foundation because the differential settlements of a raft foundation are less than those of a footing foundation designed for the same soil pressure. It has been suggested in literatures from experience that a pressure approximately twice as great as that allowed for individual footings may be used because it does not lead to detrimental differential settlements. For a soil pressure that produces a differential settlement of 19mm (3/4 inch), the maximum settlement of a raft may, however, be about 50mm (2 inches) instead of 25mm (1 inch) as for a footing foundation.

Peck et al (1974) recommended the following equation for computing net safe pressure (q_s) as

$$q_s = 2.1\ N\ \text{Tonne/m}^2, \qquad \text{......} \qquad (4.28)$$

or

$$q_s = 21\ N\ \text{kPa} \qquad \text{......} \qquad (4.29)$$

for $5 \le N \le 50$,

where N is the SPT value corrected for overburden pressure.

The equation (4.28 and 4.29) give q_s values above water table. Necessary correction factor should be used for the presence of water table as described earlier in Article 4.9.2.1. The value of SPT N to be considered in the above equation should be the average of the values obtained upto a depth equal to the least width of the raft. If the average value of N after correction for the overburden pressure is less than about 5, the sand is generallly considered to be too loose for the successful use of the raft foundation. Either the sand should be compacted or else the foundation should be established on piles or piers.

The recommended minimum depth of raft foundation is about 2.5m below the surrounding ground surface, because experience of investigators/researchers has shown that if the surcharge is less than this amount, the edges of the raft settle appreciably more than the interior because of lack of confinement of the sand.

4.10 CALCULATION OF CARRYING CAPACITY OF PILES

There are several main types of pile in general use and we shall discuss briefly only the method of estimating the carrying capacity of 'driven pile' for the culverts and small bridges covered in this manual.

4.10.1 Cohesionless Soils

4.10.1.1 Method Based on Standard Penetration Test

The ultimate carrying capacity, Q_u of a pile is equal to the sum of the ultimate resistance of the base of the pile and the ultimate skin friction over the embedded shaft length of the pile. This is expressed by the equation

$$Q_u = Q_b + Q_s \quad \text{..........} \quad (4.30)$$

where Q_b = base resistance, and

Q_s = shaft resistance

Net unit base resistance, q_{nu} is expressed as

$$q_{nu} = P_d\,(N_q - 1), \quad \text{..........} \quad (4.31)$$

and net total base resistance, Q_b is expressed as

$$Q_b = A_b P_d\,(N_q - 1) \quad \text{..........} \quad (4.32)$$

where P_d = effective overburden pressure at pile base level, and

N_q = Berezantsev's bearing capacity factor (Fig. 4.19), and

A_b = total area of the pile base

The ultimate unit skin friction, f_s on the pile shaft is given by the expression

$$f_s = \overline{K}_s P_d \tan\delta, \quad (4.33)$$

where,

$\overline{K}_s$ = an earth pressure coefficient,

δ = angle of wall friction, and

P_d = effective overburden pressure on element ΔL for the soil stratum of interest.

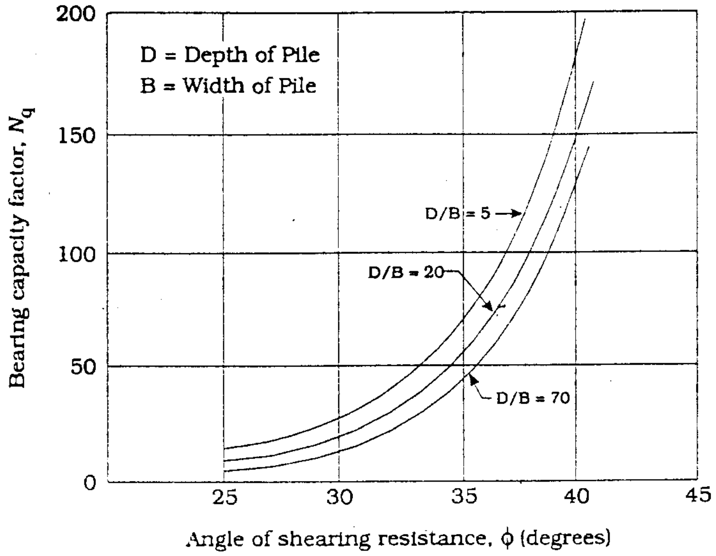

Fig. 4.19 Berezantsev's Bearing Capacity Factor, N_q.

The values of $\overline{K}_s$ and δ which are related to the effective angle of shearing resistance of cohesionless soils for various pile materials and relative densities are shown in Table 4.13.

Table 4.13 Values of $\overline{K}_s$ and δ (Tomlinson, 1986)

Pile material	δ	Value of $\overline{K}_s$	
		Low relative density	High relative density
Steel	20 ϕ	0.5	1.0
Concrete	$\frac{3}{4}\phi$	1.0	2.0
Wood	$\frac{2}{3}\phi$	1.5	4.0

The ϕ value can be used as obtained from standard penetration tests (Fig. 4.15). It is suggested at present to use a peak value of 110 kN/m^2 for straight sided piles, when the computed value of f_s with increasing length of the pile exceeds 110 kN/m^2. In many cases the skin friction on a pile in cohesionless soil is only a small proportion of its total resistance to compression loading and where piles are driven deeper than 20 diameters it may be satisfactory to use the values in Table 4.14 for the average skin friction over the whole shaft.

Table 4.14 Average Skin Friction Values for Straight-Sided Piles in the Cohesionless Soils (Tomlinson, 1986)

Relative density	Average unit skin friction (kN/m^2)
Less than 0.35 (loose)	10
0.35 - 0.65 (medium dense)	10 - 25
0.65 - 0.85 (dense)	25 - 70
More than 0.85 (very dense)	70 to not more than 110

The failure load of the pile, Q_u, is equal to the load at failure applied to the pile, Q'_u plus weight of the pile, W_p. Therefore, it has been shown that,

$$Q'_u = Q_u - W_b = Q_b + Q_s$$
$$= A_b\, P_d\, (N_q - 1) + Q_s \qquad \text{(from equation 4.32)}$$
$$= A_b\, P_d\, N_q - A_b\, P_d + Q_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.34)$$

Since the weight of the concrete in the pile is not much greater than the weight of soil displaced by the pile, then for all practical purposes the weight of the pile, W_p and A_b P_d are roughly equal for straight-sided or moderately tapered piles (Tomlinson, 1986).

$$\text{Therefore, } Q'_u = A_b\, P_d\, N_q + Q_s$$
$$= A_b\, P_d\, N_q + \overline{K}_s\, \overline{P}_d \tan\delta\, A_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.35)$$

where,

A_b = Area of the pile base

A_s = embedded surface area of pile

$\overline{P}_d$ = average effective overburden pressure over embedded depth of pile

4.10.1.2 Methods Based on Static Cone Penetration Test

Several methods of estimating pile capacity from SCPT are suggested in the literatures. The following two methods are, however, described and recommended for the purpose of this Manual.

4.10.1.2.1 Method 1

In extensive areas of Holland and Belgium, heavy structures are founded on piles driven through peats and soft clays to a suitable depth bearing on medium-dense to dense sands. In these countries an immense amount of experience has been gained in the interpretation of the static (Dutch) cone penetration test in relation to piled foundations in sands. The ultimate end-bearing resistance is taken, as being equal to the resistance of the cone. To allow for the variation in cone resistance which normally occurs it is usual to use Van der Veen's method, in which an average is taken of the cone resistance over a depth equal to three times the diameter of the pile above pile point level and one pile diameter below pile point level (Tomlinson, 1986).

Average cone resistance should be taken over this depth after Van der Veen (1957). It has been suggested that if a factor of safety of 2.5 is applied to the ultimate end resistance as determined from static cone resistance, the pile is unlikely to settle by more than 15 mm under the working load.

The skin friction on the pile shaft is also obtained from the cone resistance using the simple empirical relationships established by Meyerhof (1956). For driven (displacement) concrete piles, the ultimate unit skin friction, f_s is given by

$$f\Delta = \frac{\bar{q}_c}{2} kN/m^2 \approx \frac{\bar{q}_c}{20} t/m^2 \quad \text{.......} (4.36)$$

where $\bar{q}_c$ = average cone resistance (kg/cm^2) over the length of pile shaft under consideration.

Meyerhof suggests that for straight-sided displacement piles in cohesionless soils the ultimate unit skin friction has a maximum value of 107 kN/m^2 (calculated on all surfaces of flanges and web)

The following procedures are suggested for using the static cone penetration test results :

a) The cone resistance-depth diagram should be inspected and a provisional depth should be selected for pile tip level which will, as far as possible, utilize the maximum permissible stress on the material of the pile shaft,

b) The average cone resistance over the range of depth three times the pile diameter above the pile tip and one pile diameter below the pile points level should be taken,

c) The end resistance of the pile from the average cone resistance should be obtained from (b),

d) The skin friction should be calculated from the average cone resistance along the pile shaft using equation (4.36) for driven (displacement) pile, and

e) The total ultimate pile resistance is obtained from the sum of end resistance and skin friction and then the working load is obtained by dividing the sum by a factor of safety of 2.5.

If the working load thus obtained is less than that required for the structural design engineer's loading conditions, then the pile must be taken to a greater depth to increase the skin friction value or to reach a value where the end resistance would be higher. However, the researchers have shown that the pile should be driven at least five diameters into the bearing stratum to mobilize the peak value.

4.10.1.2.2 Method 2

The pile capacity can be estimated from q_c and f_s using the following formula :

$$Q_{allowable} = \frac{1}{2}\left(q_c A_p + \sum_0^L f_s x \Delta A_s \right) \quad (4.37)$$

q_c	=	the average cone resistance at the pile tip
f_s	=	the average sleeve friction for ΔA_S
ΔA_s	=	pile perimeter x ΔL = incremental pile surface area
F	=	safety factor
A_p	=	area of pile tip

It is not recommended that the cone penetrometer be used without some supplementary information from conventional soil borings. Data from conventional borings, that is, standard penetration blow count, unconfined shear strength from tube samplers, visual identification of soils, and water table data, are vital to accurate interpretation of cone penetrometer data.

For further information about the use of cone resistance in foundation design of a structure, Tomlinson (1986), Bowles (1988) and Dunn et al (1980) or Murthy (1993) may be consulted.

4.10.2 Cohesive Soils

In a purely cohesive soil (ϕ =0) there is, in simple terms, no skin friction. However, the term, skin friction, is widely used to denote adhesion or cohesion on the shaft of a pile in a cohesive soil. Therefore, the carrying capacity of piles driven into clays and clayey silts is taken as equal to the sum of the end bearing resistance and the skin friction of that part of the shaft in contact with the soil. The end resistance is given by the equation

$$Q_b = N_c \text{ x } C_b \text{ x } A_b \quad (4.38)$$

The bearing capacity factor N_c can be taken as being equal to 9 provided that the pile is driven at least five diameters into the bearing stratum. The shear strength C_b at the base of the pile is taken as the undisturbed shear strength provided that time is given for a regain from remoulded to the undisturbed shear strength conditions.

The skin friction on the pile shaft is given by the equation

$$Q_s = \propto \times \bar{C}_d \times A_s, \qquad (4.39)$$

where,

$\propto$ = adhesion factor (see Fig. 4.20)

$\bar{C}_d$ = average undisturbed shear strength of clay and cohesive silts adjacent to the shaft,

A_s = surface area of the shaft

In the case of uniform clays and cohesive silts or clays and cohesive silts increasing progressively in shear strength with depth the average value of shear strength over the whole shaft length can be taken for $\bar{C}_d$. Where the clay/cohesive silt exists in layers of appreciably differing consistency, e.g. soft clay/silt over stiff clay/silt, the skin friction should be separately calculated for each layer using the adhesion factor appropriate to the shear strength and overburden conditions.

The working load of the pile is equal to the sum of the base resistance and shaft friction divided by a suitable factor taking into account the range of adhesion factors (Fig. 4.20). A safety factor of 2.5 is reasonable on this sum as under :

$$\text{Allowable pile load } Q_a = \frac{Q_b + Q_s'}{2.5}, \qquad (4.40)$$

where Q'_s is the skin factor calculated from the adhesion factors shown in Fig. 4.20, using the average shear strength. Also Q_a should be not more than

$$\frac{Q_b}{3} + \frac{Q_s''}{1.5}, \qquad (4.41)$$

where Q''_s is the skin friction calculated from the adhesion factors in Fig. 4.20 using the lowest range of shear strength (Tomlinson, 1986).

4.10.3 Capacity of Pile Groups

Individual structural loads are generally supported by several piles acting as a group. The structural load is applied to a pile cap that distributes the load to the piles. If the piles are spaced a sufficient distance apart, minimum 3 times the diameter or width of the pile, then the capacity of the group is the sum of the individual capacities. It is assumed that in the construction of the culverts and small bridges covered in this manual, the piles where necessary shall be spaced at least 3 times the diameter/width of the driven piles.

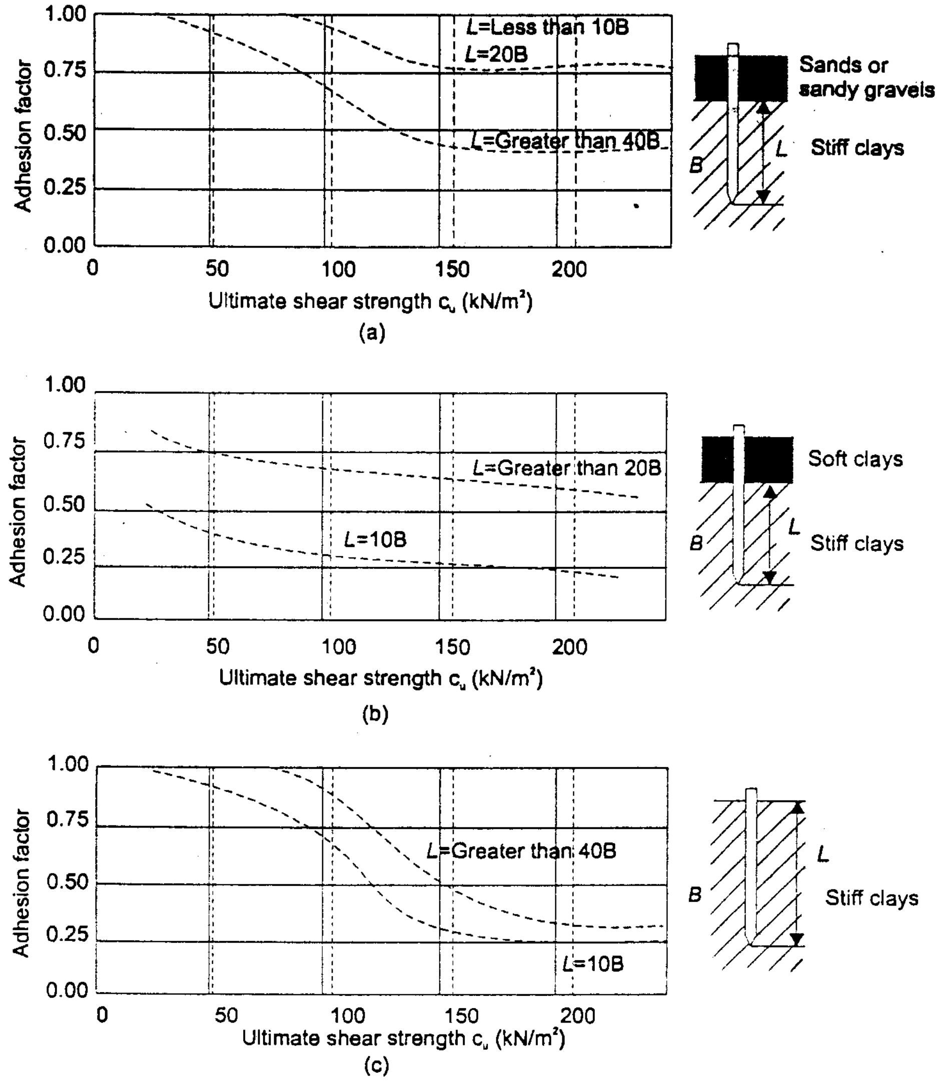

Fig 4.20 Adhesion Factors for Driven Piles in Clay. (a) Case 1: Piles Driven through Overlying Sands or Sandy Gravels. (b) Case 2: Piles Driven through Overlying Weak Clay. (c) Case 3: Piles without Different Overlying Strata (Tomlinson, 1986).

4.10.4 Settlement of Pile Groups

The settlement of pile groups should also be considered. The same general methods presented for shallow foundations in Article 4.9 are used to predict the settlement of deep foundations. Settlement in clays soils is generally more troublesome than in sandy soils (Bowles, 1988, Dunn et al, 1980 and Tomlinson, 1986).

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Hydraulic Consideration

This chapter presents the hydrologic situation prevailing in Bangladesh and attempts to put forward the methods, procedures and guidelines on how to determine various hydrologic and hydraulic parameters of a structure at the proposed site.

The important hydrologic items to address are: site selection, determination of design flood level and discharge, design flow, fixation of design road level and navigational clearance. Based on the hydrologic parameters, the hydraulic parameters such as waterway opening, scour depth, depth of foundation for piers and abutments are to be computed. The methods and procedures have been discussed in this chapter under two broad heads - defined channels and undefined channels.

Determination of design velocity and flow of a defined channel where no data is available is a difficult task. Procedure for direct measurement and use of empirical formulas and other indirect methods of determination may be adopted.

Very often, it is not possible to get hydrologic data of the desired location. Data may be available for places which are hydraulically connected to the desired site. In such cases hydrologic parameters may be determined by means of co-relation and interpolation. Where time and situation permit high flood level and discharge may be determined by direct measurement; otherwise empirical formula may be used to compute velocity and discharge of the site.

For smaller catchments, discharge of a site can be related to rainfall intensity. Nomograph may be used to compute discharge for such cases. Stream velocity curve may be used to compute velocity for small streams whose hydraulic radius (R), roughness coefficient (n) and channel (S) slope are known.

If any bridge or structure exists on the channel, hydraulic formula such as Orifice formula may be used to compute discharge (Q) through the bridge/structure.

If there is no defined channel, the situation has to be handled in a different way. The solutions for this type of situation is to build either a causeway or a culvert. Care should be given to find the velocity and area for the culvert opening. When time permits, gaps in the road embankments may be kept and observed for one season. After observing the effectiveness of the gap, its size may be finally fixed.

CHAPTER 5
Hydraulic Consideration

5.1 INTRODUCTION

Selection of a proper size of waterway opening of a road structure depends largely on the hydraulic flow conditions and the stability of soil at the stream crossing site. While the head room that is to be provided below the bridge/opening will depend on the highest likely water level and the desired navigational facilities, the width of the opening will depend on the amount of water that may flow through it without detrimental scour. An inadequate size of opening allows scour of river bed/bank and soil adjacent to the structure. On the other hand, an unnecessarily large opening increases the cost of construction. In this chapter, steps and methods for selecting proper openings for road structures and the corresponding depth of foundation are described. The problem is complex specially due to the unique deltaic geographical and hydrological characteristics of Bangladesh. In this context the hydrological condition of Bangladesh is very relevant which is described in brief in this chapter below.

5.2 HYDROLOGICAL CONSIDERATIONS FOR BANGLADESH

During the monsoon, most of Bangladesh is flooded. Within this flooded area discharge through smaller streams is controlled by the water level at the downstream. Construction of any structure across the stream may create constriction to the flow, resulting in rise of water level and considerable flooding in the upstream area. However, the downstream water level and total discharge may remain the same and therefore, the stream velocity may increase significantly. If the structure is properly designed, it should be able to evacuate the upstream area of stored water at such a rate that there is no danger of abnormal rise in upstream water level. An increase in the difference between upstream and downstream water levels may produce high velocity, endangering the stability of the structure by scouring or outflanking. The above, therefore, requires water balance analysis equating discharges through a structure to upstream storage levels and the rate of fall of the control level. This, in reality, is complicated by the presence of undersized structures downstream, tidal fluctuations, change in upstream water storage volume with rise in water level, the effect of adjacent river control structures and seasonal silting or scouring at takeoff points. The basic approach should therefore be, where economy permits, the structure built should in no way constrict the maximum flow of the stream. In order to appreciate this point, an understanding of the hydraulics of flow through a bridge is necessary.

In contrast to the above, there are areas within Bangladesh that are sufficiently elevated and out of influence of the flood submergence of the main river systems. In these areas the runoff can be related. to rainfall intensity and surface drainage characteristics of the land using some empirical relationships. However, it should be recognised that Bangladesh lies in the southwest monsoon belt. The monsoon period generally extends from June till mid-October. During this period the rainfall comes in intermittent showers, which may be intense and may occur several times during a day while there are intervening days of clear weather. Downpours of heavy rain are generally localized; deluge may occur over a small area, while a short distance away the fall may be nominal.

5.3 CHANNEL TYPE AND WATERWAY OPENING

The water paths under a road structure are usually classified as either a well defined channel or an ill defined channel. A well defined channel is one where the flow is contained within raised banks for most of the year. On the other hand, where no definite stream exists and the flow at the location is caused by a natural depression in the terrain, the water path may be termed an ill defined channel. An ill defined channel is often characterized by inundation of a large area and a sheet like flow of water.

The first step in the design of a road structure in either of the above channel conditions is the determination of the required width of waterway under the structure and the required depth of scour. The required width of waterway is decided by the flow through the channel and the depth of foundation by the anticipated depth of scour. Both these parameters are distinct for the two types of channel, well defined and ill defined. A systematic approach for their determination is given in flow chart form in Chapters 13 and 14 of Part A of Road Structure Manual.

The flow chart makes frequent reference to the articles that follow. These articles cover the hydrological and hydraulic aspects of well defined and ill defined channels and present methods of determing width of waterway and depth of foundation.

5.4 SELECTION OF SITE

Usually road structures such as culverts and bridges will be built across breaks along roads. Such breaks may be due to khals and streams or bare openings left for water to pass from one side of the road to the other. Where there is any choice, a site should be selected which:

i) is situated on a straight reach of the stream, sufficiently away from bends

ii) is so far away from the confluence of large tributaries as to be beyond their disturbing influence

iii) has well defined banks

iv) makes approach roads straight

v) offers a square crossing as far as practicable

5.5 MEASUREMENT OF STREAM CROSS SECTION AND DRAINAGE AREA

For a well defined stream; five cross sections should be taken, namely one at the selected site, two at 25m and 50m upstream and the other two at 25m and 50m downstream of the site. The cross sections should show levels at close intervals and outcrops of rocks, pools etc. Often an existing road or cart-track crosses the stream at the site selected for the bridge. In such a case, the cross section should not be taken along the centre line of the road or track as that will not represent the natural shape and size of the channel. Instead the cross section should be taken a short distance upstream or downstream of the selected site.

The cross sections should be plotted to a suitable scale if possible both horizontal and vertical being the same. The same scale plotting is preferred but may not be possible always. If the scale are different care should be taken to calculate various parameters based on the plotting. A rough longitudinal section may also be plotted from the cross sectional measurements at the five sections.

When no defined channel exists at the site, the general nature of ground slope within some 500m around the selected crossing site should be established. A map to a scale of 4 inch to 1 mile, approximately 1 in 16,000, showing the topographical features of the terrain including land elevation around the site can be obtained from the Survey of Bangladesh. From the general slope of the terrain, and from interviewing local people, the general direction of flow, the drainage areas feeding the crossing site and water levels during high floods should be established.

5.6 FLOOD LEVEL

Generally, floods occur when the river spills over the bank and submerges the agricultural land and adjacent homesteads. This type of flooding may occur once every five years. In other areas, inundation may result due to heavy rainfall. In this case, localized flooding may occur. This localized flooding is an annual phenomenon and its intensity depends on the amount of rainfall.

5.6.1 Procedure for Determining High Flood Level

5.6.1.1 HFL Determination

The High Flood Level (HFL) is determined on the basis of determining the maximum inundation at different locations along the alignment. This is done by assessing the land topography, land use patterns, catchment characteristics, etc.

To determine the HFL consideration should be given to the normal inundation level (i.e. the Annual Average High Flood Level) and not the abnormal flood levels experienced in 1988/1995 or any abnormal flood. Inundation can happen due to localized drainage congestion, free flow of water and tidal fluctuation (high tide in full moon) in the coastal belt regions.

5.6.1.2 Location (intervals) for HFL Information

The HFL should be determined at all locations along an alignment where there is a change in one or all of the following features:

- change in topography
- change in land use pattern
- change in catchment characteristics

If a catchment is large (i.e. 0.50 sq. km. or more) special attention must be given to determine the HFL when the terrain is gradually sloping upward or downward. In all cases, the longitudinal profile (or part) must never go below the HFL line.

5.6.1.3 Techniques to determine the High Flood Level

Interviewing Local People

Interviews with the local people is one of the most effective ways to determine the HFL. Discussion on the HFL would have to be based on the annual flood levels for the last 10 years, excluding the exceptional floods (i.e. 1988, etc.) At least 5 people of different groups should be interviewed to confirm the validity of information. At the same time personal judgement is also important which can be applied to come up with a reasonable estimate of the information.

Identifying Water Marks on Trees/Structures

When an area is flooded, water marks can be identified on the surface of tree trunks or on the walls and foundations of structures (e.g., buildings, bridges, culverts, etc.). If such a mark exists in the vicinity of a road alignment, it can give a more accurate reading on the HFL.

Physical Observations (Physical Features)

Observation of a physical features is also important in determining the HFL. The plinth level of a house is generally constructed above the high flood level. Existing bridges and culverts (and in particular the free board) on an alignment are also other indicators for determining the HFL.

Land Use Pattern Observed

Pattern of land use is also a factor in determining the HFL. For example. paddy/jute fields are generally flooded but fruit and vegetable gardens are normally above the flood level. When a road alignment, or any part of it, passes through a homestead area or a high land, it can be assumed that this portion is above the HFL. Only those portions which are damaged might be flooded.

Bangladesh Water Development Board (BWDB)

The Bangladesh Water Development Board (BWDB) generally implements water resource related projects on the basis of statistical analysis of available water level data. Markings are made of the highest flood levels on permanent pillars indicating the maximum level on terms to PWD datum. If these marking are located, these can also be an effective tool in determining the HFL.

Protected Areas in Consideration

Within the protected areas (FCD/I and CFD projects), flood levels are significantly lower than before these projects were there. This should be considered while measuring HFL for an alignment in a protected area.

Difference on Either Side of the Road

In the case of an alignment having a housing area or high land on one side and a low lying area on the other side, the higher level of inundation should be considered as HFL.

Comparison with Reference Year

The people should be asked what the normal flood level as at several locations along the alignment. In addition they should be asked what the flood level was in an abnormal flood year (e.g. 1988). This information should be compared to ensure a clearer picture of the flooding situation along the alignment.

5.7 FLOW VELOCITY

5.7.1 Direct Measurement

The velocity of flood or the surface velocity can be measured by observing the time taken by a floating object, preferably an wooden block, to move a known distance.

If the stream is of fairly uniform depth the mean velocity of the whole stream may vary from 80 to 90 percent of the surface velocity in ordinary cases, that is, cases in which the depth varies from 10 to 25 percent of the breadth. If the depth of the stream is less than 10 percent of the breadth the mean velocity of the entire stream may be as low as 70 percent of the surface velocity.

Details of the float, distance at which the floats should be released above the upstream cross section and the time of floats passing from upstream to downstream cross section and the procedure as a whole are defined in Article 5.8.4 — the discharge measurement by Float Method.

Typical flow velocity pattern at different points across the depth of the channel and surface velocity pattern across the width of the channels are shown in Fig. 5.1. These curves have been developed such that on simple sections the mean velocity is approximately equal to the surface velocity at the centre multiplied by coefficient 0.85.

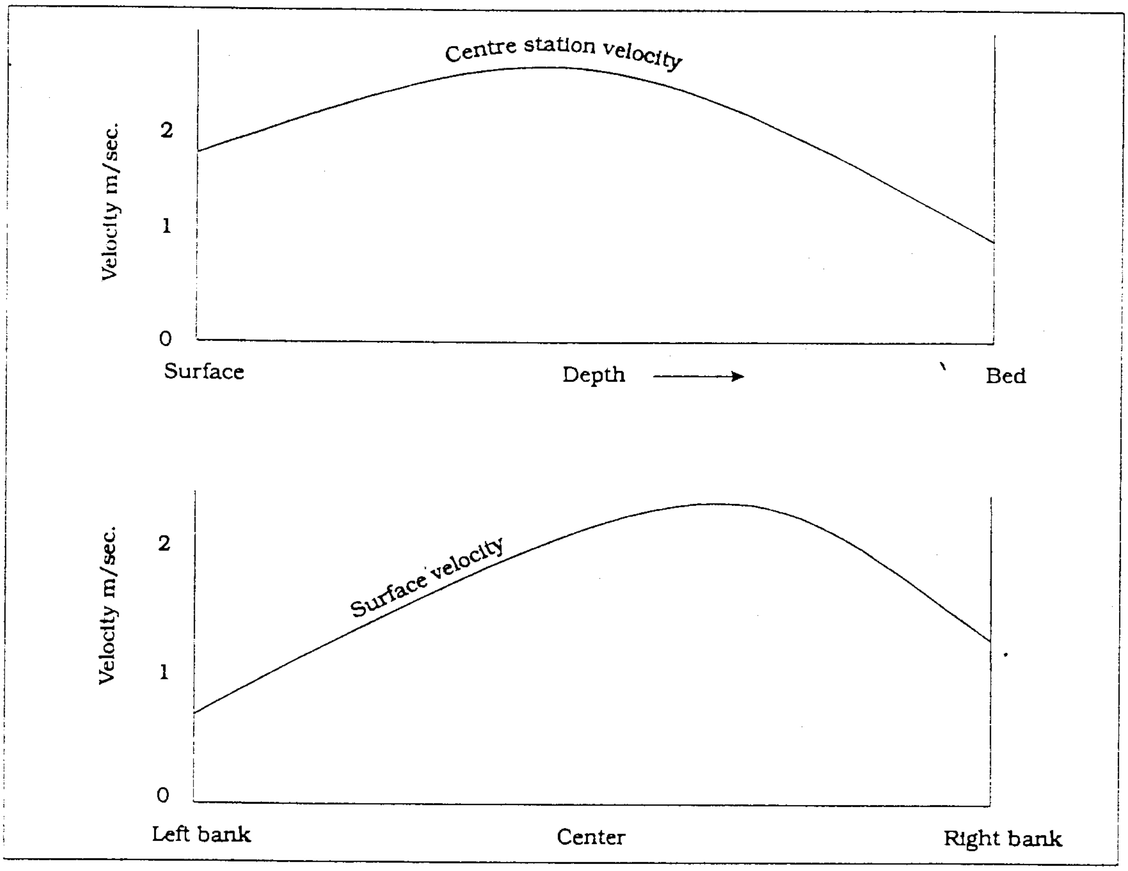

Fig. 5.1 Typical Flow Velocity Patterns

Fig. 5.1 shows a typical velocity distribution pattern. Actually, the velocity distribution pattern is dependent on the shape of the channel section, depth and position of the thalweg (deep channel).

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5.7.2 Calculation Using Bed Characteristics

The alternative to direct measurement is to use Manning's formula to estimate the mean velocity:

$$V = \frac{1}{n} R^{\frac{2}{3}} . S^{\frac{1}{2}} \qquad \text{...(5.1)}$$

where V = velocity (m/sec)

R = hydraulic radius which is calculated from the cross-sectional area divided by the wetted perimeter.*

S = gradient of the surface or bed slope

n = value of roughness coefficient taken from Table 5.1.

* The method has been shown in Fig. 5.2.

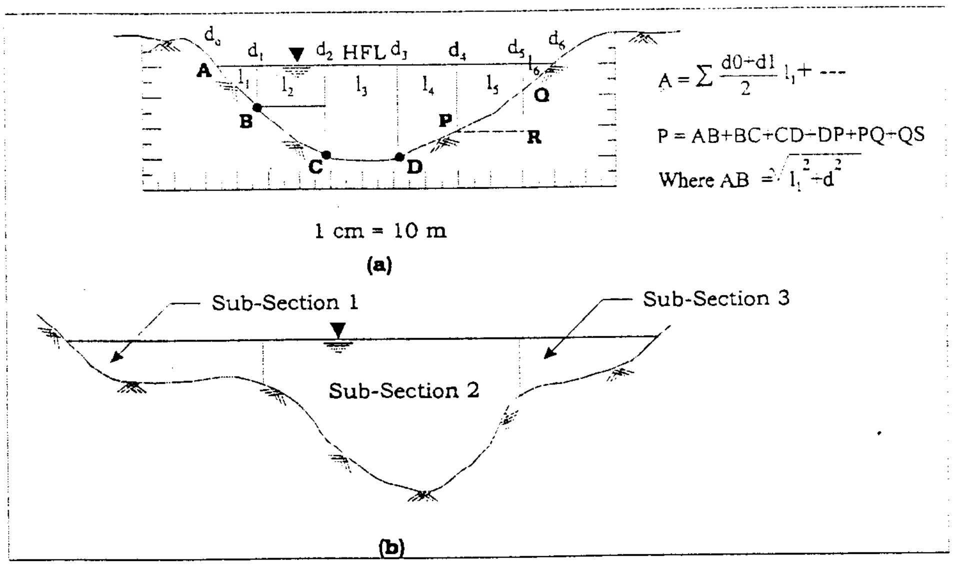


Fig. 5.2 Measurement of Wetted Perimeter from Stream Cross Section

Table 5.1: Values of Roughness Coefficient n for Natural Streams and Flood Plains

Type of channel and description		Minimum	Normal	Maximum
A.	Natural streams			
1.	Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2.	Same as above, but more stones and weeds	0.030	0.035	0.040
3.	Clean, winding, some pools and shoals	0.033	0.040	0.045
4.	Same as above, but some weeds and stones	0.035	0.045	0.050
5.	Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6.	Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
7.	Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
B.	Mountain streams, no vegetation in channel, banks usually steep, or floodways with heavy stand of timber and underbrush			
1.	Bottom : gravels, cobbles, and few boulders	0.030	0.040	0.050
2.	Bottom : cobbles with large boulders	0.040	0.050	0.070
C.	Flood plains			
1.	Pasture, no brush			
	Short grass	0.025	0.030	0.035
	High grass	0.030	0.035	0.050
2.	Cultivated areas			
	No crop	0.020	0.030	0.040
	Mature row crops	0.025	0.035	0.045
	Mature field crops	0.030	0.040	0.050
3.	Brush			
	Scattered brush, heavy weeds	0.035	0.050	0.070
	Light brush and trees, in winter	0.035	0.050	0.080
	Light brush and trees, in summer	0.040	0.060	0.080
	Medium to dense brush, in winter	0.045	0.070	0.110
	Medium to dense brush, in summer	0.070	0.100	0.160

5.8 DISCHARGE

Four methods for estimating the discharge of a river are outlined below:

5.8.1 Area Velocity Method

The most common type of indirect measurement of flood discharge is made through a reach of river channel by computing on the basis of uniform flow equation, such as the Manning's equation, involving channel characteristics, water surface slope and roughness coefficient.

For a well defined stream; five cross sections should be taken and plotted as described in Art. 5.5.

In a stream with rigid boundaries (bed and banks) the shape and size of the cross section such as show in Fig. 5.3 is significantly the same during a flood as after its subsidence. After plotting carefully the HFL and measuring the bed slope it is simple to calculate the discharge. The following guideline is given for determination of slope of channel.

$$S = hw/L$$

Where S = Channel slope

hw = difference in water level of two successive sections.

L = distance apart of sections.

If hw and L are measured, S may be computed.

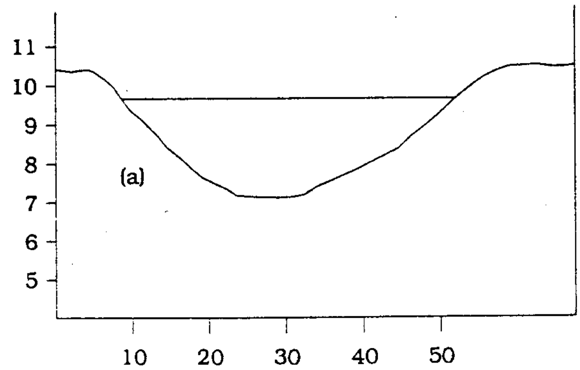
a) The channels of the Northwest zone of banglsdesh has high slope. this cen be determined from the water level data of BWDB gauges close to the site and the data may be adopted.

b) The lower part of Northwest Bangladesh, the haor areas of Sylhet and tidal influence areas of middle part of Bangladesh have also low slope. This also cen be determined from the water level data of BWDB gauges close to the site in question.

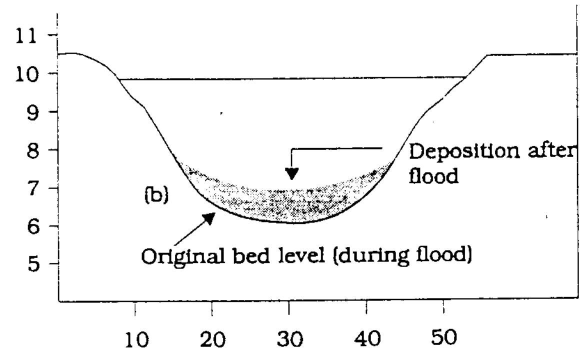
c) The hilly areas of the Southeast has high slope. This may be determined from the BWDB water levels close to the site.

d) Slope of channles may be estimated from the large scale contour map using contour interval and distance apart of the sections.

e) BWDB Water Supply Paper No. 338 (May 1972) contains information of specific rivers on channel slope and roughness values.



(a) Non erodible channel bed



(b) Erodible channel bed

Fig. 5.3 Showing Erodible and Non-Erodible Channel Sections

A stream flowing in alluvium will have a larger cross sectional area when in flood than that surveyed and plotted after the flood has subsided. During the flood the velocity is high and therefore an alluvial stream scours its bed; but when the flood subsides, the velocity diminishes and the bed progressively silts up again. A typical channel section of this nature is shown in Fig. 5.3b. From this, it follows that, before starting estimating the flood conveying capacity of the stream from the plotted cross section, ascertain the depth of scour and plot on the cross section the average scour bed line that is likely to prevail during high flood.

The best thing to do is to inspect the scour holes in the vicinity of the site, look at the size and degree of incoherence of the grains of the bed materials; whether clay silt or coarse sand; study the trial bore section, and then judge what should be taken as the probable average scoured bed line. Depending on possibility of further scour or deposition during flow, discharge calculated without the above consideration if any may be adjusted by factors shown in Table 5.2.

Table 5.2: Velocity Correction Factors for Silting and Scouring Conditions

Silt Condition at Site	Percentage of Velocity to be added
High degree of scouring	+ 25
Moderate degree of scouring	+ 10
Regime flow	0
Moderate siltation	- 20
High siltation	- 30

For calculation of velocity, plot the probable scoured bed line and measure the cross sectional area A in square metre and the wetted perimeter P in metre. Then calculate the hydraulic mean depth R = A/P. Next measure the bed slope S from plotted longitudinal section of the stream or water surface slope along a known distance of the stream. The process of calculation of area (A) and wetted peremeter are shown in Fig. 5.2. Velocity can then be calculated from Manning's formula :

$$V = \frac{1}{n} R^{\frac{2}{3}} . S^{\frac{1}{2}} \quad \text{.........} (5.2)$$

where V = velocity in m/s, considering uniform throughout the cross section.

R = hydraulic mean depth, m

S = energy slope which may be taken equal to stream bed slope* measured over a reasonably long reach or water surface slope over a known distance of the stream. (S is expressed in m/m)

n = Manning's coefficient of roughness for the river bed, Table 5.1 provides values of n for different types of streams.

*It is difficult to find slope from the water levels of successive sections. The slope varies very widely in the same river from zone to zone. The user is advised to refer to BWDB gauges close to the site and adopt slope of the channel.

The discharge may be calculated by the equation

$$Q = A.V \quad \text{.........} (5.3)$$

$$= \left(\frac{A.R^{2/3}.S^{1/2}}{n} \right) = \lambda . S^{1/2} .$$

where:

$$\lambda = \left(\frac{A.R^{2/3}}{n} \right) \qquad (5.4)$$

Now λ is a function of size, shape and roughness of the stream and is called conveyance factor. Thus the discharge carrying capacity of a stream depends on its conveyance factor and slope.

When the cross section is not plotted to the natural scale (same horizontally and vertically), the wetted perimeter can not be scaled off directly from the section and has to be calculated. Divide up the wetted line into a convenient number of parts, AB, BC, CD etc. as shown in Fig. 5.2 (a). Consider one such part, say PQ. Let PR and QR be its horizontal and vertical projections. Then $PQ = \sqrt{PR^2 + QR^2}$. Now PR can be measured in the horizontal scale and QR on the vertical scale and PQ can then be calculated. Similarly each of the other parts is calculated. Their sum gives the wetted perimeter.

The PQ so computed may be slightly different from the actual PQ. In selecting segments of the section, care should be taken so that the diagonal of the triangle be as straight as possible.

If the shape of the cross section is irregular as happens when stream rises above its banks and shallow overflow is created, it is necessary to subdivide the channel into two or three sub-sections. Then R and n are found for each subsection as shown in Fig. 5.2 (b) and their velocities and discharge computed separately.

Curves shown in Fig. 5.4 may be used to compute velocity for a given R, S and n.

The flood discharge should be calculated at each of the five cross sections, which, as already explained earlier should be plotted for all structures and highest of these values should be adopted as the design discharge Q. If the difference in the five discharges thus calculated is more than 10 percent, the engineer has to form a judgement based on the reliability of data on which each calculation was based.

There may be situations where the flood flow spills over the banks. In such situations attempt should be made to compute channel flow as stated above and overbank spills should be estimated separately. In most cases there are rural roads or embankments by the side of the river banks. The limit of overbank spills spreading on either side of the section may be measured, depth and velocity may be estimated and flow determined. This estimated flow should be added to the channel flow to determine the total flow past the channel section.

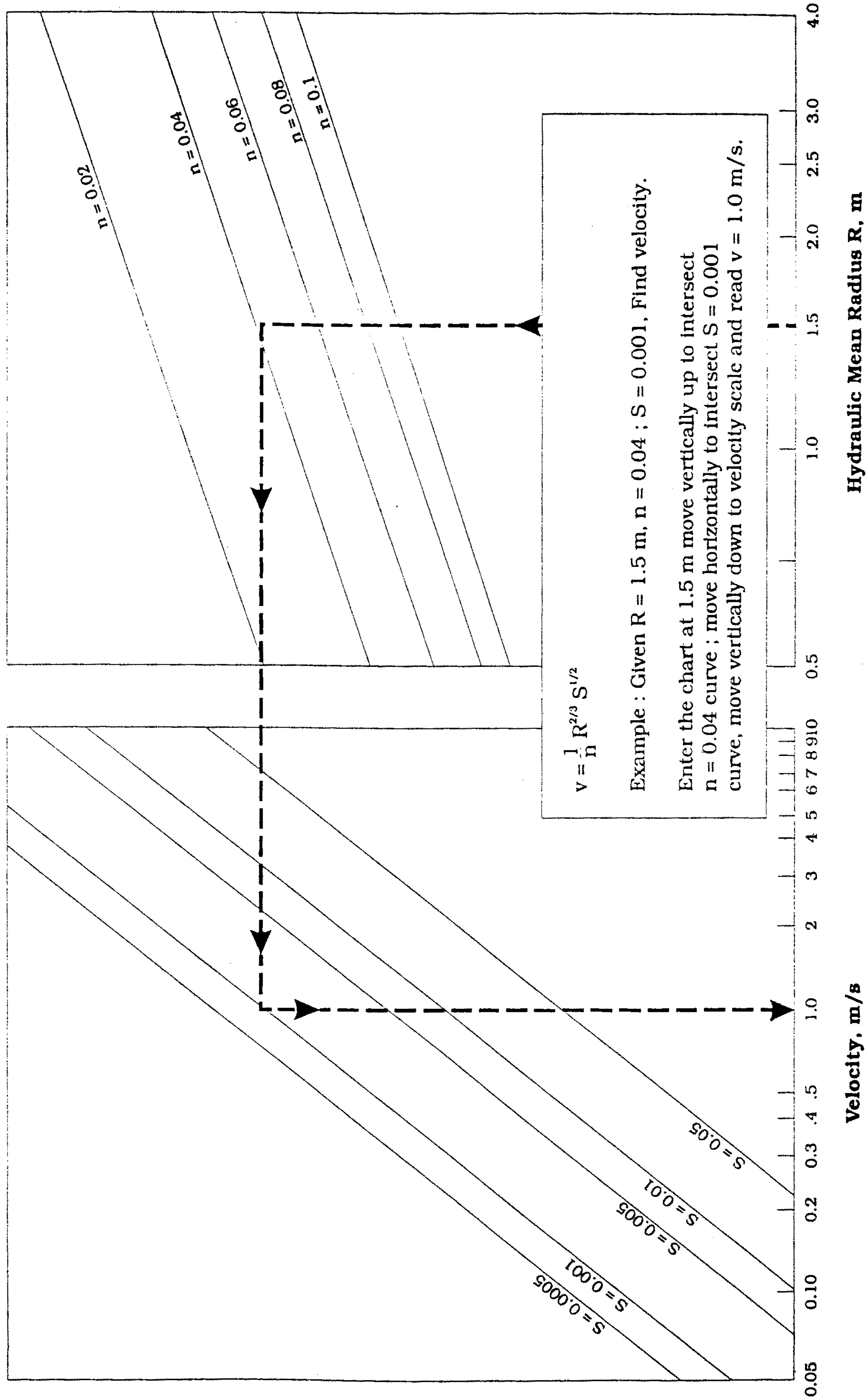


Fig. 5.4 Stream Velocty Curve

5.8.2 Catchment Runoff Method

Most of the areas in Bangladesh, stream flow is usually controlled by the flood level in the major rivers; that is by the downstream control. But at elevated lands such as high areas in the eastern and northern part of Bangladesh, flash floods occur which are not affected by water level in major rivers or downstream control and therefore may be related to rainfall intensity and catchment area characteristics. However, flow from such floods on reaching the plain lands, is again controlled by the flood level in the adjacent major rivers.

A number of empirical formulae are available to estimate the discharge by multiplying the catchment area with a factor depending upon rainfall and drainage characteristics of the area. Since such factors are not known for all areas in Bangladesh, a method based on rainfall intensity and drainage characteristics is proposed. In this method the following factors are to be considered :

1. One hour rainfall, I_O
2. The catchment area that will be drained by the stream
3. A factor P depending on the characteristics of the drainage area.

The nomograph shown in Fig. 5.5 may be used for the purpose.

The first step in this method is to determine the severest rain storm (in the region) that occurs once in 25 or 50 years. Let the total precipitation of that storm be F cm and duration be T hours. The rainfall intensity at any period t can be obtained from the equation give below.

$$\frac{i}{I} = \frac{T+C}{t+c} \qquad \text{.....(5.5)}$$

where C is a constant and in most cases equal to 1.

i = intensity of rain in t hours, cm per hour

I = intensity of rain in T hours, F/T cm per hour

The intensity over 1 hour for the storm I_o will be

$$I_o = \frac{F}{T}\left(\frac{T+1}{1+1}\right) = \frac{F}{2}\left(1+\frac{1}{T}\right) \qquad \text{.....(5.6)}$$

It is convenient that the storm potential of a region for a given period of years should be characterised by specifying the "one hour rainfall" I_o of the region for the purpose of designing the waterways of the bridge in that region.

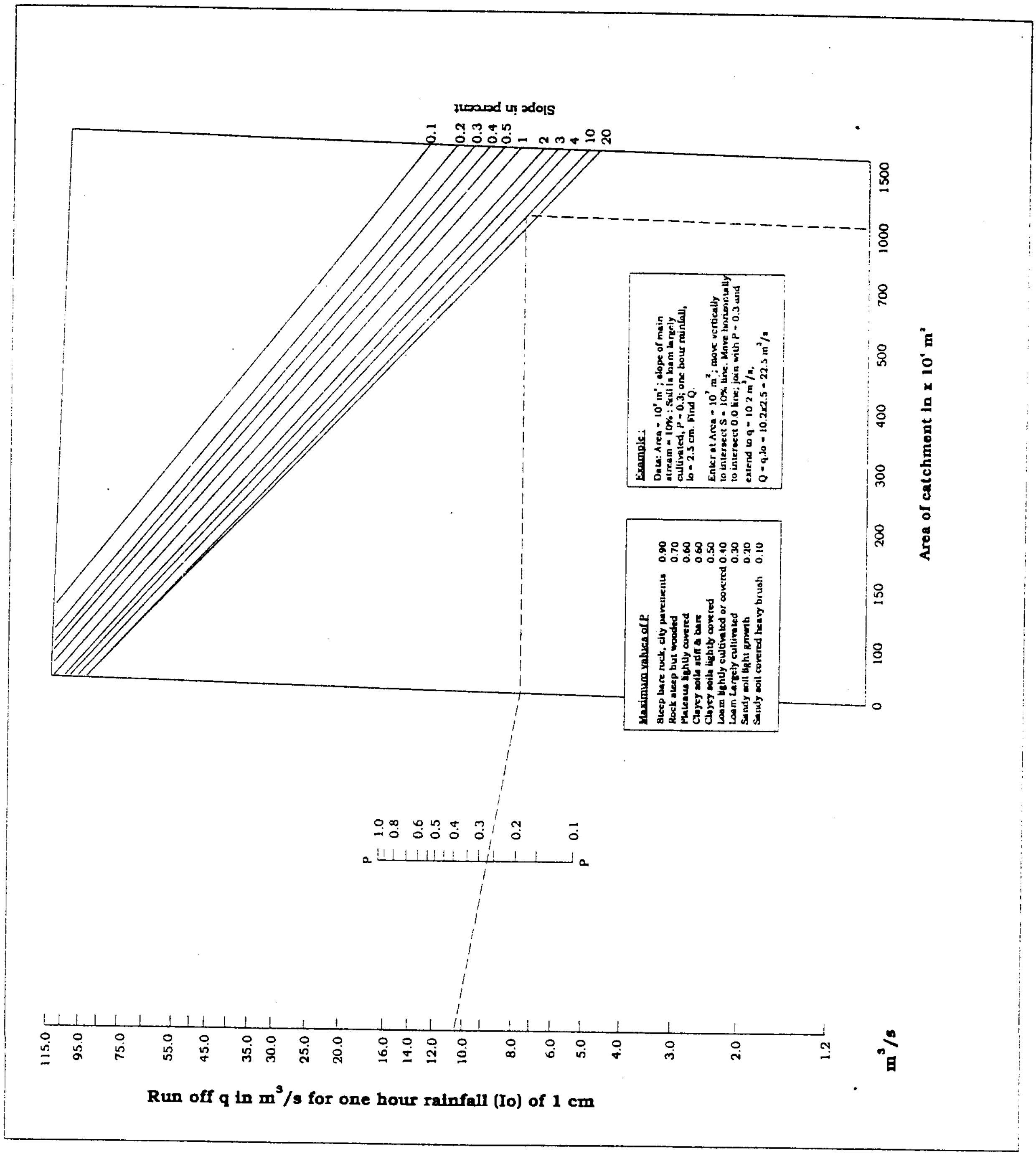


Fig. 5.5 Nomograph for Calculating Run-off for Small Catchments

I_0 has to be determined from F and T of the severest storm. That storm may not necessarily be the most prolonged storm. The procedure for finding I_0 would be to take a number of really heavy and prolonged storms and work out I_0 from F and T of each of them. The maximum of the value of I_0 thus formed should be accepted as "the one hour rainfall" of the region for designing bridges.

The factors on which the value of P depends include (i) porosity of the soil (ii) area, shape and size of the catchment (iii) vegetation cover, (iv) surface storage, i.e.

existence of lakes, marshes, (v) initial state of wetness of soil. Catchments vary so much with regard to these characteristics that it is evidently impossible to do more than to generalise on the values of P. Judgement and experience must be used in fixing P. Fig. 5.5 includes some suggestion on values of P to be used.

An example for estimating runoff is provided in Fig. 5.5. The steps to be considered in calculating runoff are as follows:

i) Delineate and determine area of catchment A in hectares from the large scale topographic maps preferably in 1:16,000 scale prepared by Survey of Bangladesh showing ground contours. Also determine ground slope from the map based on distance between the contour intervals and express in percentage.

ii) Estimate I_0 for the region preferably from rainfall records, failing that from $I_0 = \frac{F}{2}\left(1+\frac{1}{T}\right)$ where F is cm of rainfall by the severest storm in T hours.

iii) Read values of P from Fig. 5.5 for the area.

iv) Enter area and move vertically upto the slope available for drainage and then move horizontally to intersect 0.0 line.

v) Join above point with value of P by a straight line. Extend the line to q. Multiply q by one hour rainfall I_0 to obtain expected discharge.

5.8.3 Orifice Formula

The volume of flow may also be calculated from measurements taken on an existing bridge over the same river, by using the Orifice formula:

$$Q = C_0(2g)^{1/2}.L.Dd\left[(Du - Dd) + (1+e)\frac{V^2}{2g}\right]^{1/2} \qquad (5.7)$$

where

Q = volume of flow (m^3/sec)

g = acceleration due to gravity (9.8m/sec^2)

L = linear waterway, i.e. distance between abutments minus width of piers, measured perpendicular to the flow (m)

Du = depth of water immediately upstream of the bridge measured from marks left by the river in flood (m)

Dd = depth of water immediately downstream of the bridge measured from marks on the piers, abutments or wing walls (m)

V = mean velocity of approach (m/sec)

C_o and e = coefficients to account for the effect of the structure on flow, as listed in Table 5.3. Some parameters used in the above formula are shown in Figs. 5.6 and 5.7. The formula is claimed to give nearly correct volumes for most waterway shapes, but Q should be increased by 5 per cent when Du-Dd is greater than $\frac{Dd}{4}$.

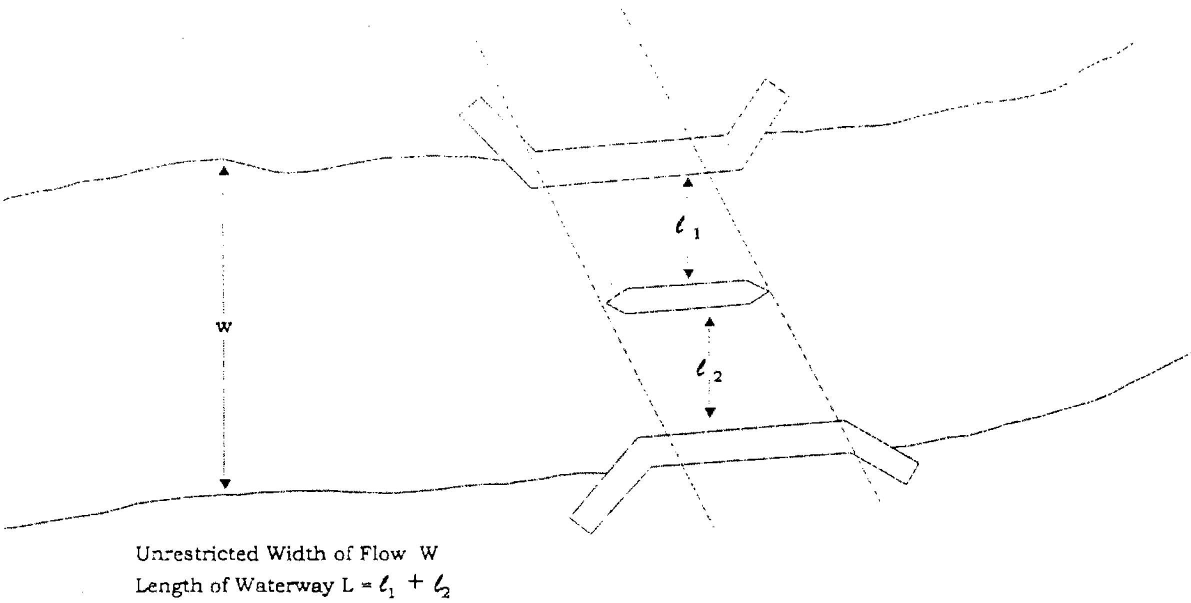


Fig. 5.6 Waterway at a Bridge

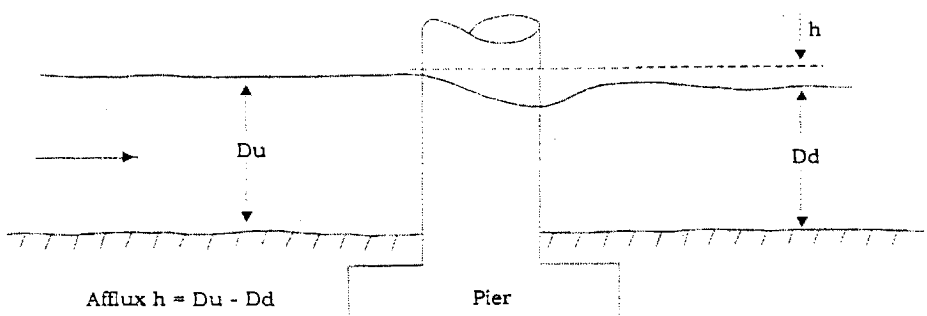


Fig. 5.7 Afflux

L = width of waterway as defined in Fig. 5.6

W = unobstructed width of the stream

Table 5.3: Values of C_o and e in the Orifice Formula

L/W	C_o	e
0.5	0.892	1.05
0.55	0.88	1.03
0.6	0.87	1.00
0.65	0.867	0.975
0.7	0.865	0.925
0.75	0.868	0.86
0.8	0.875	0.72
0.85	0.897	0.51
0.9	0.923	0.285
0.95	0.96	0.1225
Note : Intermediate values may be obtained by interpolation		

5.8.4 Float Method

Three cross sections should be selected along a reach of straight channel. The middle section will be the measuring section. The cross section should be spaced far enough apart for the time the float takes to pass from one cross section to the next to be measured accurately. A travel time of 20 seconds is recommended for the float to run from upper to the lower cross section, but a shorter time may have to be used on small rivers with high velocities, where it is often impossible to select an adequate length of straight channel. A typical plan is shown in Fig. 5.8.

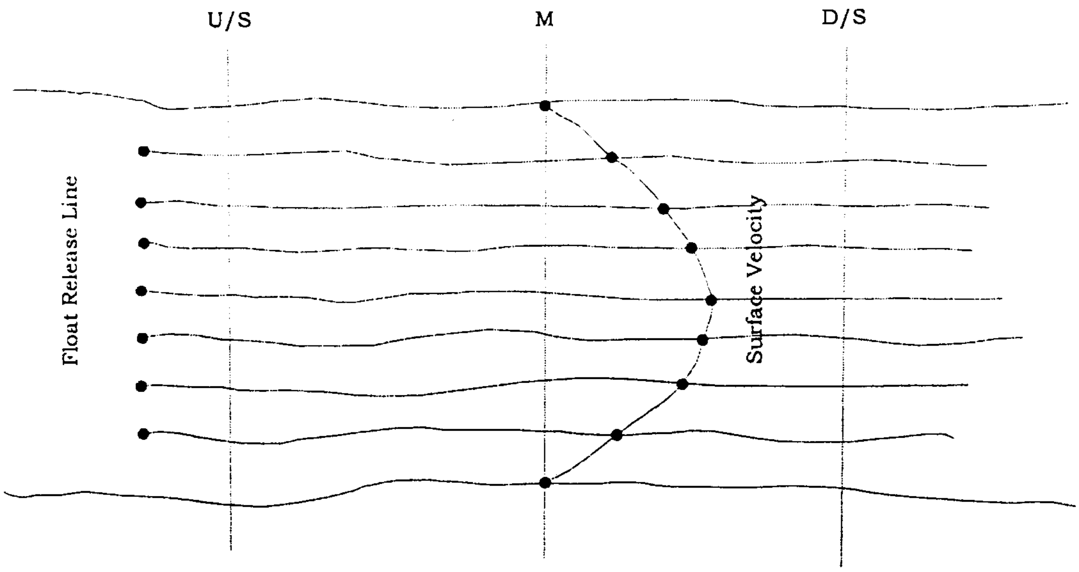


Fig. 5.8 Schematic Diagram Showing Discharge Measurement

Float : Surface velocity may be gauged by throwing in any floating body which exposes as small a surface to the air as possible. Circular disc of any hard wood about 3" diameter are considered best.

Measuring Procedure : Floats must be uniformly distributed over the stream width as far as practicable and their number shall depend on width of the stream. The float shall be released far enough above the upstream cross section to attain a constant velocity before reaching that cross section. The distance of throw may be about 20 feet above the upper line and the time at which the float crosses three cross sections (u/s, mid and d/s) shall be noted by stop watch. This procedure shall be repeated with each floats for the total number of floats used. Distance of the float as it passes the mid cross section may be determined by suitable optical means, for example, a theodolite. For small streams this may be done by graduated steel tape.

Elevation of the channel bed i.e. depth of water at points in the mid cross section may be read directly on a graduated rod set on the stream base in case of shallow streams, otherwise, it should be done by lowering a weight from a wire and depth measured at the point where the weight touches the stream bed. This exercise may be done by boat. Caution is necessary in alluvial streams to prevent the weight from settling through soft bed material. While measuring depth by wire and weight it should be carefully observed that wire remains perpendicular to the water surface as the accuracy of the measurement is increased if sufficient weight can be used to maintain the line (wire) in a nearly vertical position. These depth measurements are used to draw the profile of the mid cross section.

Velocity : Surface velocity is computed from the time required to pass the distance by the float from upper to lower cross section :

$$v = \frac{d}{t}$$

where v = surface velocity at the point a float crosses the mid cross section in m/sec

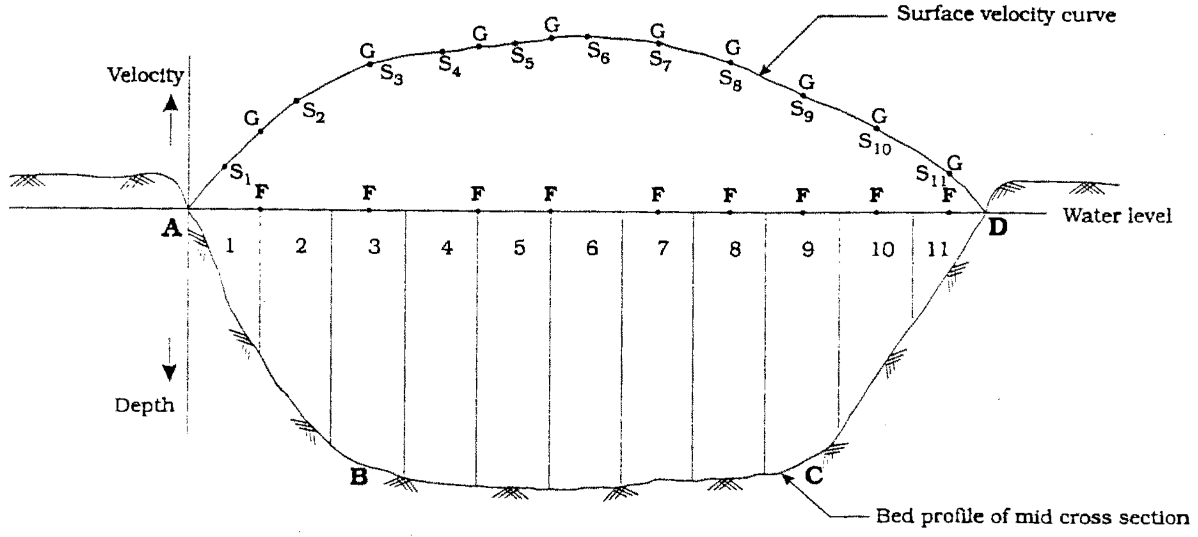
d = distance from upper to lower cross section in meter

t = time required for the float to travel from upper to lower cross section in seconds.

Discharge : Method for determining the discharge from surface velocity is described below, supported by Fig. 5.9.

i) The mid cross section is plotted on the graph paper as per depth measurement taken as described in measuring procedure.

ii) The points at which the floats passed the mid cross section are indicated on the graph.



AD = Water line
ABCD is the cross section profile as referred to in Fig. 5.7
F = Points at which floats passed and surface velocities computed
G = Surface velocities at respective F points
GGG .... = Surface velocity curve
1, 2, 3 .... = Segments
S_1, S_2, S_3 .... = Surface velocities at respective segments 1, 2, 3 ....
A_1, A_2, A_3 = Areas of segment 1, 2, 3 ....

From formula $Q = K\Sigma VA$

$$\therefore Q = 0.85\{(A_1 * S_1 + A_2 * S_2 + A_3 * S_2 + \ldots\ldots A_{11} * S_{11})\}$$

Mean velocity of flow in the whole cross section

$$V = \frac{Q}{\text{Sum of the areas of the segments}}$$
$$= \frac{Q}{\{A_1 + A_2 + A_3 \ldots\ldots A_{11}\}}$$

Fig 5.9 Computation of Discharge

iii) The velocity of floats i.e. surface velocities v at different points as in (ii) are plotted upward from the water level line in the graph and these surface velocities thus plotted are connected by firm line which will present the profile of surface velocity at the section.

iv) Cross section is then divided into desired number of segments (panel) and computed areas of segments and the surface velocities in respective segments from velocity curve as in (iii).

The discharge in each segment is computed by multiplying the area of cross section of the segment by mean velocity of flow (equal to surface velocity multiplied by co-efficient of surface velocity to mean velocity of flow). Total discharge Q is then equal to the sum of discharges in the segments and can be represented by the equation.

$$Q = K \Sigma v A$$

where, Q = Total discharge in m^3/sec

v = surface velocity at a segment in m/sec

A = Area of cross section of a segment in m^2

K = Co-efficient of surface velocity to mean velocity of flow

The value of K roughly varies from 0.75 to 0.85 and here its value is recommended as 0.85.

From the above, the mean velocity of flow through the entire cross section can be computed by dividing total discharge Q by sum of the areas of segments.

5.8.5 Fixing Design Discharge

Different procedures for estimation of flood discharge have been explained in the foregoing articles. The float method of measuring discharges is a direct procedure which would quantify the discharge at the time of its occurrence. The flood discharges are usually estimated by Area Velocity Method using Manning's equation to estimate velocity corresponding to design frequency water level. However, the discharge in a proposed project site should be estimated by at least two methods and the values obtained should be compared. The highest of these two values should be adopted for the design discharge Q. If these two values differ widely the engineer has to form a judgement based on reliability of data and measurement exercise on which the calculations were based.

If the proposed structure sites are situated at the proximity of BWDB gauge stations and are hydraulically connected, the HFL and the design discharge should be based on analysing data available from those gauge stations.

At the planning stage of a project, it should be ascertained as to whether there is any hydrological data collection station at the proximity of the structure site, and required water level and discharge data are available for quite an appreciable period of time; those data should be collected, processed and analysed to arrive at logical conclusion for design of the project.

In such cases the most important point is the selection of flood frequency. Although economy tends to be a prime factor in taking up a project in this country, the question of technical soundness and durability of a structure can not be neglected altogether.

In the major highways the bridges are generally designed to accommodate a 50 years flood, although the important bridges may be designed for 100 years flood flow. This of course mostly depends on the policy and decision of the authority concerned for selection of design frequency.

Keeping in consideration of the question of economy as well as the strength and durability of the structure it is recommended that the bridge/culverts for rural roads should be designed for flood frequency of 1 in 20 years and for more important roads the desired frequency may be of 1 in 50 years.

5.9 LINEAR WATERWAY OF BRIDGES

5.9.1 Alluvial Streams

The linear waterway should be selected such that the width is sufficient to drain the design discharge without any high afflux and high velocity of flow. If the width is restricted excessive scour may result, endangering the stability of the structure. Fixing of bridge span therefore, will require consideration for flow velocity and the type of soil at the river bed and bank. Fig. 5.10 shows velocity that is required to erode particles of defined sizes. The permissible velocities for different sizes of materials other than clay are shown in article 5.15.

The length of a linear waterway of a bridge across an alluvial stream may be determined using Lacey's formula.

$$L = W = 4.75Q^{1/2} \qquad \text{.......... (5.8)}$$

where L = the linear waterway in meter

W = regime width

Q = designed discharge in m^3/s

The above equation is for streams whose width is large compared to depth of flow, therefore, it may be somewhat conservative for small streams. If the width calculated by eqn. 5.8 is large then a check should be made on the scour depth (D) corresponding to the design discharge Q by equation 5.10. If D is less than the existing scour depth

(D"), the deepest point in the cross section under consideration, then width of crossing may be modified to :

$$L = W.\left(\frac{D}{D''}\right)^{1.64} \qquad \text{.......................................(5.9)}$$

The existing scour depth D" is the deepest point in the cross section measured from the water level corresponding to the design discharge.

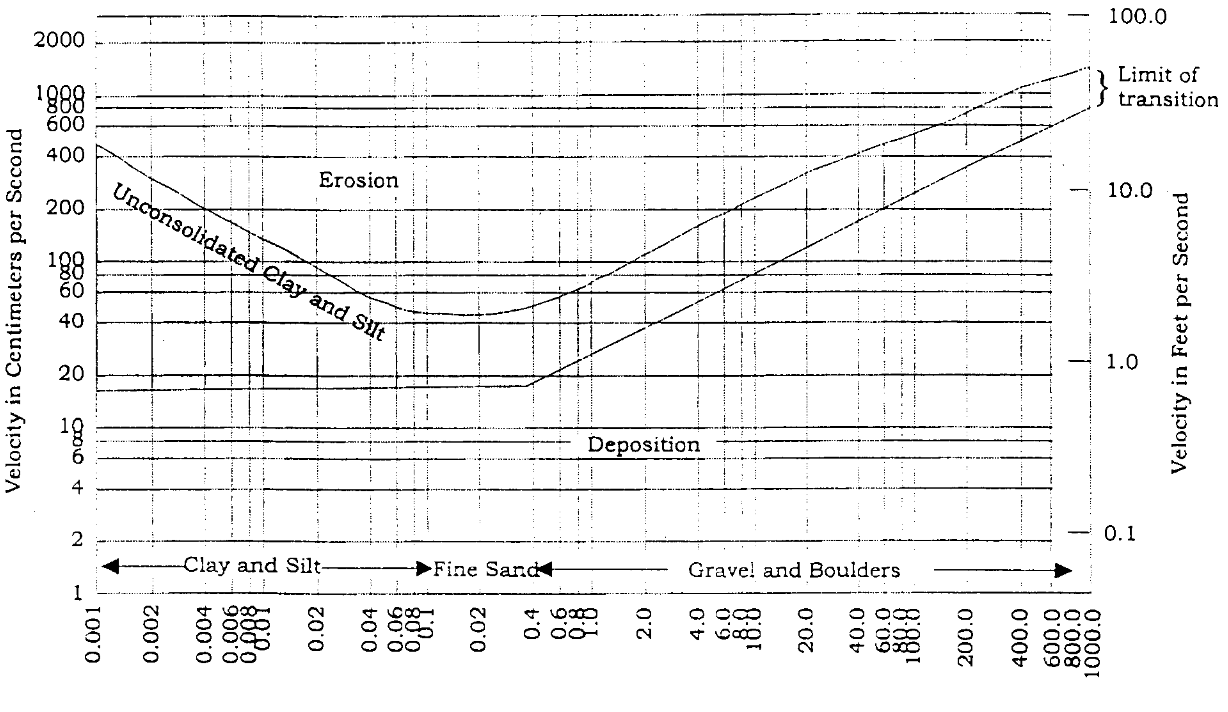


Fig. 5.10 Velocity of Flow for Sedimentary Particles at which Erosion Begins

5.9.2 Stream not Wholly Alluvial

When the banks of the stream are high, well defined and rigid (rocky or some other natural hard soil such as clay that can not be affected by the prevailing current) but the bed is alluvial, the linear waterway should be made equal to the actual surface width of the stream measured from edge to edge of water at HFL, on the plotted cross section. Such streams are referred to as quasialluvial.

5.9.3 Unstable Meandering Streams

In large alluvial streams meandering over a wide belt, there may be several active channels separated by land or shallow sections of nearly stagnant water. The total width of such stream may be much in excess of the regime width required for stability.

In bridging such a stream, if contraction is done, it will become necessary to provide training works. This will increase both the initial and recurring expenditures. Therefore, it is cheaper to adopt a linear waterway for the bridge somewhat in excess of the regime width given by equation 5.8. It is difficult to make specefic recommendation as situation is so variable.

5.10 SCOUR DEPTH

A truely alluvial stream is destined to come to regime according to Lacey. It will then be stable and have a section and slope conforming to Lacey's equation. For wide alluvial streams the stable width W can be taken as wetted perimeter P which can be calculated by equation 5.8. The corresponding normal depth of scour may be taken as

$$D = 0.473\left(\frac{Q}{f}\right)^{1/3} \quad \text{.......(5.10)}$$

where Q is the design discharge m^3/s, fixed as suggested in article 5.8, D is the depth of scour measured from the water level corresponding to the design discharge.

The term f is called silt factor and its value depends on the size and looseness of the grains of the alluvium. Its value is given by the formula $f = 1.76\sqrt{d_m}$ where d_m is the mean diameter of the particles in millimeter. Table 5.4 gives values of f for different bed materials.

Table 5.4: Lacey's Silt Factor

Type of Soil	d_m mean dia of grains in 'mm'	f, silt factor
Clayey Silt	0.05	0.4
Silt	0.12	0.6
Sandy Silt	0.21	0.8
Clayey Sand	0.26	0.9
Silty Sand	0.32	1.0
Sand	0.46	1.2

For quasialluvial streams whose banks are rigid, having a bridge span length equal to the length of the water line W between banks (the natural unobstructed width of the stream), the normal scour depth may be calculated by using the equation.

$$D = \frac{1.21Q^{0.63}}{W^{0.6} f^{0.33}} \quad \text{.......(5.11)}$$

If the linear waterway of the bridge is, for some special reason, kept less than the regime width of the stream, then the normal scour depth under the bridge will be greater than the regime depth of the stream. The modified normal scour depth D shown in Fig. 5.11 may be calculated from the regime depth D by the equation

$$D' = D.\left(\frac{W}{L}\right)^{0.61} \qquad \text{.......... (5.12)}$$

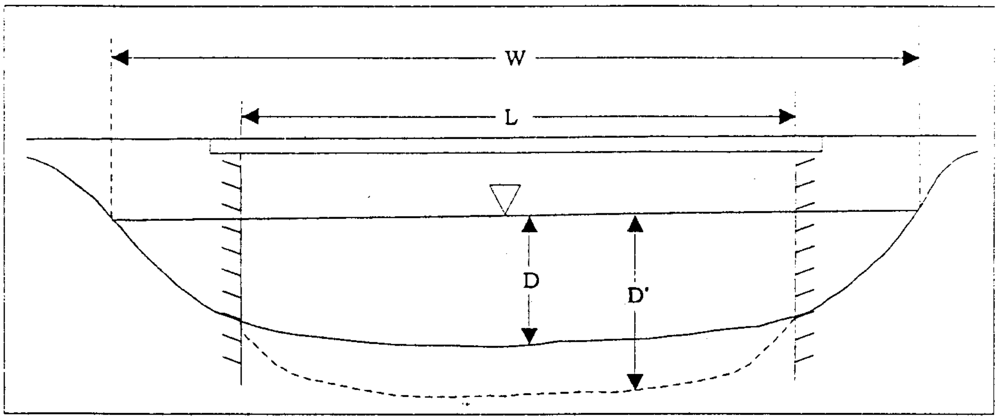


Fig. 5.11 Modified Normal Depth Due to Contraction in Stream Width

The maximum scour depth will be greater than the above calculated normal scour depth due to creation of restriction or obstruction on the channel. To determine maximum scour depth the criteria provided in Table 5.5 may be used.

Table 5.5: Maximum Depth of Scour

Location	Depth of scour measured from HFL
In straight reach	1.27 D
At moderate bends	1.50 D
At a severe bend	1.75 D
At a right angle bend	2.00 D

For constricted channel, D shall be replaced by D' in Table 5.5. Also calculate maximum scour depth by the equation given below :

$$D_{max} = D\left(\frac{W}{L}\right)^{1.56} \qquad \text{.....(5.13)}$$

The greater of the two depths calculated above may be used as design scour depth.

5.11 CONSIDERATIONS FOR ILL DEFINED DRAINAGE AREAS

In many large plain areas no deep or defined channels exist. When the rain falls, the surface water moves in some direction in a wide sheet of nominal depth, to which also water overflowing from adjacent rivers may be added. So long as this movement of water is unobstructed, no damage may occur to property or crops. But when a road embankment is thrown across the country intercepting the natural flow, relief then has to be provided by taking the water across the road through a dip or culvert.

In such flat region the road runs across a wide but shallow dip and therefore the most straight forward way of handling the surface flow is to provide suitable dips (i.e. causeway) in the longitudinal profile of the road and let water pass over them. This is recommended for flat land with high slope only where water recedes quickly. For certain time, the road will remain under water and may cause inconvenience in many ways.

However such solutions are not applicable in many situations and therefore, culverts have to be constructed on roads in such cases. The main difficulty is the estimation of flow quantity that is likely to pass through the culvert. The natural velocity of flow cannot be estimated because:

1. There is no defined cross section of the channel from which the area of cross section and wetted perimeter may be estimated.

2. There is no measurable slope in the drainage line.

Even where the velocity may be calculated or directly observed it may be so small that required area of the waterway at the culvert (A = Q/V) will be prohibitively large. In such case, the design has to be based on an increased velocity of flow through the culvert. To create this increased velocity the design must provide for heading up at the inlet end of the culvert. Economy in design being the primary consideration, the correct practice, would be to design a pipe or box culvert on the assumption that the water at the inlet end may head up to a predetermined safe level. This surface level of the headed up water at the upstream end has to be so fixed that road bank should not be overtopped, nor any property in the flood plain damaged.

Next the level of the downstream water surface should be decided. This will depend on the size and slope of the leading out channel and is, normally, the surface level of the natural unobstructed flow at the site, that prevails before the road embankment is constructed.

After above information including a rough estimate of quantity of water to flow past the culvert are obtained, the required area of cross section of the barrel of the culvert may be determined by applying the principles of hydraulics discussed in Article 5.14 culverts flowing full and Article 5.8 in which open channel flow occurs.

In all cases, adequate provision must be made at the exit against erosion by providing 'cut-off' walls . Where the exit is a free overfall, a suitable cistern and baffle wall must be added for the dissipation of energy and stilling the ensuing current.

If time permits it may be useful to provide a gap in the embankment and estimate the quantity of water that passes through such opening from measurement of flow velocity and cross- section of the opening. On the basis of adequacy of such opening for at least one flood season a culvert capable of handling slightly higher discharge (may be 10 to 25%) may be used. For new embankments, as a rough guide at least one opening at every 500m should be provided. If no time is available to leave gaps to estimate the flow, it may be handled otherwise.

By referring to large scale contour maps (1:16,000) and taking into account of the upto date development conditions around, the depth of flooding and its boundary may be determined. From the depth of flooding and its limit along the road alignment, cross sectional area may be determined referring to the design flood level. The velocity of flow along such flooded area is usually very small. If possible, it may be measured or estimated to find out the flow through the section or sections of the flooded area concerned. The area of opening for the culvert may be determined by increasing the velocity of flow.

As an example A and B are two cluster villages situated in a beel area as shown in Fig. 5.12. CD is the proposed road alignment. XY is the length of the waterway of the flooded section. To determine Q, area of waterway section may be determined with respect to design flood level. Velocity of flow for this section is normally very small but may increase with change in the downstream control. Waterway opening for this type of case may be determined as stated in the above paragraph.

5.12 HYDRAULICS OF FLOW THROUGH BRIDGES

5.12.1 Broad Crested Weir Formula

Fig. 5.13 shows the shape of water surface of flow through a bridge opening. D_u is the depth of flow upstream of the bridge and D_d is the depth at downstream of the bridge. If u is the velocity of the stream at upstream such that (D_u-D_d) is not less than $D_d/4$, then the following formula will be applicable for estimating discharge through the bridge along a width L of the bridge:

$$Q = 1.71 C_d . L \left(D_u + \frac{u^2}{2g} \right)^{3/2} \qquad \text{...(5.14)}$$

Where g is the acceleration due to gravity = 9.807m/s². Here D_u, D_d and L are measured in metres and Q in m³/s.

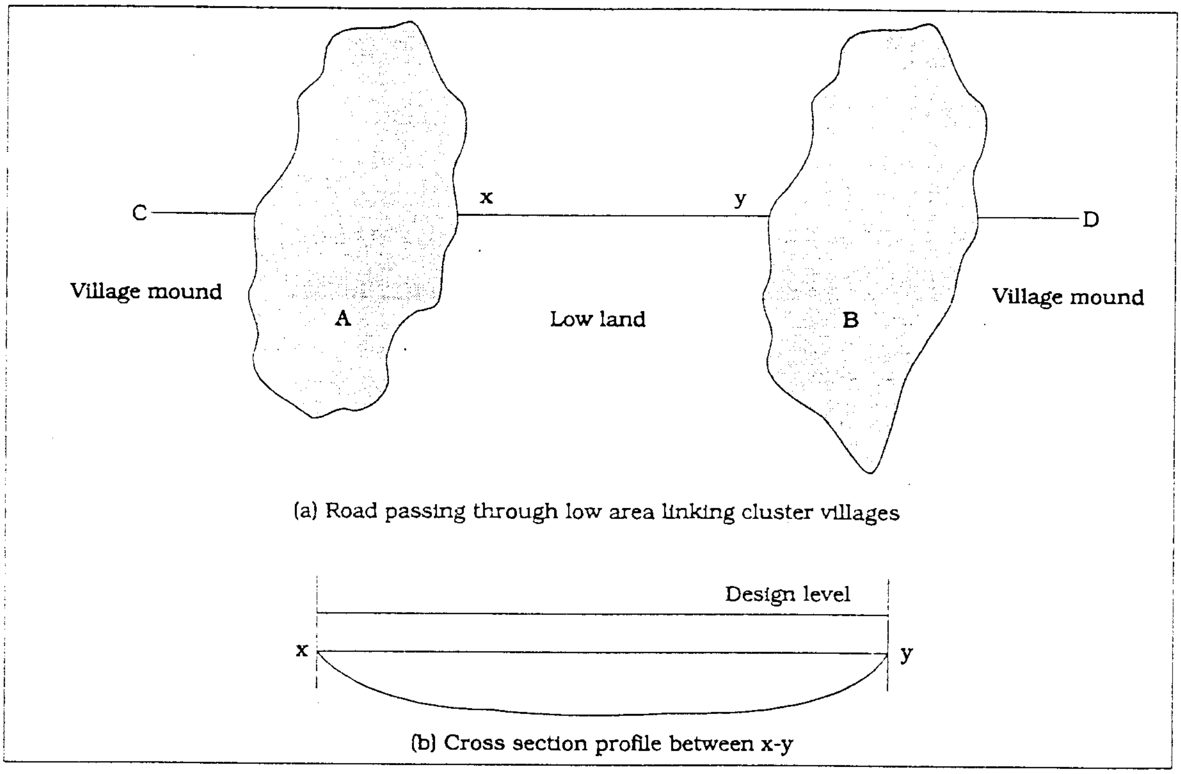


Fig. 5.12 Typical Case of Road Passing through Low Area (Undefined Channel)

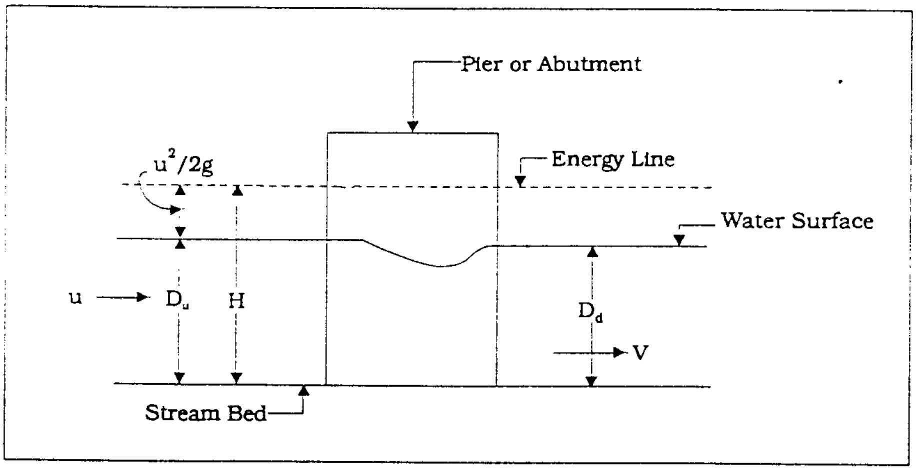


Fig. 5.13 Shape of Water Surface through a Bridge Opening

C_d is the coefficient of discharge and accounts for frictional losses. The various values of C_d for different types of openings are given in Table 5.6.

Table 5.6: Values of C_d

Type of Bridge Opening	Value of C_d
Narrow bridge opening with or without floors	0.94
Wide bridge opening with floors	0.96
Wide bridge opening with no bed floors	0.98

5.12.2 Orifice Formula

When the downstream depth D_d is more than 80% of the upstream depth D_u, the above formula (eqn. 5.14) does not hold good, i.e. the performance of the bridge opening is no longer unaffected by D_d. In such a case the discharge through a length L of the bridge opening will be given by the equation.

$$Q = Co\sqrt{2g}.L.D_d\left(h + (e+1)u^2 / 2g\right)^{1/2} \qquad \text{.........(5.15)}$$

$$i.e. \quad Q = 4.429C_o.L.D_d\left[h + (e+1)u^2 / 2g\right]^{1/2} \qquad \text{.........(5.16)}$$

where,
h = $(D_u\text{-}D_d)$, afflux in m,
e = a coefficient to be obtained from Fig. 5.14(a) depending on a/A and L/w ratio,
a = area of flow under the bridge, m^2
A = unobstructed area of flow of the stream, m^2
L = width of sum of bridge spans, m
w = unobstructed width of stream, m
C_o = a coefficient of friction to be determined from Fig. 5.14(b).

In the foregoing condition, the value of C_o will be less than unity when the opening of the bridge is less than the unobstructed natural waterway area of the stream. This means that when the bridge contracts the stream, afflux occurs. Under the normal situation and for small streams, the bridge opening should not be less than normal stream width. But in case of some alluvial streams in plains, the natural stream width may be much in excess of that required for regime, i.e. for a stable situation. When spanning such a stream, it has to be contracted to, more or less, the width required for stability by providing training works. In such a situation it is necessary to estimate afflux, to see its effect on the clearance under the bridge, on the stability of the channel upstream of the bridge and on the design of the training works.

For estimating afflux one has to know (1) the discharge Q, (2) the unobstructed width of the stream, w, (3) the inner waterway of the bridge L and (4) the average depth of flow downstream of the bridge D_d.

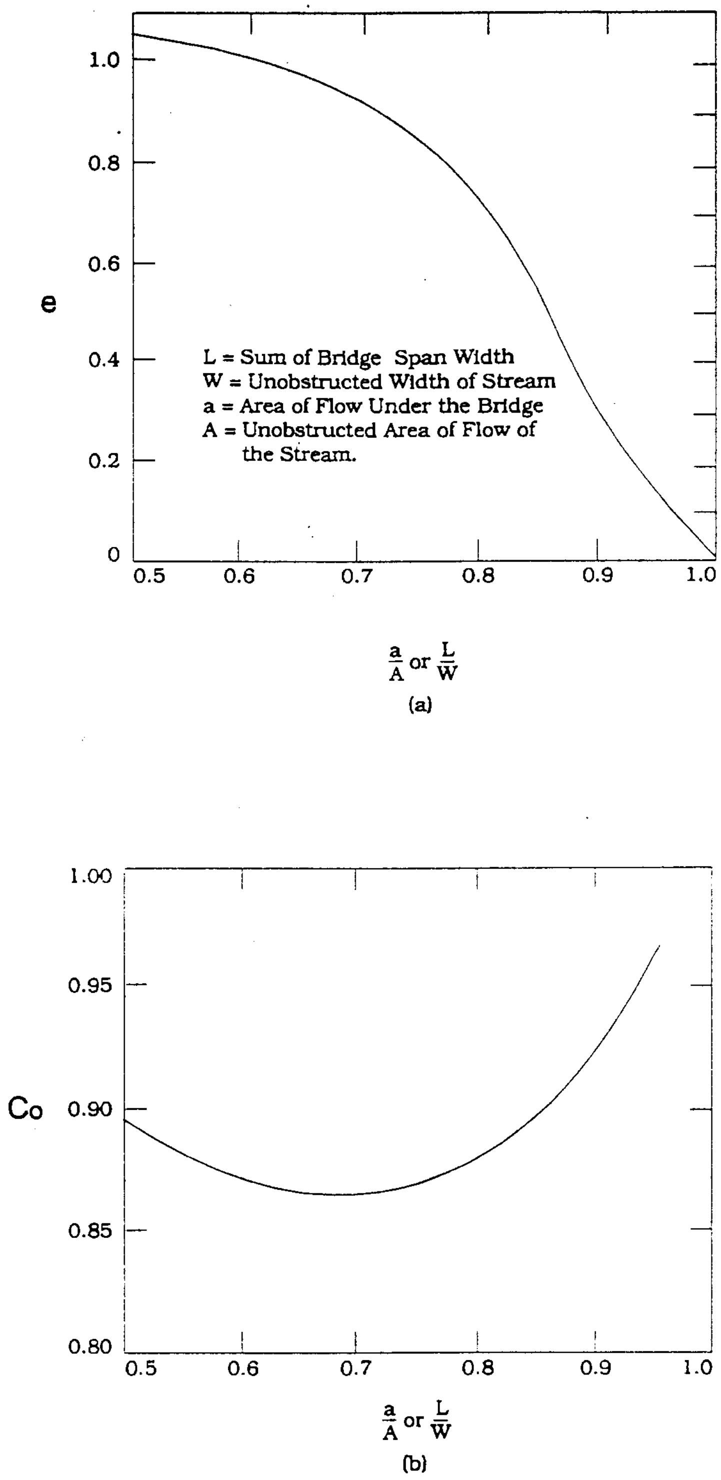


Fig 5.14 Entry and Friction Co-efficients

In the equation 5.16 for calculating discharge, the downstream depth D_d is controlled by the conveyance factor and slope of the exit channel. Also the depth that persists at the bridge site before the construction of the bridge, can be assumed to continue to persist just downstream of the bridge after its construction. Thus D_d has got to be measured or calculated from area slope data of the channel as explained in articles 5.8 & 5.10. That is, equation (5.2) can be used to estimate D_d, by considering R, the mean hydraulic depth as equal to D_d.

5.13 BACKWATER EFFECT (AFFLUX)

The restriction to free flow due to piers and abutments causes the head of water to rise on the upstream side of the structure. The water level downstream may also be lowered, but is usually assumed to be unaffected by the bridge. The engineer should estimate the rise on the upstream side to ensure that the river bank will not be breached where floods would cause damage and to check that the desired clearance for floating debris or river traffic is maintained below the superstructure.

The magnitude of back water can be affected by the shape of the river bed, its eccentricity, scour holes and related factors. It is also influenced by the shape of the bridge works, i.e. by any skew river training, embankments and relief culverts. For simple configurations, a good estimate may be made using the formula :

$$h = \frac{V^2}{2g}\left(\frac{W^2}{c^2L^2} - 1\right) \qquad (5.17)$$

where h = afflux (m)
V = average velocity of flow (m/sec)
g = acceleration due to gravity (9.8 m/sec^2)
W = unobstructed width of the stream (m)
L = linear waterway in meter
c = coefficient of discharge through the bridge, taken as 0.7 for sharp entry and 0.9 for bell mouthed entry.

Another method of finding afflux has been shown in Article 5.12 using the Orifice formula. In practice the afflux h may be directly measured by levelling the water levels, upstream and downstream of an water control structure - bridge or culvert.

The methods of calculation set out here and elsewhere in this chapter can produce only broad estimates of scour depth, afflux and other variables. Since each river bed and bridge location has distinct and complex set of characteristics, the bridge engineer is required to produce a conservative design taking into account the required service life and a realistic estimate of the quality of the maintenance available to detect and repair at an early stage any damage to the river banks and bed that may affect the structure.

5.14 HYDRAULICS OF PIPE CULVERTS FLOWING FULL

In absence of a defined channel and due to low velocity of flow, it is economical to design a culvert consisting of a pipe or a number of pipes of circular or rectangular section functioning with the inlet sub-merged. As the flood water starts heading up at the inlet, the velocity through the barrel goes on increasing. This continues till the discharge passing through the culvert equals the discharge coming towards the culvert. When this state of equilibrium is reached the upstream water level does not rise any higher.

For a given discharge the extent of upstream heading up depends on the vent way of the culvert. The latter has to be so chosen that the heading up should not go higher than a predetermined safe level, the criterion of safety being that the road embankment should not be overtopped, nor any property damaged by submergence. The fixing up of this level is the first step in the design.

It is essential that the HFL in the outfall channel near the exit of the culvert should be known. This may be taken as the HFL prevailing at the proposed site of the culvert before the construction of the road embankment with some allowance for the concentration of flow caused by the constriction of the culvert. Any structure constructed downstream later may again head up water level in the downstream and therefore, reduce discharge through the culvert. Such possibilities should be considered in fixing up the downstream water level.

In this connection two cases of flow that may be considered is shown in Fig. 5.15. In each case the inlet is submerged and the culvert is flowing full. In case (a), the tail race water surface is below the crown of the exit and in case (b) it is above that. The operating head 'H' is utilized in:

1. Overcoming the frictional resistance offered by the inside wetted surface of the culvert.

2. Supplying the energy required to generate the velocity of flow through the culvert.

3. Forcing water through the inlet of the culvert.

If the velocity of water through the pipe is v, the head expended in generating it is $v^2/2g$. Considering head expended at the entry being a fraction K_e of the velocity head and the head required for overcoming friction being K_f of $v^2/2g$, then

$$H = (1 + K_e + K_f) v^2 / 2g \qquad \text{.......(5.18)}$$

From this equation knowing H, K_e and K_f, v can be calculated.

The values of K_e and K_f depend on the shape of the inlet. The following values are commonly used :

K_e = 0.08 for bevelled or bell mouthed entry

= 0.51 for sharp edge entry.

K_f is a function of length L of the culvert, its mean hydraulic radius R and the coefficient of roughness n of the surface.

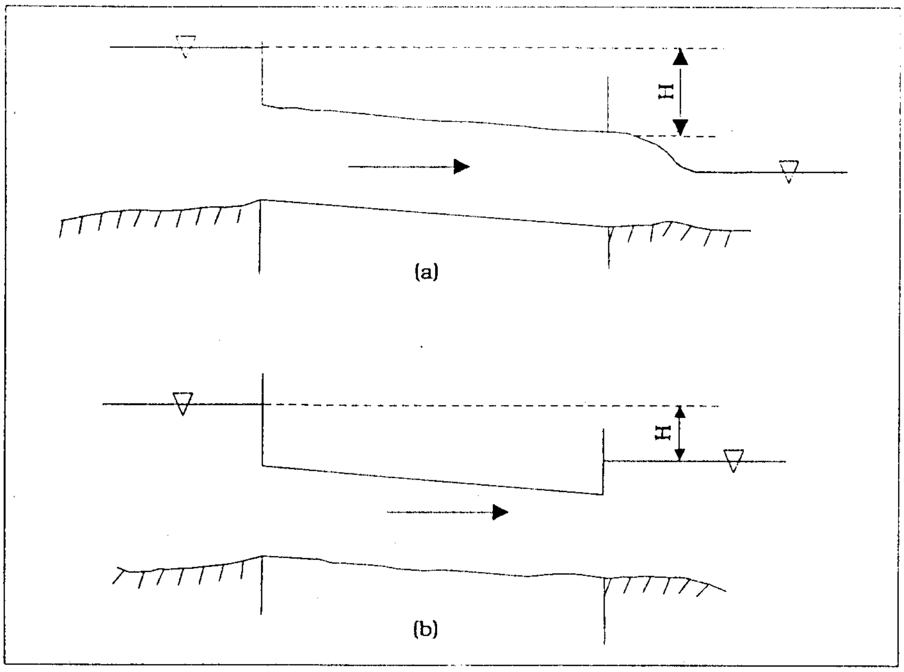


Fig. 5.15 Flow Condition for Submerged Flow in Pipes/Culverts

For cement concrete circular pipes or cement plastered masonry culverts of rectangular section, with the coefficient of roughness n = 0.015, the values recommended for K_e and K_f are shown in Table 5.7. Here R is the hydraulic mean depth of flow. Based on the coefficient in the Table 5.7 and using equation the velocity may be computed as :

$$v = \left(1 + K_e + K_f\right)^{-1/2} . \sqrt{2gH} \qquad \text{.......... (5.19)}$$

or

$$v = 4.43\left(1 + K_e + K_f\right)^{-1/2} . \sqrt{H} \qquad \text{.......... (5.20)}$$

Table 5.7: Values of K_e and K_f for $n = 0.015$

Entry and Friction Coefficient	Circular Pipes		Rectangular Culverts	
	Square Entry	Bevelled Entry	Square Entry	Bevelled Entry
K_e	$1.107 R^{0.5}$	0.1	$0.572 R^{0.3}$	0.05
K_f	$\frac{0.00394L}{R^{1.2}}$	$\frac{0.00394L}{R^{1.2}}$	$\frac{0.00335L}{R^{1.25}}$	$\frac{0.00335L}{R^{1.25}}$

The discharge/flow through the culvert may be calculated as :

$$Q = v.A = \lambda.\sqrt{2gH} \qquad \text{.......(5.21)}$$

where,

$$\lambda = \frac{A}{\sqrt{(1 + K_e + K_f)}} \qquad \text{.......(5.22)}$$

Table 5.8 shows values of conveyance factor λ in square metre for various entry conditions for circular pipes. From the values of λ, discharge Q through a pipe under a operation head H may be determined by use of Fig. 5.16.

Table 5.8 Conveyance Factor λ. (m^2) for Circular Culverts to be used in the Formula $Q = \lambda.\sqrt{2gH}$

Entry Condition	Length (m) / Diameter (m)	5	10	15	20	25	30	35	40	45	50	55	60
Entry Round Edged	0.75	0.394	0.373	0.355	0.339	0.325	0.313	0.302	0.292	0.283	0.276	0.267	0.26
	1.0	0.714	0.686	0.660	0.638	0.618	0.597	0.582	0.565	0.55	0.535	0.522	0.512
	1.5	1.637	1.600	1.560	1.520	1.485	1.45	1.420	1.395	1.365	1.340	1.315	1.295
	2.0	2.93	2.880	2.830	2.770	2.720	2.68	2.640	2.59	2.560	2.52	2.480	2.450
Entry Sharp Edged	0.75	0.381	0.333	0.319	0.308	0.297	0.288	0.279	0.271	0.263	0.257	0.251	0.245
	1.0	0.611	9.585	0.572	0.56	0.545	0.532	0.526	0.507	0.497	0.487	0.480	0.470
	1.5	1.340	1.315	1.295	1.275	1.255	1.235	1.215	1.195	1.175	1.165	1.145	1.135
	2.0	2.330	2.300	2.270	2.240	2.220	2.190	2.170	2.142	2.120	2.100	2.080	2.060

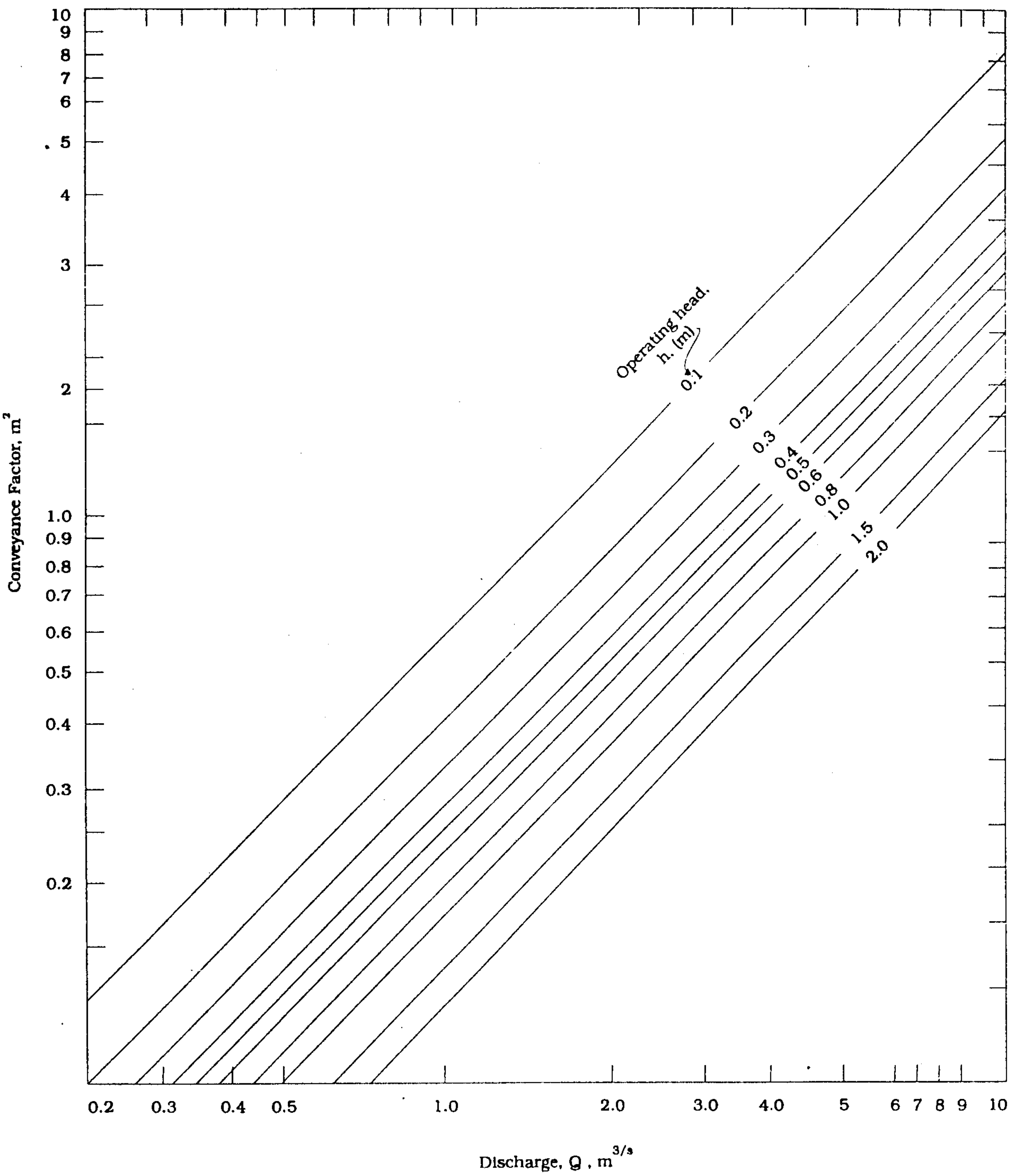


Fig. 5.16 Discharge - Conveyance Factor Relation for Pipe Culverts Flowing Full

5.15 DEPTH OF FOUNDATION

Within the scope of this Manual the structures envisaged are either single span bridges or culverts of varying sizes. The maximum limit being 20 meter in length.

In general, where no protection measure is taken, in erodible stream beds, the foundation of abutment should be taken down to a depth below the maximum scour. The foundation depth below the scour line should be a minimum 1.0m for bridges. Alternatively, the foundation should be placed at a depth $1\frac{1}{3}$ times the maximum scour measured from the HFL or $1\frac{1}{3}$ times the computed scour depth from HFL. The depth of foundation should be the greatest value obtained from the above three criteria. In the above consideration no bed floor is provided and the stream is assumed free to scour. In case of foundation of bridge piers in the river bed scour depth against the nose of the pier is taken as twice the normal scour (ref formula 5.10) for erodable beds. The depth of foundation, i.e. bottom of the pier should be well below the limit of scour.

Clay soils are relatively scour resistant, therefore, there is likely to be very little scour for this type of stream. The depth of foundation for this type of case should be at least 1m below the stream bed level.

The foundation should be carried to such a depth that the pressure on the foundation material must be well within the bearing capacity of the material. In case foundation treatment is required by piling, the top of the piles should be below bed level by at least 1.0m.

Culverts are preferable if the conditions permit due to ease and economy of construction. In most cases bed floors are provided for culverts. For erodible soils the top of the floor may be kept 0.3m below the normal bed level. The foundations of abutments (for culverts) may be taken 1.25m below the top of the floor. An upstream cut-off wall 1m to 1.25m deep and downstream cut-off wall 1.5m to 2.5m deep from the top of the floor should be provided. The depth of cut-off walls within the stated limits, will depend on the velocity of flow through the structure and erodibility of the bed material. As a rough guide scouring and non-scouring condition for various soils may be determined from the curve shown in Fig 5.10 and the following values may be recommended.

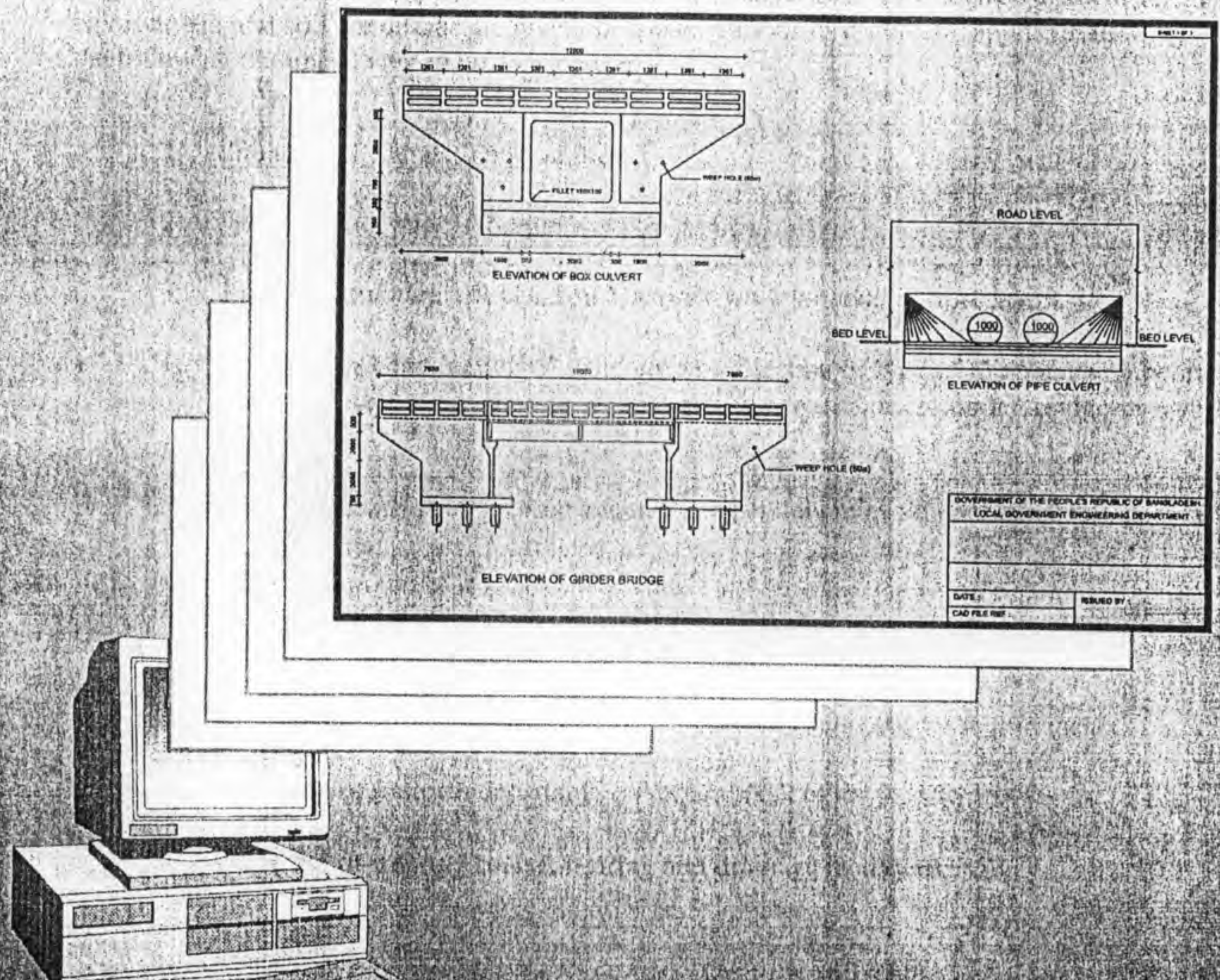
Sand/silt	0.3m/s
Coarse sand	1.0m/s
Gravel	1.5m/s

The above values may be slightly increased for stream carrying a large quantity of sediments. If above limiting velocities are exceeded, then scour depth for the stream should be calculated by an appropriate method described in Art. 5.10. In such case the wall depth should be extended to 1m below the scour depth.

SECTION III DESIGN PARAMETERS AND CRITERIA

CHAPTER 6: Design Parameters

CHAPTER 7: Design Criteria



Design Parameters

This chapter deals with the task of designing a road structure. The parameters for small scale structures (Bridges and Culverts) to be built in different areas of Bangladesh in different environment and soil conditions are given. In considerations to systematic approach to design, the following parameters are to be considered :

1. Situation of foundation structure
2. Span range of super structures in two groups.
3. Types of Structure : R.C.C and Masonry work without reinforcement.
4. Appropriate combination of structure i.e. selection of type of abutments (support system), foundation (open or piled) and supper structures (girder bridge or slab culvert)
5. List of standard structures are appended separately to avoid confusion in selection of structures.
6. Cross drains in the name of U-Drain to suit the requirement and provide the facility to irrigation activities.
7. Defining of vertical clearance for free board and navigational requirements.
8. Provision of footpath depending on requirement of the locality.
9. Provision of wearing coarse, bearing and drainage facility etc.

CHAPTER 6
Design Parameters

6.1 GENERAL

Bangladesh forms part of the Gangetic Delta - a flat alluvial plain crossed by rivers flowing from the mountains in the North down to the Bay of Bengal. Situated in the monsoon belt, the combination of prolonged rains and flooding of rivers due to snow melt in the mountains results in annual flooding of the land.

The aim of this project is to design infrastructure to develop an uninterrupted communication network in the rural area of Bangladesh, assisting drainage and minimizing the annual repair & maintenance which results from flooding. Improved infrastructure will in turn assist in the overall economic activity of rural areas.

In order to ensure strength, durability, stability and economy, suitability and cost effectiveness, a standard analysis and design procedure should be adopted. Hence, this review and revision of LGED's Road Structure Manual is a result of accumulation of field experience and the judgment of Engineers involved.

Since the foundation soil/subsoil plays a pivotal role for stability of all structures, type of soil and its physical and geotechnical properties warrant special attention. In channels concerned, incoherent alluvium flow illustrates the alluvial deposits i.e. soil deposits formed by sedimentation of soil particles from flowing water. Alluvial deposits are fine grained materials, generally silt-clay mixtures, silts or clays and fine to medium sand. These deposits are usually soft and highly compressible.

In practice, the complex properties of alluvial soil calls for conservatism in respect of judgment becomes imperative for Analysis and Design on the part of Engineers to avoid risks.

6.2 DESIGN PARAMETERS

Overall background explained above in brief and catering all possible eventualities, it can be remarked sensibly that the design parameters are derived from the consideration given below.

6.2.1 Situation of Foundation Structures

The location of all the road structures are such that flood flow during rainy season or stream flow will effect the foundation structures. Water level fluctuations will invariably change the moisture content of the soil, provide a buoyant effect, increase effective pressure with lowered water table causing additional settlement etc.

6.2.1.1 Foundation Structures On The Bank Of Regime Channel

The presence of flowing water, means that the base of the abutment must be located at a depth such that erosion or scour does not undercut the soil and cause failure to the structure. The scour depth will depend on the geological history of the site (depth of prior erosion to bedrock and subsequent redeposition of sediments, stream velocity and area run-off). Where the redeposition of sediments in the stream bed is on the order of 30 to 50m, a careful analysis of borings into the sediments to predict the depth of maximum scour* is necessary in order to provide an economical foundation.

A. It may be possible to use spread footings if they can be placed at a sufficient depth for the following structures:

   i. Brick abutment
   ii. Box culvert (Integral Foundation)
   iii. U-type culvert
   iv. Slab culvert

B. Piles are normally required to support the foundation. An accurate prediction of scour depth is necessary to use as short a pile length as possible, as in case of :

   i. R.C.C. abutment
   ii. Stub abutment

C. If careful records of driving resistances are kept, these may help to predict the scour depth which will be where the penetration resistance (SPT) increases substantially. Scour occurs principally during floods and it usually leaves a scour hole. Scour holes formed during floods are usually refilled as high water falls. Scour is accelerated if the foundation creates a channel obstruction; thus to reduce scour the foundation should create a minimum obstruction to normal stream-flow patterns.

***Note:** Scour, scour effect, scour depth etc. are explained in detail in Chapter 5.

6.2.1.2 Foundation Structures For Ill-Defined Drainage Areas

In many large plain areas no deep or defined channel exists. During and after rainfall the water flows towards the lower profile as surface runoff, forming a wide sheet of water of nominal depth. Water overflowing from the adjacent channel also adds to its depth causing a certain area to remain under water. So long as there is water flow unobstructed, no damage may be caused to the property or crops excepting the submergence of the whole area to a specific depth (may be danger level) of water. But when a road embankment runs across the area obstructing the flow of water, there will be a differential depth of water on both sides, which vehemently tends to equalize the level. Failure to do so, hydraulic gradient (difference in water level) will give rise to detrimental effect to the road embankment. In this situation, pipe culverts or box culvert may be stable and suitable structures for drainage of water.

Moreover, where there is displacement of soil because of the installation of a foundation (a common feature) and in case of expansive soil which may be very common in the ill-defined drainage areas, the curtain wall/drop wall will control the direction of expansion- by allowing the soil to expand into cavities built in the foundation.

6.2.2 Appropriate Combination of Structures

The basis of an appropriate combination of structures i.e. span and height (abutment) relationship lies in the consideration of loading systems, carriage way width, provision of side walk etc. which are explained in detail in Chapter 8. However, the type and number of structures in view of an appropriate combination have been grouped in 2 (two) ranges in consideration to span:

Length:- Range - 1 having span 1.5m to 12m
Range - 2 having span 12.5 to 20m

excepting pipe culverts, U-type culverts, box culverts and slab culverts. For easy selection of structures as well as from a view point of chronological system in analysis and design and type of structures as listed are enumerated below:

A. Pipe culverts:

- Minimum depth of fill on the structure will be 300mm
- Head walls will be provided.
- Concrete cradle bedding with positive negation (depending on soil condition) projection embankment will be provided as foundation structure.
- Pipe culverts will be designed for FRB, RR1 and RR2 & RR3 roads.

B. U-Drain

U-Drain are used mainly for irrigation purpose.

C. Box Culvert

The width of the top slab (carriage way) will be 3.66m for FRB, RR1 & RR2 and 2.44m for RR3

D. Slab culvert:

The width of the slab (carriage way) will be 3.66m for FRB, RR1, RR2 and 2.44m for RR3

E. Girder Bridge with footpath:

Loading: H10, H15 and H20

F. Girder Bridge without footpath:

Loading: H10, H15 and H20

G. Brick abutment:

- Gravity type structure with splayed type (varying depth) wing wall.
- Depending on the site conditions favourable soil/sub-soil information (bearing capacity 150 kPa min^{m}) and channel characteristics, open (shallow) foundation should be provided

H. Abutment Full Depth (Pile supported R.C. Abutment) with Footpath

- Rigid structure with return wall as suitable to the depth of abutment
- Depending on the site condition and unfavourable soil/sub-soil information (maximum scouring effect, maximum stream flow and drainage of fine grain soil towards stream or erosion problem etc.) and channel characteristics. Pile foundation (precast or cast-in-situ) should be provided.

J. Abutment Full Depth (Pile Supported R.C. Abutment) without Footpath

- Parameters involved are same as 'H' mentioned above

K. Stub Abutment with Footpath.

- Parameters involved are same as 'H' mentioned above

L. Stub Abutment without Footpath.

- Parameters involved are same as 'H' mentioned above

M. R.C.C. Open Abutment with Footpath

N. R.C.C. Open Abutment without Footpath

O. General Arrangement Girder Bridge with Footpath

P. General Arrangement Girder Bridge without Footpath

The list of structures has been presented in Annexures.

6.2.3 Free Board [Vertical Clearance] Above HFL

The clearance required above HFL for minor bridges/culverts, where there is no navigation may be considered 300mm as free board.

6.2.4 NAVIGATIONAL CLEARANCE

Generally provision should be made for adequate headroom above HFL in the water course to be bridged and this clearance is kept on the basis of the size & capacity of the vessel in terms of displacement tonnage. In this case channels leading to rural/suburban markets/growth-centers may be assumed to be water ways for small to medium size motor driven cargo boats, passenger boats etc. which do not have a high-mast. Water transports will be normally observed to use the channels during the rainy season when the channels will have adequate draft for vessels. Considering the local perspective, navigational clearance may be provided for the channels to be bridged with span 12m to 20m. The maximum clearance (head room over HFL) of 2.0m may be assumed to be sufficient which is equivalent to the requirement for a 50 D.W.T vessel.

6.2.5 Footpath

Generally localities having school, college, market, growth centers etc. and rural & suburban inhabitants should have free and safe walking facility over the bridges of large span so as to avoid the disturbance in the normal traffic flow along the carriage way. Considering the necessity in the local perspective, a footpath of clear width 450mm (excluding railing) will be provided for the bridges having span 12m to 20m (Range-2).

6.2.6 Railing and Post

Configuration of railing will be according to the previous practice of Relief & Rehabilitation Department for the projects (bridges & culverts) funded by USAID in lieu of the same according to the practice for projects under PL-480 (large bridges and culvert projects).

6.2.7 Wearing Surface

In consideration to the durability of the structure, provision of wearing surface can not be ignored. Moreover it is easy to maintain although it is not considered as structural component of Deck Slab. In acknowledgment of the effective benefits, wearing surface of the same mix-ratio of concrete for deck slab, having maximum thickness of 50mm (average) with side slopes on both from crown and parallel to traffic will be provided.

6.2.8 Bearing for Girders on Bridge Seat

Girders having span upto 12m, concrete bed block will be provided and for the span more than 12m, Elastomeric bearing will be provided to facilitate the rotation of girder end due to deflection and shock absorption.

6.2.9 Drainage

Provision for drainage for the deck slab will be made only in the direction parallel to traffic. At the interface of curb and deck slab, spouts will be provided at interval of ± 3m on both sides of carriage way depending on the uniformity in the adjustment with longitudinal span length. The material for spouts may be of PVC materials.

Design Criteria

This chapter gives guidelines for design practice and performance. The guideline consists of the standard practice of design for road structures with relevant codes such as AASHTO, ASTM, ACI and BNBC.

In consideration to the stability and durability of structures and minimum construction problems, criteria are adopted with care. Criteria are applicable to two types of load bearing structures such as reinforced concrete and masonry structures without reinforcement.

Design criteria are adopted to conform with "Working Stress Method". Where repetitive loads are involved, primary attention should be focused on stress conditions at service load levels. So, "Working Stress Method" can be justifiably adopted for design of Road Structures.

Design criteria are defined for the following parameters of loaded members composed of specific materials :

1. Stress-strain relationship within elastic limit.
2. Ultimate/ yield strengths/stresses and their limits.
3. Allowable stresses for flexure, shear and normal.
4. Slenderness effect and effective length of axially loaded members.
5. Protection against corrosion.
6. Earth pressure co-efficients.
7. Strength parameters of different type of soil in consideration to effective and stable foundation structures.
8. Factor of Safety against external stability and settlement or deformations of the structures.

CHAPTER 7
Design Criteria

7.1 GENERAL

Once the Design parameters are established compatible to the spirit and purpose of the project (as covered in Chapter 6), designers are in a position to evaluate Design Criteria to be adopted for Analysis & Design of structures. Moreover, it stimulates the designers to impart proper judgments in any event in course of their work. It is true that in absence of field data/information, any case study, data/information on failure mode and failure data (if happened so) of structures built according to the practice of existing RSM, designers feels some prohibitary problem to be more close to the field conditions and environment. Design Criteria are mainly derived from the material properties of the structures, durability, stability and interaction of foundation structure and soil.

7.2 DESIGN CODE AND STANDARDS

All structures shall be designed in accordance with the following Codes and standards:

A.	AASHTO	:	American Association of state Highway and Transportation Officials, 15th Ed, 1992
B.	ASTM	:	American Society for Testing and Materials
C.	ACI	:	American Concrete Institute
D.	BNBC	:	Bangladesh National Building Code, 1993

7.3 UNIT WEIGHT

Unit weight of the materials are furnished in Chapter 8.

7.4 METHOD OF DESIGN

Structure should be designed in Working Stress method.

7.5 UNITS

All units are in SI unless otherwise specified.

7.6 ELASTIC CONSTANTS (STRENGTH PARAMETERS) OF MATERIALS

A. REINFORCING BAR AND WELDABLE ANGLES & PLATES

- Modulus of Elasticity	:	$E_s =$	200000	N/mm^2
- Specified yield strength	:	$f_y =$	275	N/mm^2
- Corresponding maximum strain:			0.35	percent
- Poisson's Ratio μ	:		0.3	

B. REINFORCED CONCRETE [NORMAL WEIGHT CONCRETE]

Concrete shall be constituted by (1) Normal weight coarse aggregate (2) Fine aggregate having F.M not less than 1.8 and (3) Portland Cement conforming to code.

- Modulus of Elasticity : $E_c = 4700\sqrt{f'c}$ $N/mm^2 \leq$ 22000 N/mm^2
- Specified compressive strength (28 days cylinder strength) for structural use of concrete : $f'c = 21$ N/mm^2
- Corresponding to fy, maximum strain in concrete in tension : 0.0035
- Poisson's ratio : $\mu = 0.2$

C. MASONRY WORK

A masonry unit is the assemblage of bricks of specified strength and shape properly bonded together with mortar. So, the strength of the masonry unit is the function of both the strength of the bricks and the strength of mortar joints. The strength of the mortar varies with the varying mixing ratio of cement, fine aggregate and water. Therefore the strength of the bricks alone does not absolutely represent the strength properties of masonry assemblage.

- 3 (three) wythes shall be considered for single unit of abutment structure i.e. one abutment and two wing walls.
- Modulus of elasticity of masonry : $E_m \leq 15000$ N/mm^2
- Specified compressive strength of masonry at 28 day $f'_m \leq 6.0$ N/mm^2
- Poisson's ratio : $\mu = 0.2$

Note :

1. Reinforcing bars shall be deformed of Grade: 40

2. The straightening plates and angles shall be done by methods that will not produce fracture or other injury to the metal. Distorted members shall be straightened by mechanical means or as approved by the Engineer.

3. Degree of workability for concrete shall be medium.

4. In the saline zone, sulphate resistant cement shall be used, unless otherwise specified according to sulphate exposure.

5. Modulus of elasticity of masonry shall be determined by the secant method. The slope of the line connecting the points 0.05 f'm and 0.33 f'm on the stress-strain curve shall be taken as the modulus of elasticity of masonry.

6. Specified compressive strength: f'_m shall be in accordance with the following:

   a. Masonry Prism Testing : The compressive strength of masonry based on tests at 28 days in accordance with "standard Test Method for Compressive Strength of Masonry Prisms", (ASTM E447) for each set of prisms shall equal or exceed f'_m. Verification by masonry prism testing shall meet the following:

      - Testing Prior to Construction : A set of five masonry prisms shall be built and tested in accordance with ASTM E447 prior to the start of construction. Materials used for prisms shall be same as used in the project. Prism shall be constructed under the observation of the Engineer or an approved agency.

Remarks :
Presumably it will not be possible to carryout the above mentioned tests neither prior to construction nor during construction. Fortunately no testing during construction shall be required when 50% of the allowable stresses are used in design. Accordingly allowable stresses will be furnished in the next clause.

7.7 ALLOWABLE STRESSES

A. REINFORCING BARS

- Flexural reinforcement : $fs = 0.45fy \leq 125\ N/mm^2$

- Design yield strength of shear reinforcement : $fv \leq 275\ N/mm^2$

- Minimum area of shear reinforcement : $Av = 0.35\frac{b_w s}{fy}$

  Where, b_w = Web width
  S = Spacing of shear reinforcement
  fy = yield strength

- At any section of a beam, where positive reinforcement is required by analysis, the ratio P provided shall be:

$$P_{min} = \frac{1.38}{fy}$$

- Bearing stress $\leq 0.4\ f_y$

B. REINFORCED CONCRETE

1. Flexure

   - Extreme fiber stress in compression : $f_c = 0.4\ f'_c$

2. Shear in Beams and footings:

   - For members subjected to shear & flexure only : $v_c = 0.079\sqrt{f'_c}$

   - For member subjected to shear and axial compression : $v_c = 0.079\sqrt{f'_c}$

   - For members subjected to shear and axial tension :

     $V_c = 10.84\ [(0.0068 + 0.004\ (N/Ag)]\ \sqrt{f'_c}$

     Where, N = Design axial load normal to cross-section occurring simultaneously with shear force 'V' to be taken as negative for tension.

   - Maximum shear stress : $v_{max} = 0.332\sqrt{f'_c}$

   Note: V_c represents shear force of concrete
   v_c represents shear stress in concrete

3. Shear Friction

   Provisions for shear - friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concrete cast at different times.

   - Shear stress shall not exceed : $0.09\ f'_c$

4. Compressive (bearing) stress in axially loaded members $0.3\sqrt{f'_c}$ (in case of piles extended from ground or columns etc.)

5. Shear capacity of slabs & footings in the vicinity of concentrated load as in the following cases:

- Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance 'd' from the face of the concentrated load or reaction area:

- Two way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b_o is a minimum but not closer than d/2 to the perimeter of the concentrated load or reaction area:

  Shear stress shall not exceed : $0.149\sqrt{f'_c}$

6. Shear capacity of pile cap :

   Shear stresses at the critical section of the pile cap where b_o is the perimeter of the critical section and d is the effective depth, shall not exceed: $0.332\sqrt{f'_c}\,b_o d$.

7. Maximum allowable driving stresses in pile materials for top driven precast concrete pile : a. 0.85 f'_c (compression)
   b. 0.70 fy of steel reinforcement (tension)

C. MASONRY WORK

1. Flexure

   - Compressive stress : $F_b = 0.167\ f'_m \leq 5\ N/mm^2$

   - Tensile stresses without tensile reinforcement is dependent on the type of joints and mortars which are appended below:

     a. Normal to Bed Joints : $Ft_B = 0.1\ N/mm^2$ for mortar M_1 & M_2

     b. Normal to Head Joint : $Ft_H = 0.2\ N/mm^2$ for mortar M_1 & M_2

   Where mortar M_1 represents the ratio, cement : sand = 1:3
   and Mortar M_2 represents the ratio, cement : sand = 1:4

   - Tensile stresses for mortar 1:5 & 1:6, value in a & b above should be reduced to 25%

2. Shear stress for flexure member when no reinforcement in use :
   $F_b = 0.0415\sqrt{f'_m} \leq 0.125\ N/mm^2$

3. Bearing stress

- Unreinforced masonry walls and columns subjected to compression only, axial stress : $F_a = \frac{f_m}{10}[1-(h'/42t)^3]$

Where, h' = Effective height of wall or column & t = effective thickness of a wythe wall or column.

7.8 EFFECTIVE LENGTH FACTOR, 'K' FROM PILES EXTENDED FOR GROUND & COLUMN, ETC.

The effective length of a column (any member behaving like a column) KL, has been used in the equations for allowable compression stress in the members as stated above. K is the ratio of the effective length of an idealized pin-end column or piles in cap (pile cap free to translation or side sway) to the actual length of a member with various other end conditions. KL represents the length between inflection points of a buckled member. Restraint against rotation and translation of the member ends influences the position of the inflection points in a member. The theoretical value of 'K' for the idealized vertical members (pile or pile group restraint by pile cap) free to translate is given below:

a. Theoretical value of K = 1

b. Design value of K when ideal conditions are approximated : K = 1.2

7.9 PROTECTION AGAINST CORROSION

The following minimum concrete cover should be provided for reinforcement:

		Minimum cover (mm)
a. Concrete cast against and permanently exposed to earth	:	75
b. Concrete in 'a' above in Marine environment	:	100
c. Concrete exposed to earth or weather		
- Primary reinforcement	:	50
- Stirrups, ties and spirals	:	40

d. Concrete deck slab in mild climates

- Top reinforcement	:	50
- Bottom reinforcement	:	25

e. Concrete not exposed to weather or in contact with ground

- Primary reinforcement	:	40
- Stirrups, ties and spirals	:	25

f. Precast Concrete piles or piles cast against and/or permanently exposed to earth	:	50
g. Concrete members in 'f' above in marine environment	:	75

Note : In saline zone extra 25mm cover shall have to be provided in addition of the above concrete cover.

7.10 EARTH PRESSURE COEFFICIENT

It is evident that back fill parameter γ_s (unit wt. of soil) & φ (angle of internal friction) are needed for earth pressure computation. It is implicit that in at least a limited zone defined by abutment and wing/return wall a granular backfill will be used and earth pressure co-efficient shall be taken K_o, where K_o being an effective stress state at zero strain. This co-efficient in this case is appropriate providing a more conservative wall pressure.

$$K_o = 1 - \sin \phi$$

7.11 ULTIMATE FRICTION FACTORS FOR DISSIMILAR MATERIALS

The following table should be used for general guidance in selecting sliding friction factor between wall base and foundation soil.

Interface Materials	Friction Factor, f (DIM)
a. Mass concrete on clean sand to medium sand, silty medium to coarse sand, silty or clayey gravel.	0.45 to 0.55
b. Mass concrete on fine sand, silty or clayey fine to medium sand.	0.35 to 0.45
c. Mass concrete on fine sandy silt, nonplastic silt.	0.30 to 0.35

d. Mass concrete on medium stiff and stiff clay and silty clay	0.3 to 0.35

Note: 1. Masonry on foundation materials has the same friction factors.

2. Additive resistance resulting from adhesion of the soil is neglected so that a more conservative value against the sliding is obtained.

7.12 FACTOR OF SAFETY (Wherever Applicable)

Ultimate value	Factor of safety
a. Ultimate bearing capacity of open (shallow) foundation.	3.0
b. Ultimate bearing capacity of deep foundation (pile bearing capacity - individual pile).	3.0
c. Ultimate bearing capacity of pile group.	2.5
d. Against overturning	2.0
e. Against sliding	1.5

7.13 SOIL CONDITION

Structures adjacent to the channel having stream flow and the sites experiencing fluctuation of water table, will be defined evidently in the sites of fine grained soil subjected to drained loading condition.

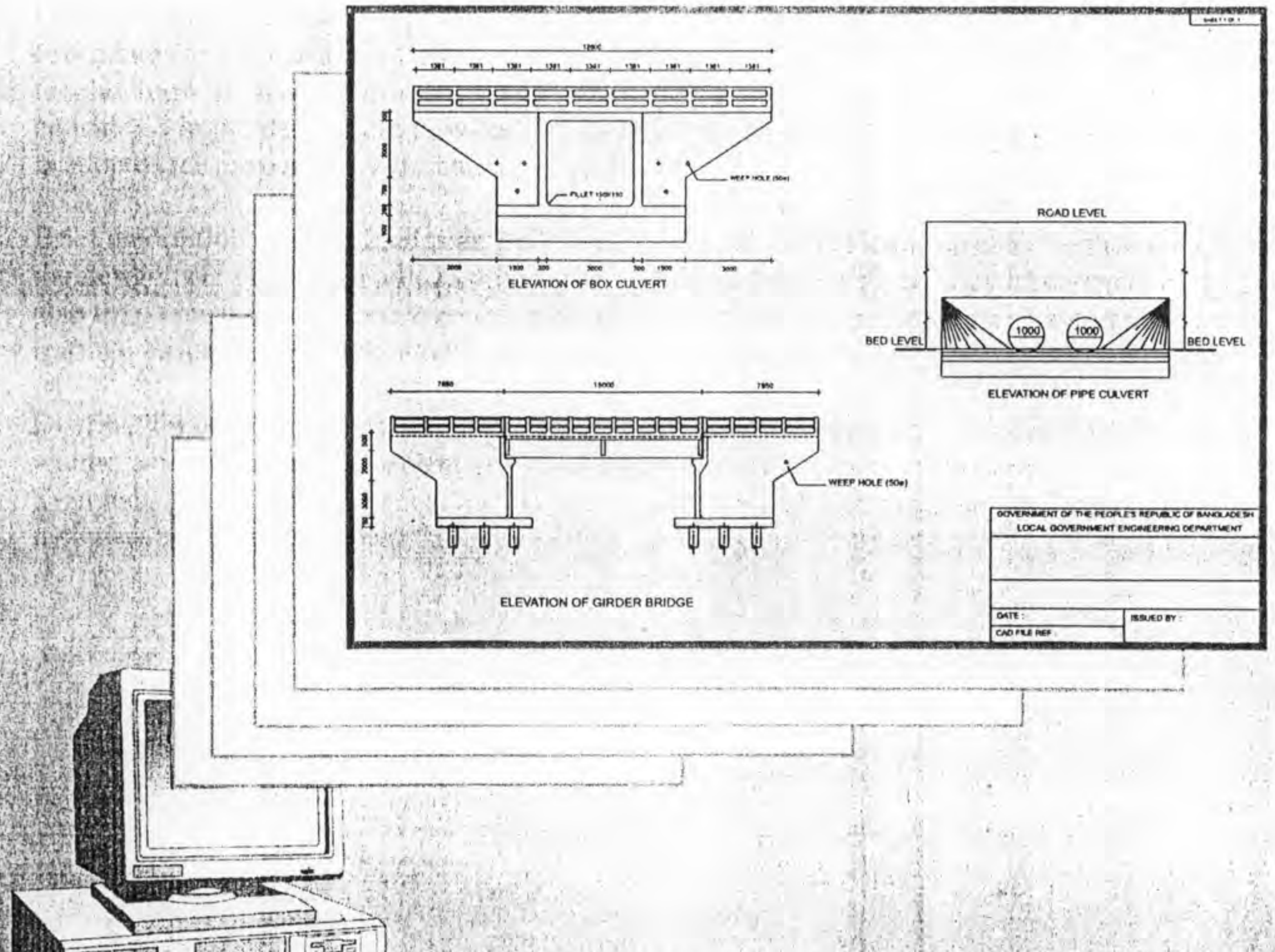
SECTION IV DESIGN CONSIDERATION AND PROCEDURES

CHAPTER 8: Superstructure for Bridges and Slab Culverts

CHAPTER 9: Superstructure and Foundation for Bridges and Slab Culverts

CHAPTER 10: Pipe Culverts

CHAPTER 11: Box Culverts



Superstructure for Bridges and Slab Culverts

This chapter describes the design considerations and procedures for superstructure of bridges and slab culverts. For bridges, simply supported single spans ranging from 6.0m to 20.0m and two different widths of carriageway i.e. 2.44m and 3.66m have been considered. Procedure for calculations of dead load and unit weight of relevant construction materials in tabular form is given. Different type of live loads and its impact or dynamic effect as considered in the design and provided in ASSHTO is described. Service load design method is adopted for the design of structural elements.

Brief description of wind load, side walk load, curb load, railing load and their application in the design procedure is given. Distribution of loads and design procedures of concrete deck slab for (i) main reinforcement perpendicular to traffic and (ii) main reinforcement parallel to traffic is described. Procedure for of cantilever slab is also described.

Distribution of wheel loads and other design features of girders such as compression flange width, minimum reinforcement requirements, transverse deck slab reinforcement in T-girders, lateral reinforcement of flexural members and reinforcement of compression members has been described.

CHAPTER 8

Superstructure for Bridges and Slab Culverts

8.1 GENERAL

Superstructures are the upper part of a bridge structure that directly carry traffic loads. Deck slab, beams or girders, footpaths, railings, rail posts etc. are considered as elements of superstructure of a bridge. Reinforced concrete bridge superstructures with spans ranging from 6.0m to 20.0m are provided in this RSM.

The bridges have been designed with two different widths of carriageway viz. 2.44m and 3.66m. For bridges larger than 12m pedestrian footpaths on both sides have been provided.

The structural system of the superstructure consists of a continuous one-way slab, forming the roadway supported by two numbers of girders. The girders are simply supported at the abutments. Railings have been provided on both sides of the roadway flanked by a pair of curbs to separate traffic flow and pedestrian. Two diaphragms have been used at the end of girders for spans upto 12m. One additional diaphragm has been provided at the point of maximum positive bending moment for spans exceeding 12m.

For bridges upto 12m span, girders have been rested on concrete seats. For bridges larger than 12m, elastomeric bearings have been used.

8.2 LOADS AND FORCES FOR DESIGN

8.2.1 General

The following loads and forces shall be considered in the design of superstructures of bridges:

- Dead load;
- Live load;
- Impact or dynamic effect of live load; and
- Wind load

8.2.2 Dead Load

The dead load shall consist of the weight of the structure complete including the roadway, sidewalks, pipes, conduits, cables and other public utility services. The unit weights to be used for computing dead loads are presented in Table 8.1 below:

Table 8.1 Unit Weights of Materials to be used for Dead Load Computation

Sl. No.	Name of Material	Weight (Kg/m^3)
1.	Steel or cast steel	7840
2.	Cast iron	7200
3.	Timber, treated or untreated	800
4.	Concrete, plain or reinforced	2400
5.	Compacted sand, earth, gravel or ballast	1920
6.	Loose sand, earth or gravel	1600
7.	Macadam or gravel, rolled	2240
8.	Pavement other than wood block	2400
9.	Brick masonry	1920

8.2.3 Live Load

The live load shall consist of the weight of the applied moving load of vehicles, cars and pedestrians. Provisions of AASHTO regarding live loadings shall be applied for designs. As per LGED requirements structures will be designed using H20, H15 and H10 loadings. Structures designed with H20 loadings are to be used for FRB roads and those with H15 loadings for RR1 and RR2 roads. For RR3 roads H10 loading is to be considered.

The AASHTO H loadings consist of a two-axle truck or the corresponding lane loading as illustrated in Fig. 8.1A and 8.1B.

8.2.4 Impact or Dynamic Effect of Live Load

Live load stress produced by H loadings shall be increased for items in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowance shall not be applied to items in Group B.

(i) Group A - Impact shall be included in

1) Superstructure including supporting columns.

2) The portions above the groundline of concrete or steel piles that support the superstructures.

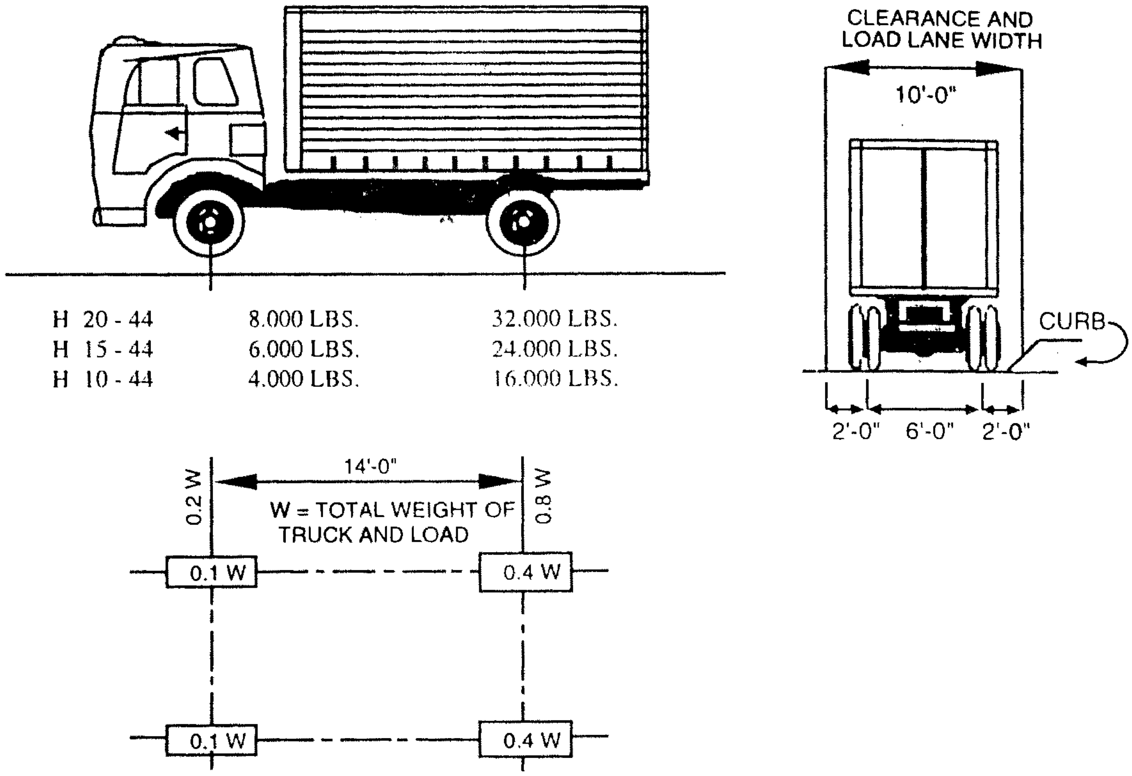


Fig. 8.1A Standard H Trucks

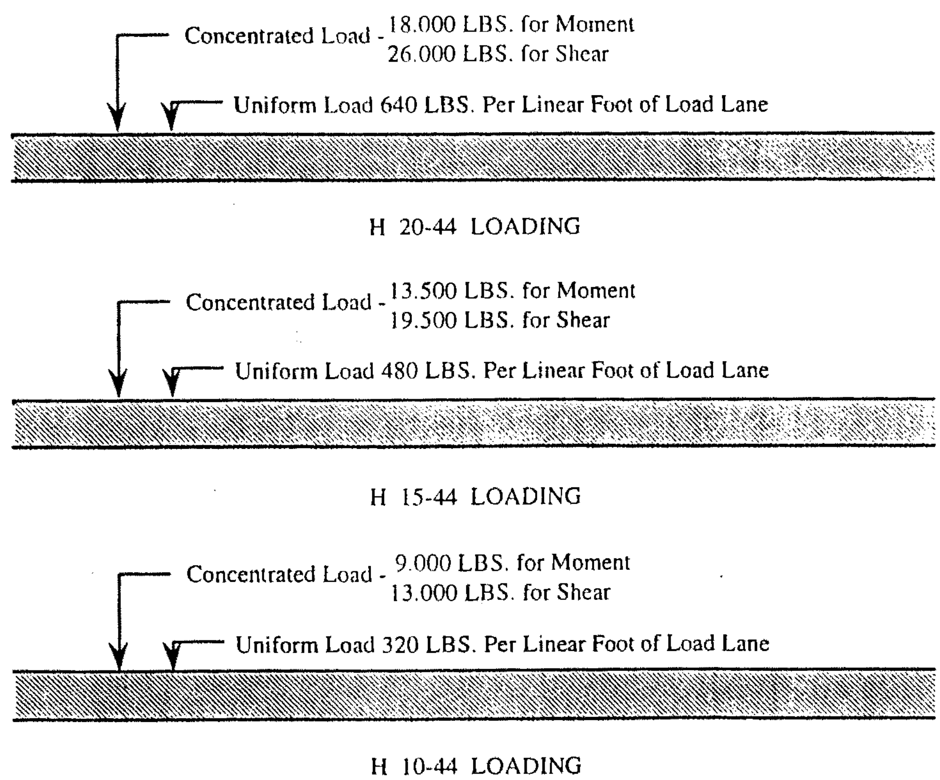


Fig. 8.1B Lane Loading

(ii) Group B - Impact shall not be included in

1) Abutments, retaining walls, piers, piles except as specified in Group A above.

2) Foundation pressures and footings.

3) Sidewalk loads.

4) Culverts and structures having 0.9m or more cover.

The amount of impact allowance or increment shall be calculated as a fraction of the live load stress as follows:

$$I = \left(\frac{15.24}{L+38}\right)$$

in which

I = impact fraction (maximum 30 percent).

L = length in meter of the portion of the span that is loaded to produce maximum stress in the member.

For uniformity of application, in this formula, the loaded length L, shall be as follows:

(a) For roadway floors : the design span length.

(b) For transverse members, such as floor beams : the span length of members center to center of supports.

(c) For computing truck load moments : the span length, or for cantilever arms the length from the moment center to the farther most axle.

(d) For shear due to truck loads : the length of the loaded portion of the span from the point under consideration to the far reaction; except, for cantilever arms, use a 30 percent impact factor.

For culverts with fill thickness, t =

$0m < t \leq 0.3m, I = 30\%$

$0.3m < t \leq 0.6m, I = 20\%$

$0.6m < t \leq 0.9m, I = 10\%$

8.2.5 Wind Loads

A wind load of 244 Kg per square meter (50 lb/ft^2) shall be applied horizontally at right angles to the longitudinal axis of the structure. The total force shall not be less than 446 Kg per linear meter (300 lb/ft) on girder spans.

8.3 APPLICATION OF LIVE LOAD

8.3.1 Traffic Lane Units

In computing the stress, 3.66m lane load or single standard truck shall be considered as a unit load, and fractions of load lane widths or trucks shall not be used. For roadway width less than 3.66m the actual width shall be used.

8.3.2 Number and Position of Traffic Lane Units

All bridges will be designed for single lane. Therefore, a single truck or a single lane load as in Fig. 8.1A and 8.1B shall be considered for design.

8.3.3 Loading for Maximum Stress

The bridge girders will be designed as simply supported girders using the working stress method and shall be proportioned to withstand safely the applicable group load combinations. The following combinations are found to govern and shall be used for the design of girders:

Group 1 : D+L+I; stress 100%; wheel at 0.6m from face of curb.

Group 2 : For all loadings less than H20 - D+2(L+I); stress 150%; wheel at 0.6m from face of curb.

Group 3 : When girder supports sidewalk -

(i) D+L+I+SWLL; stress 125%; wheel at 0.6m from face of curb.

(ii) D+L+I; stress 150%; wheel at 0.3m from face of rail.

where, D = Dead load, L = Live load, I = Impact and SWLL = Sidewalk live load.

8.3.4 Sidewalk Loading

Sidewalk floors shall be designed for a live load of 415 kg per square meter of sidewalk area. Girders shall be designed for the following sidewalk live loads:

- Spans 0 to 7.6m in length ---------------- 415 Kg/m^2
- Spans above 7.6m to 20m in length ------ 293 Kg/m^2

8.3.5 Curb Loading

Curbs shall be designed to resist a lateral force of 744 Kg per linear meter of curb, applied at the top of the curb.

8.3.6 Railing Loading

The maximum design loading (W) for a railing shall be 360 Kg per linear meter, transversely and vertically, acting simultaneously on each longitudinal member. Posts shall be designed for a transverse load of WL (where L is the spacing of posts) acting at the center of gravity of the upper rail, or for high rails, at 1.5m above the walkway.

8.4 REDUCTION IN LOAD INTENSITY

All bridges will be designed for single lane. Hence, no reduction in load intensity will be considered.

8.5 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS

It will be assumed that when a concentrated load is placed directly over one of the beams of a T-beam bridge, the load is not carried entirely by that beam. The concrete slab will be sufficiently rigid to transfer part of the load to adjacent beams (i.e. girders). No distribution will be assumed in the direction of the span of the member. The effective span length as in Article 8.5.1 below shall be used in calculating distribution of loads and bending moments for slabs.

8.5.1 Span Length

For simply supported slab bridges/culverts the span length shall be the distance center to center of supports but not to exceed clear span plus thickness of slab. For girder bridges the slab is monolithic with the beam (without haunches) and the span length will be the clear span between girders.

8.5.2 Edge Distance of Wheel Load

In designing slabs, the center line of wheel load shall be assumed to be placed at 30 cm distance from the face of the curb.

8.5.3 Bending Moment

The bending moment per linear unit width of slab shall be calculated according to the following methods:

A. Main Reinforcement Perpendicular to Traffic (Girder Bridges)

The live load moments for simple span can be determined by the following simple formula: (without impact):

H-20 loading:

$$\left(\frac{S+0.61}{9.74}\right)\times P_{18} = \text{Moment in kN-m per meter width of slab}$$

H-15 loading:

$$\left(\frac{S+0.61}{9.74}\right)\times P_{13.5} = \text{Moment in kN-m per meter width of slab}$$

H-10 loading:

$$\left(\frac{S+0.61}{9.74}\right)\times P_{9} = \text{Moment in kN-m per meter width of slab}$$

where S = effective span length in meter

P = load on one rear wheel to truck (P_{18} or $P_{13.5}$ or P_9)

P_{18} = 71 kN; $P_{13.5}$ = 53 kN; P_9 = 36 kN

B. Main Reinforcement Parallel to Traffic (Slab bridges)

For wheel loads, the distribution width E in feet shall be (4+0.06S) but shall not exceed 7 ft. or 2.13m. Lane loads shall be distributed over a width of 2E.

8.5.4 Shear and Bond

Slabs designed for bending moment in accordance with Article 8.5.3 shall be considered satisfactory in bond and shear.

8.5.5 Cantilever Slabs

Cantilever slabs will be provided on both sides of the carriageway of girder bridges. The slabs shall be designed to support the load independently of the effects of any edge support along the end of the cantilever. Considering main reinforcement in slabs perpendicular to traffic the following formulas will be used for distribution of loads on cantilever slabs:

$$E = 0.8X + 1.143$$

$$M = (P/E) \times X$$

where E = width of distribution of the load in meters;
X = distance in meters from load to point of support;
P = load in kN;
M = moment in kN.m per meter width of slab.

8.5.6 Longitudinal Edge Beams

Edge beams shall be provided for all slabs having main reinforcement parallel to traffic (for slab bridges). The edge beam of a simple span shall be designed to resist live load moment of 0.10 PS, where

P = wheel load in kN
S = span length in meter.

8.5.7 Distribution Reinforcement

To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of slabs. The amount of distribution reinforcement shall be the percentage of the main reinforcement steel required for positive moment as given by the following formula:

For main reinforcement parallel to traffic:

$$\text{Percentage} = \frac{55}{\sqrt{S}} \quad \text{Maximum } 50\%$$

For main reinforcement perpendicular to traffic:

$$\text{Percentage} = \frac{121}{\sqrt{S}} \quad \text{Maximum } 67\%$$

where S = the effective span length in meter.

8.6 DISTRIBUTION OF WHEEL LOADS TO LONGITUDINAL GIRDERS

In calculating end shears, end reaction and bending moments in longitudinal girders, no longitudinal distribution of the wheel loads shall be assumed. The live load supported by the girder will be the reaction of the truck wheels, assuming the flooring to act as a simple beam between the girders. The dead load supported by the girders shall be that portion of the floor slab carried by the girder. Dead load of curbs, railing which will be placed after the slab has cured, will be distributed equally to all girders.

8.7 COMPRESSION FLANGE WIDTH

The total width of slab effective as a T-girder flange shall not exceed one-fourth of the span length of the girder. The effective flange width overhanging on each side of the web shall not exceed six times the thickness of the slab or one-half the clear distance to the next web.

8.8 DIAPHRAGMS

Diaphragms (i.e. cross-beam) shall be provided to the girders to act as stiffeners. For girders of span more than 12m, one intermediate diaphragm will be provided.

8.9 DEEP BEAMS

If the depth of the side face of a member exceeds 900mm longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance d/2 nearest the flexural tension reinforcement. The area of skin reinforcement per foot of height on each side face shall be $\geq$ 0.012 (d-30). The maximum spacing of skin reinforcement shall not exceed the lesser of d/6 or 300mm. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement.

8.10 REINFORCEMENT OF FLEXURAL MEMBERS

8.10.1 Minimum Reinforcement

At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete. The above requirement may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis based on the loading combinations.

8.10.2 Transverse Deck Slab Reinforcement in T-Girders

At least one-third of the bottom layer of the transverse reinforcement in the deck slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook. If the slab extends beyond the last girder web, such reinforcement shall extend into the slab overhang and shall have an anchorage beyond the exterior face of the girder web not less than that provided by a standard hook.

8.10.3 Lateral Reinforcement of Flexural Members

Compression reinforcement used to increase the strength of flexural members shall be enclosed by ties or stirrups which shall be at least 9mm ø for longitudinal bars that are 30mm ø or smaller and at least 12mm ø for larger size dia and bundled longitudinal bars. The spacing of ties shall not exceed 16 longitudinal bar diameters. Such stirrups or ties shall be provided throughout the distance where the compression reinforcement is required.

8.11 REINFORCEMENT OF COMPRESSION MEMBERS

8.11.1 Maximum and Minimum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall not exceed 0.08 times the gross area Ag of the section.

The minimum area of longitudinal reinforcement shall not be less than 0.01 times the gross area of the section. When the cross section is larger than that required by consideration of loading, a reduced effective area shall be used. The reduced effective area shall not be less than that which would require one percent of longitudinal reinforcement to carry the loading. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement. The minimum size of bars shall be 16mm.

8.11.2 Lateral Reinforcement

In a compression member that has a larger cross section than that required by conditions of loading, the lateral reinforcement requirements may be waived where structural analysis show adequate strength and feasibility of construction.

Spiral reinforcement for compression members shall conform to the following:

Spirals shall consist of evenly spaced continuous bar or wire, with a minimum diameter of 10mm.

The ratio of spiral reinforcement to total volume of core, p_s, shall not be less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}$$

A_g = gross area section

A_c = area concrete

f'_c = specific compressive strength of concrete

where, f_y is the specified yield strength of spiral reinforcement.

All bars spaced more than 150mm apart shall be enclosed by lateral ties which shall be at least 8mm in size.

The spacing of ties shall not exceed the least dimension of the compression members or 30 cm.

Substructure and Foundations for Bridges and Slab Culverts

This chapter discusses the design aspects of substructures and foundations for bridges and slab culverts. Brick masonry and reinforced concrete abutments have been provided for bridges and slab culverts. Brick abutments shall be used at the site where good sub-soil conditions prevail and piling will not be required and possibility of scour and erosion is rare. For brick abutment height will be from 1.0m to 4.0m. Reinforced concrete abutments have been designed for all sub-soil conditions and height will be from 3.0m to 8.0m. Two types of reinforced concrete abutment namely normal full depth abutment and stub abutment have been provided. Brick masonry abutments have been provided with splayed wing walls and reinforced concrete abutments have been provided with return type wing walls. Reinforced concrete abutments have been designed with open foundation and with both cast-in-situ and precast pile.

Loads such as dead load, live load, earth pressure, wind load and longitudinal forces to be considered for the design of substructures and abutments have been described. General description of loading for maximum stresses, checking of stability against sliding, over turning and bearing capacity failure have been discussed. Different methods for determination of bearing capacity of non-cohesive and cohesive soils have been given. Procedure for structural design of footing have been described. Different procedures for calculation of load carrying capacity of driven and cast-in-situ bored piles in cohesive and non-cohesive soils are also given in details. Structural design procedure of different type of piles along with minimum pile size to be used is also discussed. Load testing procedure of pile is also discussed.

CHAPTER 9

Substructures and Foundations for Bridges and Slab Culverts

9.1 GENERAL

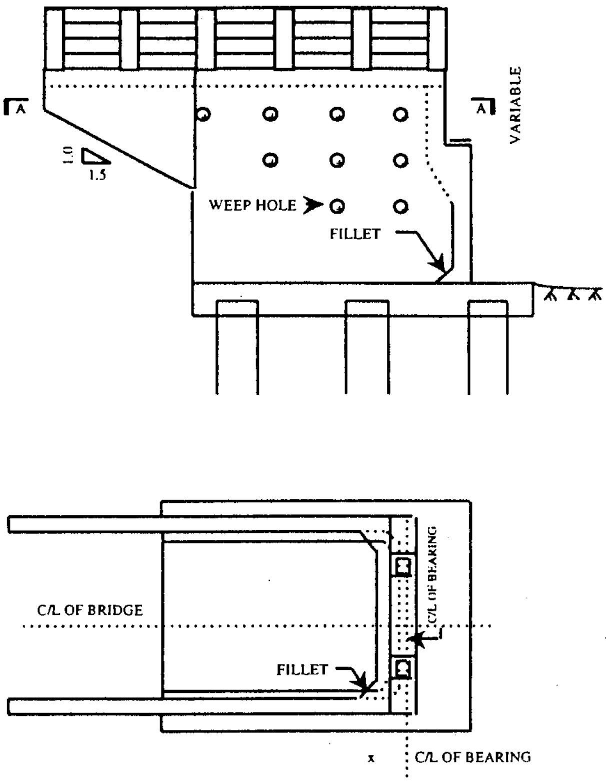
Substructures are that portion of a bridge which lies below the level of bearings and above the foundation. These are necessary to transmit loads from the superstructure to the foundation. Abutments, piers, wing walls, etc. are generally considered as substructures for a bridge structure.

Foundations are that part of a bridge structure below the substructure that transmits loads from the superstructure to the earth. These are footings of abutments, piled foundation, well or caisson foundation, etc.

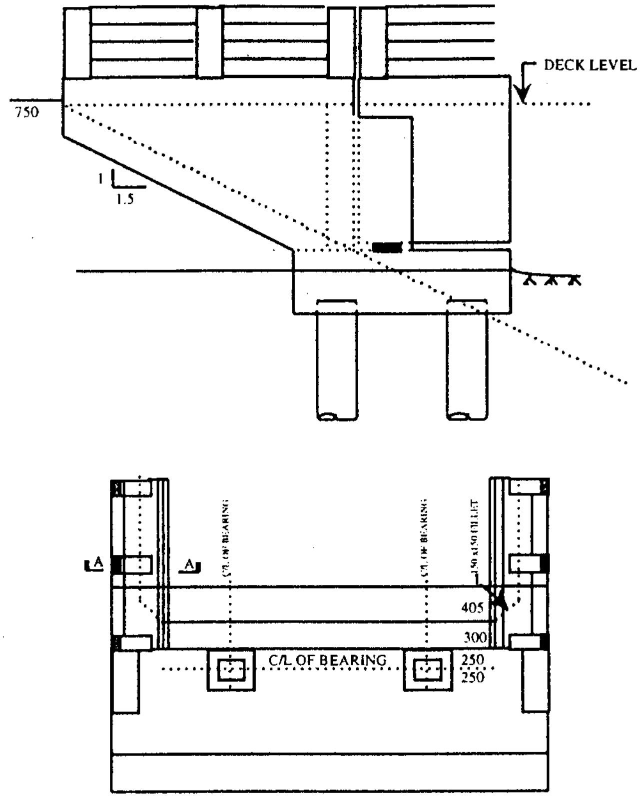
The design of substructures and foundations is an important element of the overall bridge design and affects considerably the aesthetics, the safety and the economy of bridge design. The reinforced concrete girder bridges and slab culverts presented in this RSM are designed with brick masonry and reinforced concrete abutments. Brick abutments have been designed for two subsoil conditions namely clay and sand, and height from 3.0m to 6.0m. Reinforced concrete abutments have been designed for all soil conditions for heights from 2.0m to 8.0m. Three types of reinforced concrete abutment namely open RC abutment, full-depth abutment and stub abutment have been provided in the RSM. These are illustrated in the sketches in Fig. 9.1.

Brick masonry abutments have been provided with splayed wingwalls and reinforced concrete abutments have been provided with return-type wingwalls to retain the earth of the embankment.

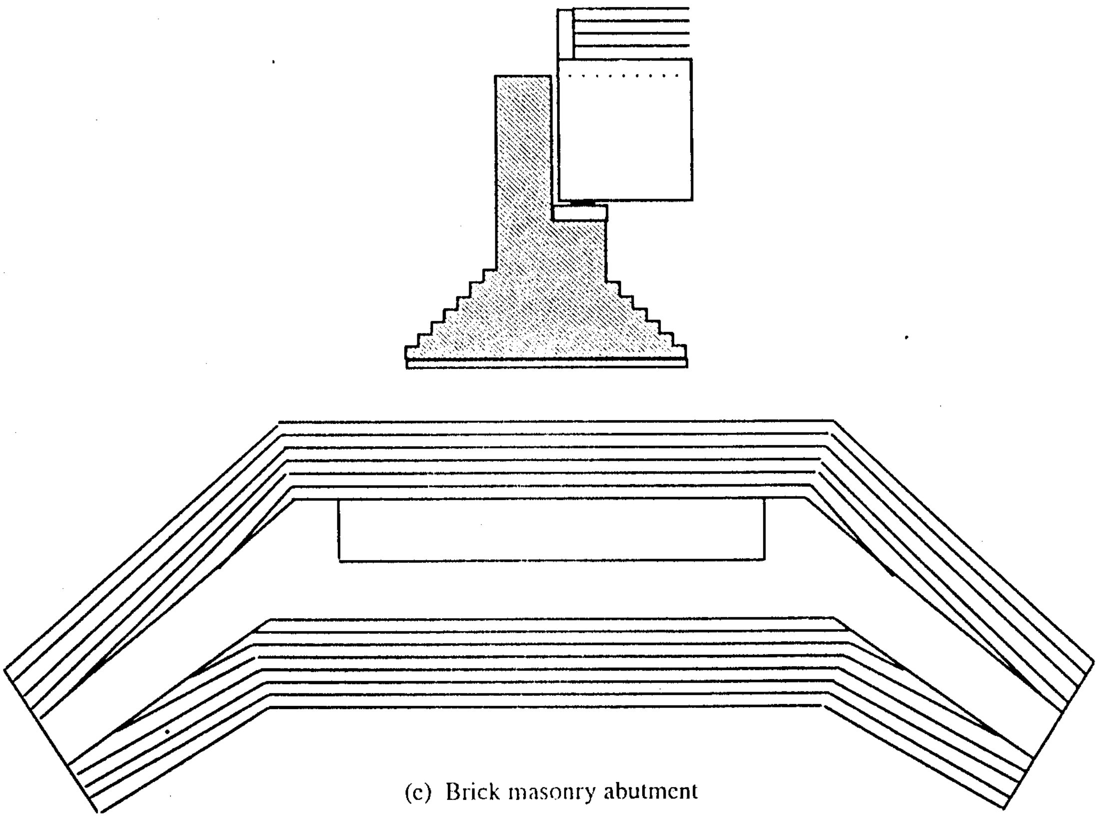
Brick masonry abutments have been designed for open foundations and are recommended for those sites where there is no possibility of scour and erosion near the abutment. Reinforced concrete abutments have been designed with both cast-in-situ and precast pile foundations and also for open foundations.



(a) Normal full-depth abutment



(b) Stub abutment



(c) Brick masonry abutment

Fig. 9.1 Types of Abutments used in the RSM

9.2 LOADS AND FORCES FOR DESIGN

9.2.1 Type of Loads and Forces

The following loads and forces have been considered in the design of substructures and foundations of bridges:

- Dead load;
- Live load;
- Wind load;
- Longitudinal force.

9.2.2 Dead Load

For design of abutments and foundations the dead load shall consist of the weight of superstructure and substructure. The weights as provided in Table 8.1 in Chapter 8 are to be used for computing dead loads.

9.2.3 Live Load

Reactions from AASHTO standard trucks or lane loadings have been used in the design. Structures have been designed using H20, H15 and H10 loadings. No impact has been considered for design of abutments and piles.

9.2.4 Earth Pressure

The earth pressure acting on the abutments or the wingwalls have been determined using the following formula:

in which (Ref. Fig. 9.2)

P_o = Earth pressure for fill soil at rest;

C_o = Coefficient of earth pressure at rest;

γ = Unit weight of fill soil fill;

h = Total height of abutment;

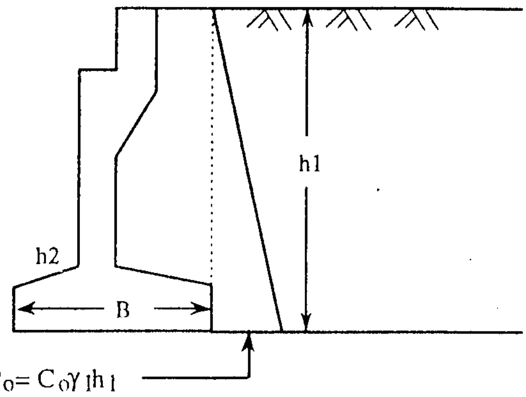


Fig. 9.2 Soil Pressure acting on the

Coefficients of earth pressures at rest is determined by the following formulas:

$$C_o = 1 - \sin\phi$$

where, ϕ is the angle of internal friction of fill soil.

Values of the adopted angles usually adopted for the internal friction of soils are given below:

Sl. No.	Soil type	Unit wt.(Kg/m^3)	ø (degree)
1.	Sand of gravel without fine particles	1760-1920	33-40
2.	Sand or gravel with silt mixture	1920-2080	25-35
3.	Silty sand, sand and gravel with high clay content	1760-1920	23-30
4.	Medium of stiff clay	1600-1920	25-35
5.	Soft clay, silt	1440-1760	20-25

Normally for soil characteristics in Bangladesh the value of ø is taken as 33°.

For the design of abutments without any approach slab a surcharge effect of 0.6m earth on the top of the embankment shall be considered.

9.2.5 Wind Load

Forces transmitted to the substructure by the superstructure and forces applied directly to the substructure by the wind loads shall be assumed to be as follows:

a) **Forces from superstructure**

W (wind load on structure) :

244 Kg per square meter (2394 Pa), transverse;

58.5 Kg per square meter (575 Pa), longitudinal.

Both forces shall be applied simultaneously.

WL (wind load on live load) :

150 kg per linear meter (1460 N/m), transverse; point of action 1.8m above the deck top

60 Kg per linear meter (584 N/m), longitudinal.

Both forces shall be applied simultaneously.

b) **Forces applied directly to the substructure**

An assumed force of 195 Kg per square meter (1915 Pa) shall be applied to the substructure transversely.

9.2.6 Longitudinal Forces

A longitudinal force of 5 percent of the live load without impact shall be applied to the substructure. The center of gravity of the longitudinal force shall be assumed to be 1.8m above the floor slab and to be transmitted to the substructure through the superstructure.

9.3 LOADING FOR MAXIMUM STRESS

Abutments have been designed using working stress design method. They have been designed to withstand safely the following load combinations:

Group 1	:	D+L+E, Allowable stress 100%;
Group 2	:	D+E+W, Allowable stress 125%;
Group 3	:	D+L+E+0.3W+WL+LF, Allowable stress 125%.

where, D = Dead load, L= Live load, E= Earth pressure, W = Wind load, WL = Wind on live load and LF = Longitudinal force.

9.4 CHECKING STABILITY AGAINST SLIDING

Abutments shall be checked for stability against sliding. The factor of safety against sliding is calculated as:

$$F_S = \frac{P_v}{P_h}$$

where, P_h = Total horizontal force at the bottom of footing and Pv = Total resisting force = f x $\sum V$.

In the above formula for P_V, f is the coefficient of friction between concrete and soil and $\sum V$ is the summation of all vertical components of forces acting on the abutment footing.

Value of F_S shall not be less than 1.5.

9.5 CHECKING STABILITY AGAINST OVERTURNING

Abutments with open foundation shall be checked for stability against overturning. The factor of safety against overturning is calculated as:

$$F_o = \frac{M_r}{M_o}$$

where, M_r is the resisting moment against overturning and M_O is the overturning moment. Both resisting and overturning moments are calculated taking moments about the toe of all the forces tending to resist and overturn the abutment. Values of F_O shall not be less than ≥ 2.

9.6 CHECKING STABILITY AGAINST BEARING CAPACITY FAILURE

Abutments with open foundations shall be checked for stability against bearing capacity failure. The distance $\bar{x}$ of the resultant from O (Fig. 9.3) can be computed as:

$$\bar{x} = \frac{M_r - M_o}{\sum V}$$

The eccentricity of the resultant about the vertical centroidal axis of the footing is given by:

$$e = \frac{B}{2} - \bar{x}$$

The value of e should not exceed B/6 in order to avoid tensile stress under the footing. The stresses in the soil at points O and A of the footing can be computed as:

$$f_{max} \text{ (at O)} = \frac{\sum V}{B}\left(1 + \frac{6e}{B}\right)$$

$$f_{min} \text{ (at A)} = \frac{\sum V}{B}\left(1 - \frac{6e}{B}\right)$$

f_{max} shall not exceed the allowable bearing capacity of underlying soil.

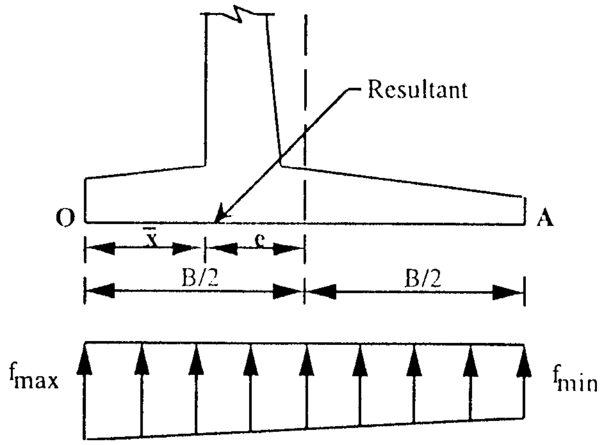


Fig. 9.3 Typical Bearing Pressure Distribution under Footing

9.7 DETERMINATION OF ALLOWABLE BEARING PRESSURE OF SOILS

9.7.1 Non-cohesive Soils (Sand)

The allowable bearing pressure under foundations in non-cohesive soils is governed by the permissible settlement of the structure due to consolidation of the soils under the applied loading.

If standard penetration tests have been performed in boreholes the values of S.P.T. (N) can be used to obtain allowable bearing pressures for various foundation dimensions. In case the results of standard penetration tests are not available the presumed values of allowable soil bearing pressure as presented in Table 9.1 may be used for preliminary design purpose.

The allowable bearing capacity for different non-cohesive soils on the basis of N values are presented in Fig. 9.4. The allowable bearing pressure in this context is that which causes 25mm of settlement under the given breadth of foundation front to back, B, on the assumption that the water table always remains at a depth of at least B below foundation level. For water level higher than this, which is a normal case for abutments in Bangladesh, the allowable pressure should be halved.

Table 9.1 Presumed Bearing Values

Foundation in non-cohesive soil				Foundation in cohesive soil			
Description of Soil	Bearing value, kN/m² Foundation width:			Description of Soil	Bearing value, kN/m² Foundation width:		
	1m	2m	4m		1m	2m	4m
Very dense sands	600	500	400	Hard clays	800	600	400
Dense sand	500	400	300	Very stiff clay	600	400	200
Medium dense sand	250	200	150	Stiff clay	300	200	100
Loose sand	100	75	75	Firm clays	150	100	75
				Soft clay	75	35	0
Note : Bearing values are at a minimum depth of 1m below ground level.							

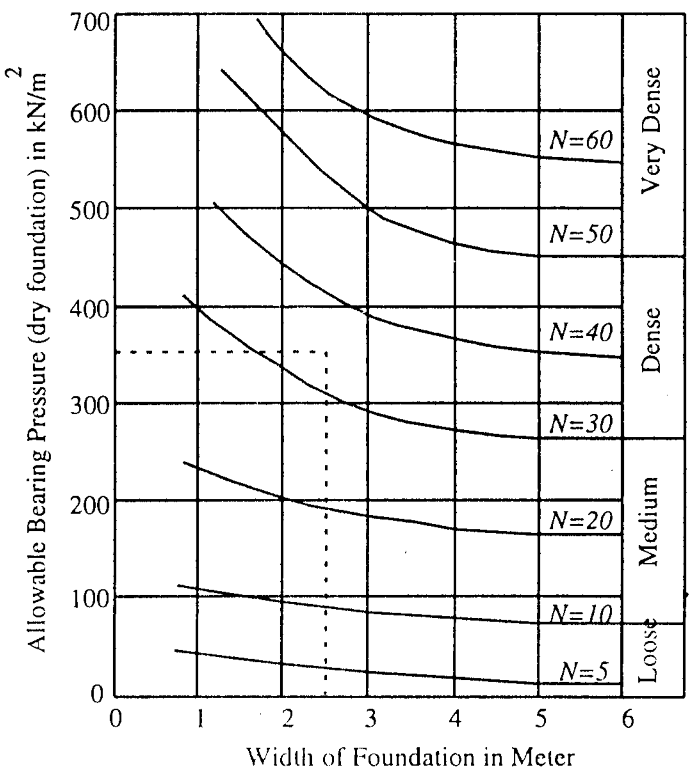


Fig. 9.4 Allowable Bearing Pressure/Penetration Value Relationships

Before applying the relationships of Fig. 9.4, it is necessary to correct the standard penetration values measured in the boreholes, since the tests seriously underestimates the relative densities of cohesionless soils at shallow depths. To allow for this, a correction factor, obtained from Fig. 9.5 should be applied to the measured N values.

Very loose uniformly graded sands with N equal to 5 or less and subject to rapid changes of water level are liable to suffer large settlements under load. In such cases either the sand should be dug out and thoroughly recompacted or the foundation should be supported on piles.

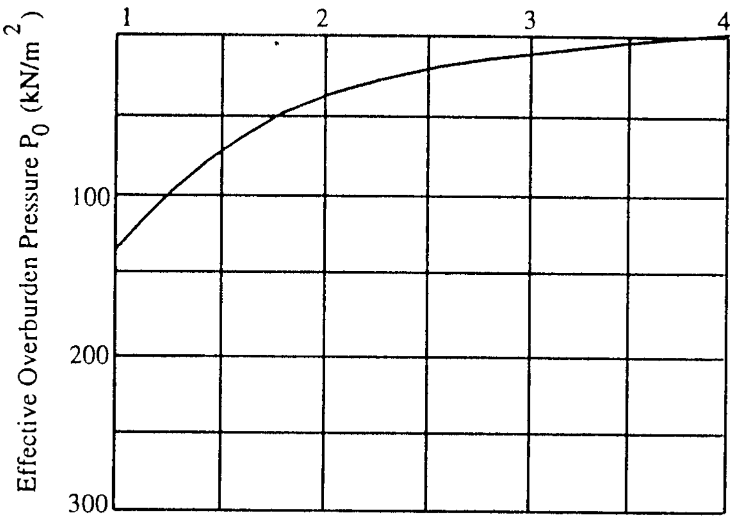


Fig. 9.5 Correction Factors for Cohesionless Soils at Shallow Depths

9.7.2 Cohesive Soils (Clay)

Most cohesive soils at foundation level are saturated and have an angle of shearing resistance equal to zero. The ultimate bearing capacity of such soils can be calculated from the following formula:

Ultimate bearing capacity $q_u = c_u N_c + p$

where, c_u = undrained shear strength in kN/m^2, obtained from field vane shear tests multiplied by the correction factor from Fig. 9.7

N_c = bearing capacity factor

p = total overburden pressure at foundation level in kN/m^2. , $\gamma . D$

γ = density of soil in kN/m^2 above foundation level (submerged density where applicable)

D = depth in meter of foundation below ground surface.

Values of bearing capacity factor N_c for square or circular foundation can be obtained from the graph in Fig. 9.6. For rectangular foundations:

Nc rectangle = $\left(0.84 + 0.16\frac{B}{L}\right).N_c square;$ where B = breadth and L = length of foundation.

Allowable bearing pressure,

$q_a = q_u / 3$, where 3 is the safety factor.

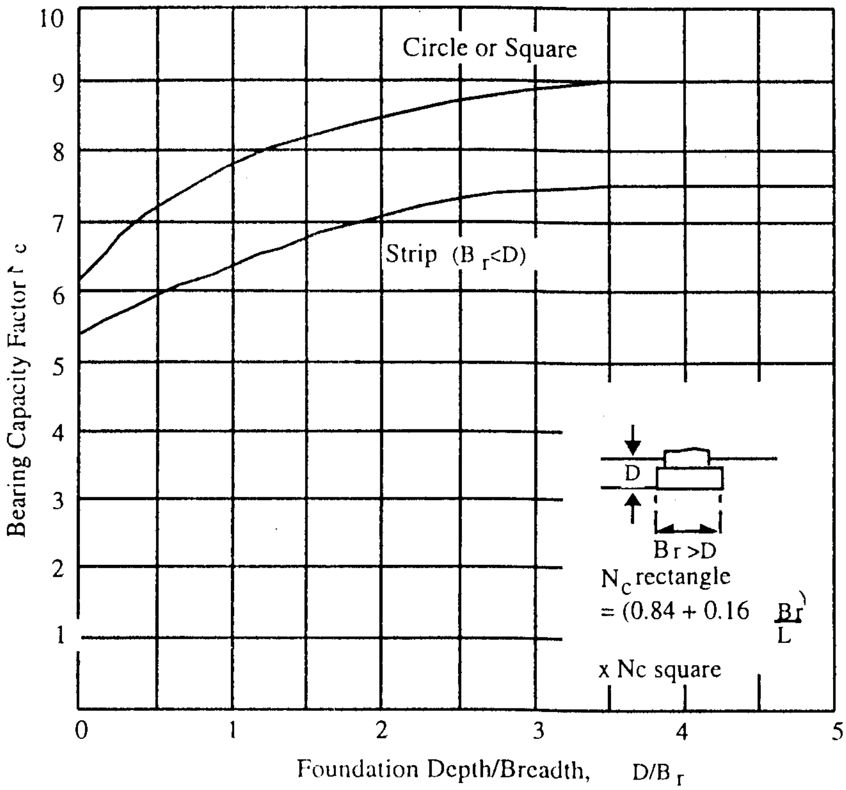


Fig. 9.6 Bearing Capacity Factors

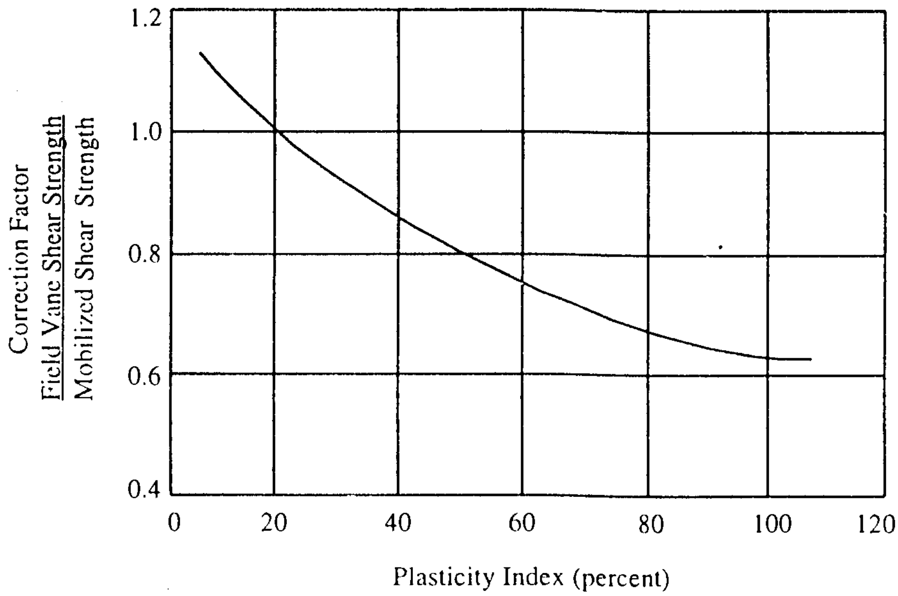


Fig. 9.7 Vane Shear Strength Correction Factor

In absence of actual soil characteristics for calculating the allowable bearing capacity, for purpose of preliminary design the presumed values as given in Table 9.1 may be used.

9.8 ABUTMENTS WITH PILE FOUNDATION

In cases where the pressure on soils beneath the footing of the abutment exceeds the allowable bearing capacity of the soil, pile foundations shall be provided to transmit the loads to the earth. Pile foundations are also required where there is the possibility of heavy scouring and erosion. In the present RSM all reinforced concrete abutments are designed with pile foundations, whereas all brick masonry abutments are designed with open foundation. For designing abutments with piles they are to be positioned beneath the abutment such that the resultant reaction lies in the middle third of the pile cap. Stability of the abutment against sliding need not be checked for the pile foundations, but the number of piles must be such as to resist all horizontal shear forces coming to the pile from the abutment.

9.9 STRUCTURAL DESIGN OF BREAST WALL

For an appropriate loading the moments and shears at different sections of the stem can be computed considering the stem to be cantilevered out from the footing. Using the values of moments and shears, the reinforcement in the stem can be calculated by the usual reinforced concrete design methods.

Except in gravity abutments, not less than 0.80 sq. cm of horizontal reinforcement per 30cm of height shall be provided near exposed surfaces, not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

9.10 STRUCTURAL DESIGN OF FOOTING

9.10.1 Loads and Reactions

Footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces.

Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centres, each pile being assumed to carry the computed portion of the total footing load.

When a footing supports more than one column, or wall, the footing slab shall be designed for the actual conditions of continuity and restraint.

9.10.2 Moments

The external moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of the footing on one side of that vertical plane. The critical section for bending shall be taken at the face of the column, or wall. In case of circular columns, the section shall be taken at the side of the concentric square of equivalent area.

Reinforcement of one-way and two-way square footing shall be distributed uniformly across the entire width of footing.

Reinforcement of two-way rectangular footings shall be distributed uniformly across entire width of footing in the long direction. In the short direction, the portion of the total reinforcement given by following Equation shall be distributed uniformly over a band width (centered on the centreline of column) equal to the length of the short side of the footing. The remainder of reinforcement required in the short direction shall be distributed uniformly outside the centre band width of the footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)}$$

Where, β is the ratio of the footing length to width.

9.10.3 Shear

For shear computation location of the critical section shall be measured from the face of column or wall.

Shear capacity of footings shall be governed by the most severe of the following two conditions:

a) Beam action for the footing, with a critical section extending in a place across the entire width and located at a distance "d" from the face of the concentrated load or reaction area. (d = Thickness of footing).

b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter is a minimum, but not closer than d/2 to the perimeter of the concentrated load or reaction area. (d = Thickness of footing).

For footings on piles shear at the critical section shall be computed in accordance with the following:

- Entire reaction from any pile whose centre is located at a distance equal to half the pile dimension or more outside the critical section shall be considered as producing shear on that section.

- For the intermediate position of pile centres, the portion of the pile reaction to be considered as producing shear on the critical section shall be based on linear interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section. (where d_p = diameter of pile).

9.11 PILES

9.11.1 General

Piles shall be considered when footings cannot be founded on reliable soil stratum within a reasonable depth. Piles may also be used where an unacceptable amount of settlement of spread footings may occur, or where there is possibility of scouring or erosion near the abutment. Considering low bearing capacity and the possibility of scour and erosion, either precast of cast-in-place concrete piles have been recommended for all reinforced concrete abutments in the present RSM.

9.11.2 Pile Penetration

Pile penetration shall be determined based on vertical and lateral local capacities of both the pile and subsurface materials. In general, the design penetration for any pile shall be not less than 10 feet into hard cohesive or dense granular material nor less than 20 feet into soft cohesive or loose granular material. In case of a soft or loose upper stratum overlying a hard or firm stratum piling shall be used to penetrate by a sufficient distance to fix the ends against lateral movement of the pile tip. Driving points or shoes may be necessary to accomplish this penetration.

9.11.3 Pile Types

Piles shall be classified as "friction" or "end bearing" or a combination of both according to the manner in which load transfer is developed.

A pile shall be considered to be a friction pile if the major portion of support capacity is derived from soil resistance mobilized along the side of the embedded pile.

A pile shall be considered to be an end bearing pile if the major portion of support capacity is derived from the resistance of the foundation material on which the pile tip rests.

Under certain soil conditions and for certain pile materials, the bearing capacity of a pile may be considered as the sum of the resistance mobilized on the embedded shaft and that developed at the pile tip, even though these forces that are mobilized simultaneously are not necessarily maximum values.

9.11.4 Selection of Soil Properties

Soil properties defining the strength and compressibility characteristics of the foundation materials shall be obtained by sub-soil investigations as discussed in Chapter 4.

9.11.5 Selection of Design Pile Capacity

The design pile capacity is the maximum load the pile shall support with tolerable movement. In determining the design pile capacity, the following items shall be considered:

- Ultimate geotechnical capacity; and
- Structural capacity of the pile section

9.11.5.1 Ultimate Geotechnical Capacity

The ultimate axial capacity of pile shall be determined from:

$$Q_u = Q_s + Q_T$$

Where, Q_u = ultimate capacity

Q_s = friction capacity

Q_T = end bearing capacity

The allowable design axial capacity shall be determined from :

$$Q_a = Q_u/F_S$$

Where, Q_a = allowable capacity

F_S = factor of safety depending on the construction control and formula used for ultimate bearing capacity calculation.

In determining the design axial capacity, consideration shall be given to :

- The difference between the supporting capacity of single pile and that of a group of piles;
- The possibility of scour and its affect on axial and lateral capacity;
- The effects of negative friction or uplift loads;
- The influence of construction techniques such as auguring or jetting on capacity; and
- The influence of fluctuations in the elevations of the ground water table on capacity.

9.11.5.2 Calculation of Pile Capacity from S.P.T. Value (N) for Non-cohesive Soil (Sand)

The following formula may be used to compute the bearing capacity of pile from standard penetration test value.

Ultimate Pile Capacity, $Q_u = 40NA_p + \dfrac{\overline{N}A_s}{5}$

where, N = average blow count from SPT at pile tip (average from pile tip to twice the diameter below it)

$\overline{N}$ = average blow count from SPT over the length of the shaft

A_p = area of pile tip in square meters

A_s = area of pile shaft embedded in soil in square meters.

The minimum factor of safety to be used with this formula shall be 4.

9.11.5.3 Computation of Pile Capacity from Soil Parameters

(A) Piles in Non-cohesive Soil (Sand)

For a non-cohesive soil (C = 0), the total ultimate pile resistance is given by the expression.

$$Q_u = N_q P A_b + \tfrac{1}{2} K_s P \tan\sigma\, A_s$$

where, P = effective overburden pressure at pile base level

N_q = bearing capacity factor (Fig. 9.8)

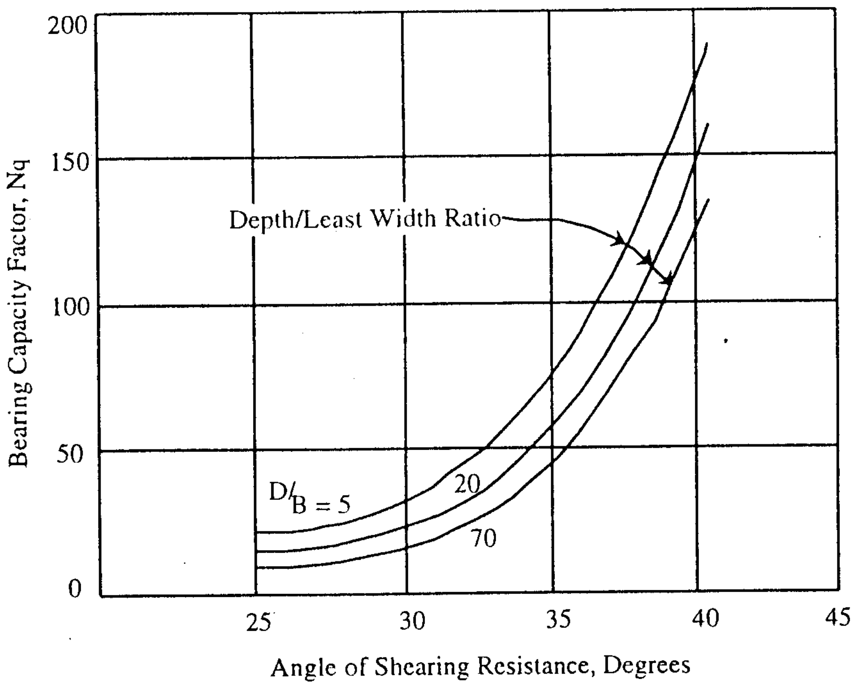


Fig. 9.8 Bearing Capacity Factor of Berezantsev et al

A_b = area of the base of the pile

K_s = coefficient of horizontal soil stress (Table 9.2)

σ = average value of angle of friction between pile and soil (Table 9.3)

A_S = area of shaft of pile.

A safety factor of 3 is recommended for this formula.

Table 9.2
Values of the Coefficient of Horizontal Soil Stress, K_S

Installation Method	K_S/K_O
Driven piles, large displacement	1 to 2
Driven piles, small displacement	0.75 to 1.25
Bored and cast-in-place piles	0.70 to 1.00
Jetted piles	0.50 to 0.70

Typical values of K_O for a normally consolidated sand are :

Relative Density	K_O
Loose	0.50
Medium-dense	0.45
Dense	0.35

Table 9.3
Values of Angle of Pile to Soil Friction for Various Interface Conditions

Pile/Soil Interface Condition	Angle of Pile/Soil Friction, δ
Precast concrete/sand	$0.8\bar{\phi}$ *to* $1.0\bar{\phi}$
Cast-in-place concrete/sand	$1.0\bar{\phi}$

(B) Piles in Cohesive Soils (Clay)

For a cohesive soil the total ultimate bearing capacity of pile is given by the expression:

$$Q_u = N_c C_b A_b + \propto \bar{C}_u A_s$$

where, N_C = bearing capacity, factor, approximately equal to 9

C_b = area of base

$\propto$ = adhesion factor (Fig. 9.9)

$\bar{C}_u$ = average cohesion of the soil surrounding the pile shaft

A_S = area of pile shaft.

A safety factor of 3 is recommended for this formula.

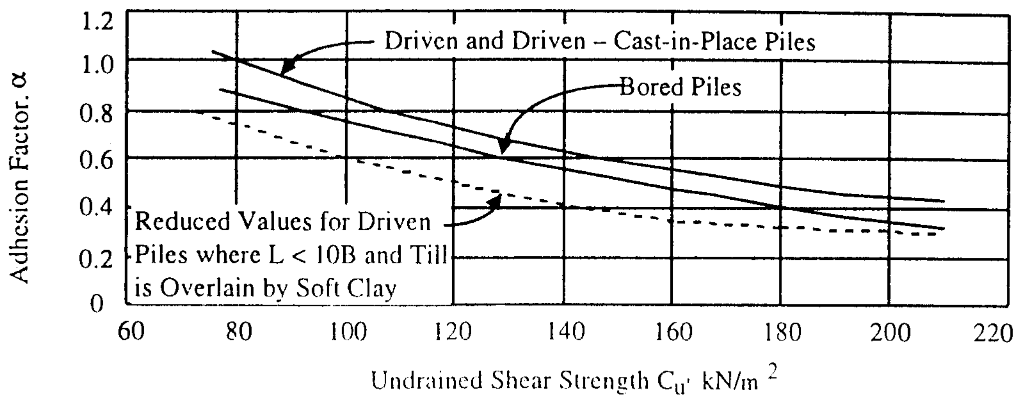


Fig. 9.9 Adhesion Factors for Piles (after Weltman and Healy)

9.11.5.4 Computation of Pile Capacity of Precast Pile based on Dynamic Formula

The most commonly used dynamic formula for calculation of pile capacity is the Engineering News Formula:

$$Q_u = \frac{10E_h}{S_e + c}$$

where, Q_u = ultimate pile capacity, Kg

E_h = energy of hammer, kg.cm

= $W_h.H$

where, W_h = weight of hammer in Kg

H = height of free fall in cm

S_e = final set (penetration) per blow in mm; usually taken as the average penetration for the last 5 blows of a drop hammer

c = empirical constant in mm; 25 mm for drop hammer.

For this formula the allowable bearing capacity shall be determined using a safety factor of 6.

9.11.5.5 Consideration of Group Action Effect in Piles

The supporting capacity of a group of vertically loaded piles can be considerably less than the sum of the capacities of the individual piles comprising the group. The numerical value of the reduction is calculated by the efficiency formula. The following Converse-Labarre Formula is normally used to determine the pile group efficiency factor:

$$E = 1 + \frac{\alpha}{90} \cdot \frac{(n-1)m + (m-1)n}{mn}$$

where, m = number of rows in a group

n = number of piles in a row

α = arc tan(d/s)

s = centre to centre spacing of pile

d = pile diameter.

9.11.6 Lateral Loads on Piles

The design of laterally loaded piles shall be governed by lateral movement criteria.

In lieu of definite information, the following Table gives suggested allowable thrusts for Vertical Piles based on a Safety Factor of 3 applied to the load, required for 6mm deflection

Table 9.4
Suggested Safe Allowable Lateral Forces on Vertical Piles, Kg

Pile Type	Medium Sand	Fine Sand	Medium Clay
Free end concrete piles 400 mm dia	3200	2000	1950
Fixed end concrete piles 400 mm dia	3200	2000	1950
Source: Pile Foundations; Robert D. Chellis; P-221			

Further, concrete piles shall be reinforced to resist bending stresses caused by lateral thrusts.

9.11.7 Structural Design of Piles

9.11.7.1 General Design Requirements

Piles shall be designed to withstand stresses caused during their installation and subsequently when they function as supporting members in a foundation structure.

Pile caps and capping beams shall be designed to transfer loading from the superstructure to the heads of piles, and to withstand pressures from the soil beneath and on the sides of capping members.

9.11.7.2 Design of RC Precast Piles

Precast concrete piles shall have a cross sectional area of not less than 15.2 sq.cm.. In saline area a minimum cross sectional area of 21.7 sq.cm. shall be used. For square sections the corners shall be chamfered at least 2.5cm.

Piles shall preferably be cast with a driving point and for hard driving, preferably shall be shod with a metal shoe.

The requirement of reinforcing the piles to withstand bending stresses caused by lifting shall be considered for precast piles. Precast piles shall be designed considering a lifting point at L/3 from the head (where L = length of pile). The maximum allowable stress in a precast concrete pile shall not exceed $0.33f_c'$ on the gross cross sectional area of concrete. In computing stresses due to handling the static loads shall be increased by 50 percent as an allowance for impact and shock.

The stresses due to driving of the pile shall also be considered. The driving stress shall be assumed to be the ultimate driving resistance divided by the cross sectional area of concrete and this must not exceed the allowable concrete stress.

Vertical reinforcement of precast square concrete piles shall consist of not less than four bars. If more than four bars are required, the number may be reduced to four in the bottom 1200 mm of the pile. The amount of reinforcement shall be at least 1.5 percent of the total cross section.

The full length of vertical steel shall be enclosed with ties. For providing lateral reinforcement in precast concrete piles provisions of BS 8004 may be adopted.

The reinforcement shall be placed at a clear distance from the face of the pile of not less than 50mm and when piles are used in saline zones, this clear distance shall not be less than 75mm.

9.11.7.3 Design of Bored Cast-in-Situ Piles

Cased or uncased cast-in-situ bored piles shall be used depending on soil conditions. These piles shall have a uniform cross section of minimum 15.2 sq.cm.. In saline area a minimum cross sectional area of 21.7 sq.cm. shall be used.

Bored cast-in-situ piles shall be designed as columns. The depth below ground surface to the point of contraflexure shall be taken as 1m in firm ground and 3m in weak ground such as soft clay or silt.

Reinforcement shall be provided in all piles under the abutments of bridges. Detailing of the reinforcement shall be done in accordance with DTU 13.2 (France), where a minimum longitudinal reinforcement has been recommended as 0.5% of cross section and the spacing of helical reinforcement has been limited to 200mm pitch.

The clear cover of reinforcement in the piles shall be as specified for precast concrete piles.

9.11.8 Test Piles

Test piles shall be considered for each substructure unit to determine pile installation characteristics, evaluate pile capacity with depth and to establish contractor pile order lengths. Piles shall be tested by static loading for cast-in-place bored piles or conducting driveability studies for precast piles.

Before manufacturing precast piles a test pile shall be made and driven at a point very close to the site and driveability and load capacity of the pile shall be determined. Final length of precast pile shall be fixed on the basis of such test.

Load test shall also be conducted for working cast-in-situ pile for each foundation unit. Test shall be conducted using a pile constructed in a manner and of dimensions and materials completely identical to the proposed designed pile.

Load tests shall be conducted following prescribed procedures as per ASTM D1143 (Standard Method of Testing Pile under Static Axial Compressive Load) or IS : 2911 (Part 4) - 1985.

9.12 BRICK MASONRY ABUTMENTS

Brick masonry abutments for reinforced concrete girder and slab bridges have been recommend in this Manual for heights up to 6.0m. These abutments are suitable for sites with good bearing capacity of soils and where there is no possibility of scour or erosion of foundation soil. The design of a brick masonry abutment shall consist in assuming preliminary dimensions depending on the type and size of superstructure, checking stability and bearing capacity of underlying soil and also checking stresses in the body of abutment.

The design for stability of brick masonry abutments against sliding, overturning and bearing capacity failure follows the same procedure as described earlier. For checking stresses at base the eccentricity e of the resultant R of all forces about the vertical centroidal axis is to be determined and then extreme stresses at the base are to be calculated from the formula below (Ref. Fig. 9.10).

All calculations are to be done for 1 meter strip of the abutment. These stresses shall always be within the maximum permissible stresses for brick, which are 100 t/m^2 compressive stress and $20 t/m^2$ for tensile stress, in bending.

9.13 WING WALLS

Splayed type brick masonry wing walls have been provided for brick masonry abutments. Stability analysis and the checking of stresses in these wing walls shall follow the same principle as described for design of brick masonry abutments.

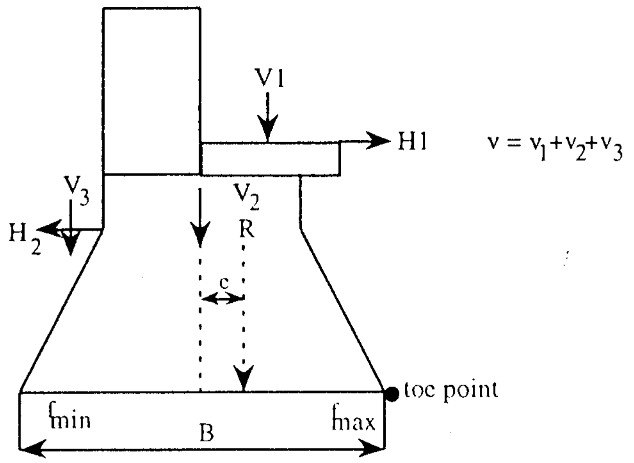


Fig. 9.10 Checking Maximum Stresses at the Base of Masonry Abutment

$$f_{max} \text{ or } f_{min} = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right)$$

Return type cantilever wing walls have been provided for reinforced concrete abutments. Wing walls shall be designed taking into account self weight, horizontal earth pressure and weight of earth above the base. For structural analysis wing walls shall be considered fixed at two sides - at the stem of abutment and at the base of the wall.

9.14 DRAINAGE REQUIREMENT FOR ABUTMENTS

Proper attention should be given to the drainage of water from the embankment behind the abutment. Accumulation of water in the embankment increases the active earth pressure on the abutment resulting in reduced values of factor of safety against sliding and overturning. To facilitate drainage weep holes in the form of 100 mm dia pipes shall be embedded in the stem of abutment and wing wall. These should be spaced at a maximum 3m both horizontally and vertically upto the probable high water level. To prevent clogging at least 0.1 m^3 of crushed stone or best quality brick chips of 40 to 50mm size should be placed at the rear of each weep hole. In addition, a continuous back drain with clean broken stone or bricks mixed with sand should be provided behind the abutment for a length of one meter from the wall.

Pipe Culverts

Use of pipe culverts is recommended as a cross-drainage structure on a road when the discharge through the culvert is small and there is no defined channel. Non pressure light duty type pipes have been recommended for H10 and H15 loadings and for H20 loading non-pressure heavy duty type pipe is recommended. Culverts with single, double, triple and quadruple vents with pipe diameters of 0.30m, 0.60m and 0.9m is provided in the Manual. Bedding of cement concrete over one layer of flat brick soling is recommended for all type of pipe culverts. Brick masonry head wall at a distance 1.0m 1.5m and 2.0m from the toe of the embankment is provided for 0.3m, 0.6m and 0.9m diameter size respectively.

Design of pipe culverts involves two broad discipline (i) hydraulic design to determine number and size of pipes required and (ii) structural design of pipes.

As for hydraulic design, the diameter of pipe and number of vents are so selected that for a given discharge the road embankment is not over-topped nor any property damaged by submergence. Procedure has been set out and formulae are given to calculate the discharge through the pipe.

For structural design of pipes, a wide variety of designs with a different wall thickness and reinforcements have been provided for different loads on a pipe culvert depend on the height of earth fill and amount of concentrated wheel load. Standard design of pipes for various live loads and heights of fill have been prepared and presented in Tables 10.5 to 10.7.

Pipe sections are usually cast in short length for ease of construction, transportation and placement. Three methods of jointing of pipes have been described. For simplicity of construction the concrete collar joint is recommended.

CHAPTER 10

Pipe Culverts

10.1 GENERAL

Pipe culverts may be used as a cross-drainage structure on a road, when the discharge through the culvert is small and when there is no defined channel. Pipes may be non-pressure type and pressure type. They also may be of light duty and heavy duty. Non-pressure type, both light duty (for H10 loading) and heavy duty (for H15 & H20 loading) pipes are recommended in this Manual for use in LGED roads.

Three sizes of pipe diameter are included in this Manual. They are 0.3m, 0.6m and 0.9m. Using these diameters single, double, triple and quadruple vent pipe culverts are provided in the Manual.

A typical design of a double vent pipe culvert is given in Fig. 10.1. Normally pipes have a length of 2.44m. If the overall length is not a multiple of the standard length, special pipe lengths should be ordered from the manufacturer. Such special pipes should be placed at the center.

10.2 BEDDING FOR PIPES

Bedding is required to distribute the vertical reaction around the lower exterior surface of the pipe and reduce stress concentrations within the pipe wall. For use in LGED roads bedding of cement concrete over one layer first class flat brick soling is recommended for all types of pipe culverts.

10.3 HEAD WALLS

Brick masonry head walls at a distance of 1.0m, 1.5m and 2.0m from the toe of embankment have been provided on both ends of pipes for 0.3m, 0.6m and 0.9m diameter sizes respectively.

10.4 DESIGN OF PIPE CULVERT

10.4.1 General

The design of a pipe culvert involves the following:

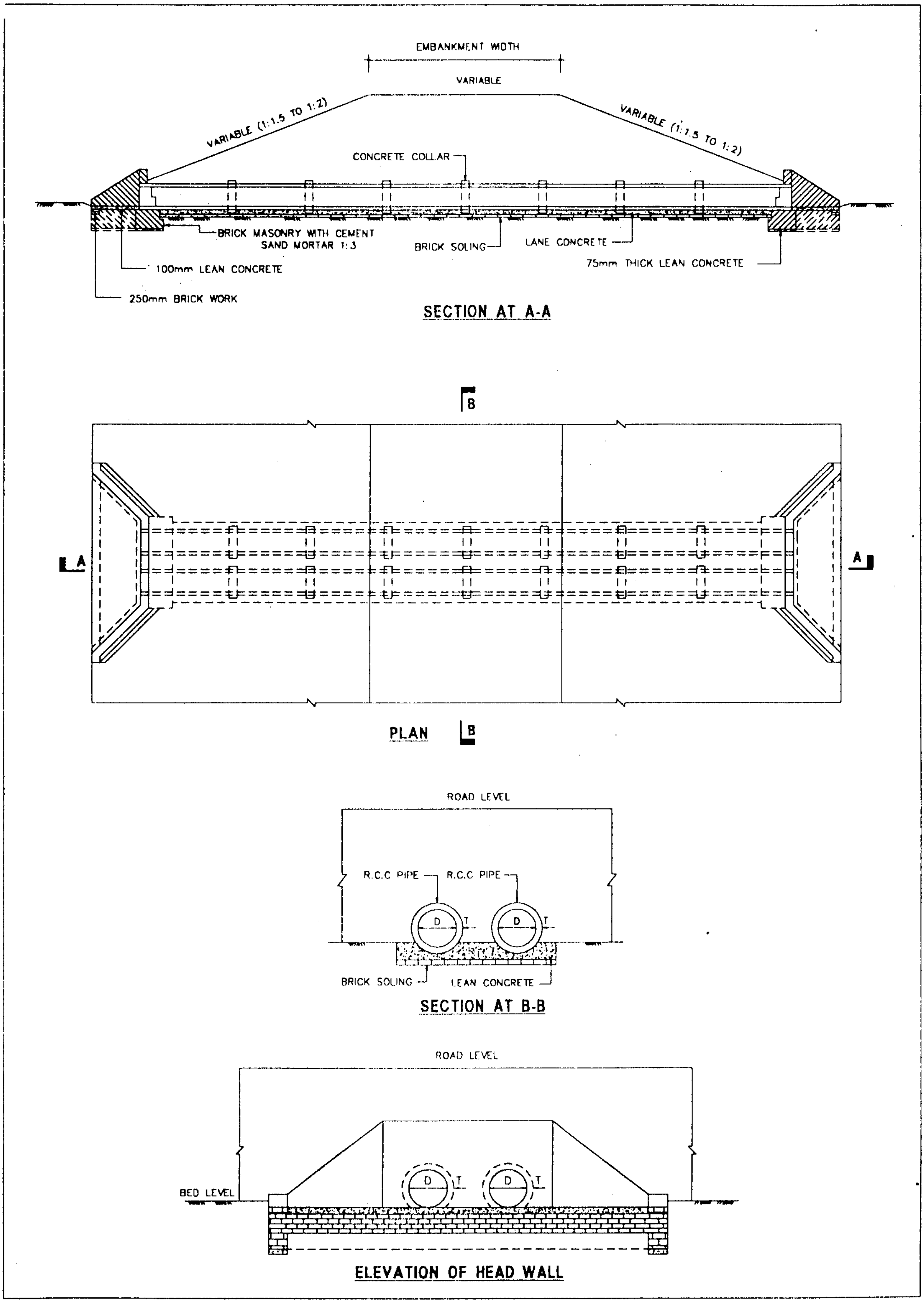


Fig. 10.1 A Typical Pipe Culvert

a) Hydraulic design of the ventway requirement i.e., to determine the number and size of pipes required to pass the discharge.

b) Structural design of the pipe.

10.4.2. Hydraulic Design

The size and number of pipes required for a given site depend on a number of factors such as volume of water to be discharged through the pipe, conditions of entry at the inlet, frictional resistance of inside surface of pipe, length of pipe, hydraulic mean radius of pipe and operating head of water. For design of pipe size, pipe culverts are assumed to flow full. Heading up of water on the upstream side is required to create a head to cause flow through the pipe. The diameter of pipe and number of vents are so selected that for a given discharge the heading up of water does not go higher than a predetermined safe level, the criteria for safety being that the road embankment is not overtopped, nor any property damaged by submergence. It is also essential that the high flood level of water on the outfall side near the exit of culvert is predetermined. The difference of the levels of water at inlet and exit is called the operating head. This head is required to :

Overcome the frictional resistance of the wetted surface of the culvert.

Supply the energy required to generate velocity of flow.

Force water through the inlet of the culvert.

The discharge (m^3/sec) through the pipe flowing full is given by the equation:

$$Q = AK\sqrt{2gH}$$

where,
H = operating head in meters
g = acceleration due to gravity in m/sec^2 = 9.81 m/s^2
K = a constant, called conveyance factor. K is a function of length L of the culvert, its mean hydraulic radius R and the coefficient of roughness n of the surface of the pipe.
A = area of the pipe.

The conveyance factor K is determined by the formula:

$$K = \frac{1}{\sqrt{1 + K_e + K_f}}$$

where,
K_e = head lost at entry; K_e = 0.08 for bell mouthed entry and 0.51 for sharp edged entry.
K_f = head lost due to friction of pipe

$= 0.0033\ L/®^{1.3}$ for concrete pipe

L = length of pipe in meters

R = hydraulic mean radius in meters.

Using the formulas above for any given set of conditions the size and number of pipes required can be determined by trial.

10.4.3 Structural Design of Pipe

10.4.3.1 General Formula

The supporting strength of a pipe is the vertical field load per meter run on the pipe as laid, which will cause its failure. This is given by the standard three-edge bearing strength test as below :

Supporting strength of pipe = Strength factor x three-edge bearing strength.

The strength factor depends on the type of loading and the conditions of bedding of the pipe. Loading can be two types – (1) earth filling and (2) surface live loads. The strength factor for earth filling may be taken approximately as 2.3 for first class bedding with selected sand. For surface live loads the strength factor is taken as 1.5.

The type of non pressure pipe for a given bedding condition (first class sand bed in this case) should be chosen so that under the worst combination of field loading a factor of safety of 2.0 is available as in the following equation :

$$\frac{\text{Three} - \text{edge bearing strength in kN / m run}}{\text{Factor of safety } (= 2.0)} =$$

$$\frac{W_E \text{ due to filling material } (\text{kN / m})}{\text{Related strength factor } (= 2.3)} + \frac{W_L \text{ due to live load } (\text{kN / m})}{\text{Related strength factor } (= 1.5)}$$

10.4.3.2 Determination of Load Due to Embankment Filling

The load on a pipe due to embankment material can be calculated from equation

$$W_E = C_c.w.D^2$$

where, W_E = vertical external load in kg/m on pipe due to embankment material.

C_c = coefficient depending on the ratio of height of embankment h to external diameter of pipe and condition of loading

w = density of embankment material in ton/m^3

D = external diameter of pipe in meter.

For normal conditions, the vertical load due to earthfill may be obtained from Table 10.1 which is based on a value of 1920 kg/m³ for density of fill.

Table 10.1 Load on Pipe due to Earthfill

Internal Diameter of Pipe (mm)	Loading in Ton/m for Various Depths 'h' in Meter of Earth Fill							
	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00
300	1.3	2.6	4.0	5.4	6.8	8.1	-	-
600	2.0	4.5	6.6	8.0	11.0	14.2	17.1	19.8
1000	3.0	6.3	10.8	14.1	17.4	21.5	24.3	27.6

10.4.3.3 *Determination of Live Load*

Live load considerations are necessary in the design of pipe culverts installed with shallow earth fill upto 3.0m. The effect of live load decreases with an increase of depth of earthfill and becomes negligible with depth greater than 3.0m. Highway live load from wheel of standard trucks transmitted to pipe can be computed from the following equation :

$$W_L = 4C_S IP$$

where, W_L = vertical external load in kN/m due to concentrated surface load

C_S = influence coefficient depending on diameter of pipe and height of fill above the pipe from Table 10.2

I = impact factor = 1.3

P = concentrated wheel load in kN
= 71 kN for H-20 loading
= 53 kN for H-15 loading
= 36 kN for H-10 loading.

Influence Coefficient C_S for various diameters and depth of filling is presented in Table 10.2 below :

Table 10.2 Influence Coefficient C_S for Live Load Determination

Pipe Dia 'd' (mm)	Influence Coefficient CS for Depth of Fill 'h' in m				
	0.60	0.80	1.00	2.00	3.00
300	0.080	0.055	0.040	0.013	0.006
600	0.131	0.094	0.068	0.022	0.010
1000	0.162	0.123	0.095	0.032	0.015

10.4.3.4 Selection of Pipe Section

Standard designs of reinforced concrete pipe culverts have been presented in this Manual with three diameters of 300mm, 600 mm and 1000mm. Considering that loads on a pipe culvert depend on the height of earthfill and amount of concentrated wheel load, for each diameter a wide variety of designs with different wall thicknesses and reinforcements have been provided. These typical designs are presented in Table 10.4.

The supporting strengths of a pipe in Table 10.4 have been presented in terms of so-called d-loads for each pipe diameters. The three-edge bearing capacity in kN per meter run of pipe is the product of d-load by inside diameter of the pipe.

The required d-load capacity of a circular pipe on first class sand bedding with fill load W_E and live load W_L can be computed as

$$\text{d-load} = \left(\frac{W_L}{1.5} + \frac{W_E}{2.3}\right)\frac{2}{d'}, \text{ where } d' \text{ is the internal dia of pipe in meter.}$$

A safety factor of 2 has been considered for the above formula.

The ultimate three edge bearing strengths of various pipe culverts with different sizes and reinforcements are presented in Table 10.4. For any pipe dia (300mm, 600mm & 1000mm) for a given height of fill and live load, the required d-load capacity can be calculated using the formula above and then from Table 10.4 the required thickness and reinforcement can be obtained.

Following the procedure above standard design of pipes for various live loads and heights of fill have been prepared and presented in Table 10.5 to 10.7.

Table 10.4 Standard Designs of Pipes and their Ultimate d-load Capacity in kN/m per Meter of Internal Diameter

Pipe Diameter = 300 mm (Internal)			
Ring Reinforcement	Wall Thickness, mm		
	50	75	100
6 ø @ 75 c/c	145.2	235.2	271.0
6 ø @ 100 c/c	115.5	182.2	252.6
6 ø @ 125 c/c	95.6	150.4	203.9
6 ø @ 150 c/c	81.1	127.5	172.9
6 ø @ 175 c/c	70.3	110.4	149.2
6 ø @ 200 c/c	63.2	97.1	129.3
10 ø @ 75 c/c	-	-	621.2
10 ø @ 100 c/c	-	349.4	500.4
10 ø @ 125 c/c	-	295.9	414.3
10 ø @ 150 c/c	155.5	254.4	355.1
10 ø @ 175 c/c	138.3	224.0	310.3
10 ø @ 200 c/c	125.1	199.7	274.6
Longitudinal Reinforcement	6 ø @ 250c/c	6 ø @ 150c/c	6 ø @ 125c/c

Pipe Diameter = 600 mm (Internal)					
Ring Reinforcement	Wall Thickness, mm				
	50	75	100	125	150
6 ø @ 75 c/c	36.3	58.8	81.0	104.3	127.4
6 ø @ 100 c/c	28.9	45.6	63.2	79.6	97.2
6 ø @ 125 c/c	23.9	37.6	51.0	64.5	77.3
6 ø @ 150 c/c	20.3	31.9	43.2	53.7	65.9
6 ø @ 175 c/c	17.6	27.6	37.3	47.4	-
6 ø @ 200 c/c	15.8	24.3	32.3	41.1	-
10 ø @ 75 c/c	-	-	155.3	205.5	254.4
10 ø @ 100 c/c	-	87.3	125.1	161.9	199.8
10 ø @ 125 c/c	-	74.0	103.6	133.4	162.5
10 ø @ 150 c/c	38.9	63.6	88.8	112.8	137.9
10 ø @ 175 c/c	34.6	56.0	77.6	98.6	118.9
10 ø @ 200 c/c	31.3	49.9	68.6	87.0	106.0
Longitudinal Reinforcement	6 ø @ 250c/c	6 ø @ 150c/c	6 ø @ 125c/c	6 ø @ 100c/c	6 ø @ 75c/c

Pipe Diameter = 1000 mm (Internal)					
Ring Reinforcement	Wall Thickness, mm				
	50	75	100	125	150
6 ø @ 75 c/c	13.8	22.3	30.7	39.6	48.4
6 ø @ 100 c/c	10.9	17.3	23.9	30.2	36.9
6 ø @ 125 c/c	9.1	14.3	19.3	24.5	29.4
6 ø @ 150 c/c	7.7	12.1	16.4	20.3	25.0
6 ø @ 175 c/c	6.7	10.5	14.2	18.0	-
6 ø @ 200 c/c	6.0	9.2	12.3	15.5	-
10 ø @ 75 c/c	-	-	58.9	78.0	96.6
10 ø @ 100 c/c	-	33.1	47.5	61.5	75.8
10 ø @ 125 c/c	-	28.1	39.3	50.5	61.6
10 ø @ 150 c/c	14.8	24.2	33.7	42.8	52.3
10 ø @ 175 c/c	13.2	21.3	29.5	37.4	45.1
10 ø @ 200 c/c	11.9	18.9	26.0	33.0	40.2
Longitudinal Reinforcement	6 ø @ 250c/c	6 ø @ 150c/c	6 ø @ 125c/c	6 ø @ 100c/c	6 ø @ 75c/c

Table 10.5 Standard Designs of Reinforced Concrete Pipes for H20 Loading

Internal Diameter (mm)	Height of Fill h (m)	Live Load p (kN)	Thickness t (mm)	Ring Reinforcement		Longitudinal Reinforcement		d-Load Capacity Required (kN/m/m)
				Dia (mm)	Spacing (mm)	Dia (mm)	Spacing (mm)	
300	1	71	75	6	175	6	150	102.52
	2	71	50	6	125	6	250	95.27
	3	71	75	6	150	6	150	123.58
	4	71	75	6	125	6	150	153.53
	5	71	75	6	200	6	150	193.65
	6	71	75	6	75	6	150	230.30
600	1	71	125	10	200	6	100	84.23
	2	71	125	10	200	6	100	82.04
	3	71	125	6	75	6	100	102.04
	4	71	125	10	150	6	100	113.74
	5	71	100	10	75	6	125	156.39
	6	71	125	10	75	6	100	201.88
	7	71	150	10	75	6	75	243.12
1000	1	71	150	10	100	6	75	72.34
	2	71	150	10	100	6	75	69.49
	3	71	150	10	75	6	75	99.51

Table 10.6 Standard Designs of Reinforced Concrete Pipes for H15 Loading

Internal Diameter (mm)	Height of Fill h (m)	Live Load p (kN)	Thickness t (mm)	Ring Reinforcement		Longitudinal Reinforcement		d-Load Capacity Required (kN/m/m)
				Dia (mm)	Spacing (mm)	Dia (mm)	Spacing (mm)	
300	1	53	50	6	150	6	250	85.95
	2	53	50	6	125	6	250	89.86
	3	53	50	10	200	6	250	121.10
	4	53	75	6	125	6	150	153.54
	5	53	75	10	200	6	150	193.65
	6	53	75	10	175	6	150	230.32
600	1	53	100	10	200	6	125	70.08
	2	53	100	10	175	6	125	77.46
	3	53	125	10	175	6	100	99.96
	4	53	125	10	150	6	100	113.74
	5	53	150	10	125	6	75	156.39
	6	53	150	10	100	6	75	201.88
	7	53	150	10	75	6	75	243.12
	8	53	150	12	75	6	75	281.50
1000	1	53	100	10	75	6	125	60.50
	2	53	125	10	100	6	100	65.50
	3	53	150	10	75	6	75	97.64

Table 10.7 Standard Designs of Reinforced Concrete Pipes for H10 Loading

Internal Diameter (mm)	Height of Fill h (m)	Live Load p (kN)	Thickness t (mm)	Ring Reinforcement Dia (mm)	Ring Reinforcement Spacing (mm)	Longitudinal Reinforcement Dia (mm)	Longitudinal Reinforcement Spacing (mm)	d-Load Capacity Required (kN/m/m)
300	1	36	50	6	175	6	250	70.23
	2	36	50	6	150	6	250	84.76
	3	36	50	6	100	6	250	118.73
	4	36	75	6	125	6	150	153.54
	5	36	75	10	200	6	150	193.65
	6	36	75	6	75	6	150	230.32
600	1	36	75	6	75	6	150	56.72
	2	36	75	10	175	6	125	73.14
	3	36	125	10	175	6	100	98.00
	4	36	125	10	125	6	100	113.74
	5	36	125	10	100	6	100	156.39
	6	36	125	10	75	6	100	201.88
	7	36	150	10	75	6	75	243.12
	8	36	150	12	75	6	75	281.51
1000	1	36	125	10	125	6	100	49.30
	2	36	150	10	125	6	75	61.73
	3	36	150	10	75	6	75	95.87

10.4.3.5 Pipe Joints

Pipe sections are usually cast in short lengths of 2.44m to 3.0m for ease of construction, transportation and placement. When laid end to end on prepared beddings at the site the sections need to be jointed together. There are three methods of jointing. These are :

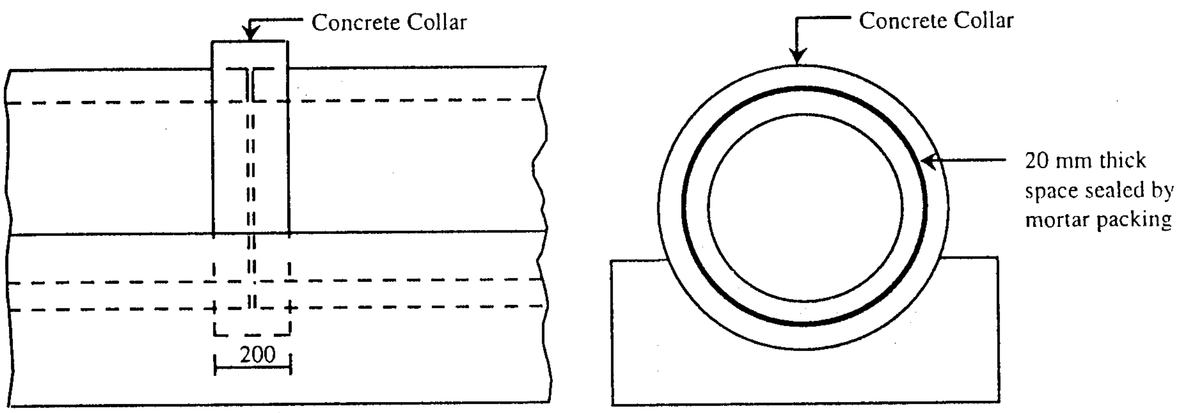
1) Concrete collar joint;

2) Spigot and socket joint; and

3) Tongue and groove joint.

For the concrete collar joint the collar is placed in position along with the pipes and sealed by means of mortar packing. When a better sealing of the joint is desired, the spigot and socket joint may be used. In this method one end of the pipe is formed into a socket using a special mould for casting and the joint after placing of the pipes is sealed by mortar packing. For higher thicknesses of pipe the tongue and groove joint is a better solution. Each segment of pipe is cast with a 'tongue' formed at one end and a 'groove' at the other. The joint is sealed by a mortar or mastic packing.

For simplicity of construction the concrete collar joint is recommended for the rural roads of LGED. The joint is shown in Fig. 10.2. The collar shall have a width of 200mm and a thickness equal to the thickness of the pipe itself. The internal diameter of collar shall be such that there will be a cauking space of 20mm in between the main pipe and the collar. This space shall be sealed by mortar packing (1:4). The reinforcement in the collar shall be the same as that in the main pipe.



Note : 1. Thickness of collar and main pipe shall be equal
2. Collar shall have the same reinforcement as in the main pipe

Fig. 10.2 Details of Concrete Collar Joint

Box Culverts

In this Manual standard box culvert spans vary from 1.0m to 4.3m with single, double and triple cells. The heights of the boxes also vary from 1.0m to 4.30m. For H10 loading carriage way width of 2.44m is considered while for H15 and H20 loadings, carriage way width of 3.66m is considered.

Traffic load distribution has been adopted according to the AASHTO requirements. Loads from wearing course, deck slab, vertical wall and lateral earth pressure and pressure from contained water have been considered.

Structural analysis of the barrel of a box culvert has been performed considering the box section as a monolithic rigid frame. Maximum bearing pressure on the soil underneath the culvert has been calculated and required bearing capacity of the soil is recommended. Wing wall have been analysed considering the end part as cantilever and other part as fixed on two sides. Structural design has been performed in accordance with the AASHTO requirements.

CHAPTER 11

Box Culverts

11.1 GENERAL

Reinforced concrete box culverts are rigid frame structures with square or rectangular openings with spans varying from 1.0m to about 5.0m. The height of the box may also vary from 1.0m to about 5.0m. For economy of design the span to height ratio is usually kept in the range from 1:1 to 1:1.5. The height to span ratio may also vary from 1:1 to 1:1.5. Box culverts may have single cell or multiple cells.

Reinforced concrete box culverts are suitable for sites with comparatively small water way requirement upto about 15m with low to medium height from deck level to bed level. Provision of monolithic construction of the whole structure and continuity of the walls and slabs give economical solution for the structures, where they are suitable. The continuity, however, makes the structure statically indeterminate and to some extent difficult for analysis. The difficulties can however, be overcome by use of computer programme for analysis. Plan, elevation and section of a typical box culvert are presented in Fig. 11.1.

Box culverts consist of the main box or boxes and wingwalls. Main boxes are provided with required length perpendicular to the road to accommodate the carriageway, footpaths and railings on the top slab. The top slab is supported by the vertical walls which come out of the base slab. The wing walls are provided to retain the earth of the embankment on both sides. Additional protective works against erosion are sometimes provided upstream and downstream of the culvert.

All box culverts designed and provided in this Manual assume that no earth fill would be placed on the top slab.

11.2 SIZES AND DIMENSIONS

Standard designs of box culverts with various sizes and dimensions are provided in this Manual in Part B. The box spans vary from 1.0m to 5m with single, double and triple cells. The heights of boxes also vary from 1.0m to 5m. The span to height and height to span ratios have been adopted in the range from 1:1 to 1:1.5 Two widths of carriageway have been considered- 3.66m for HI5 and H20 AASHTO Live load and 2.44m for H10 load.

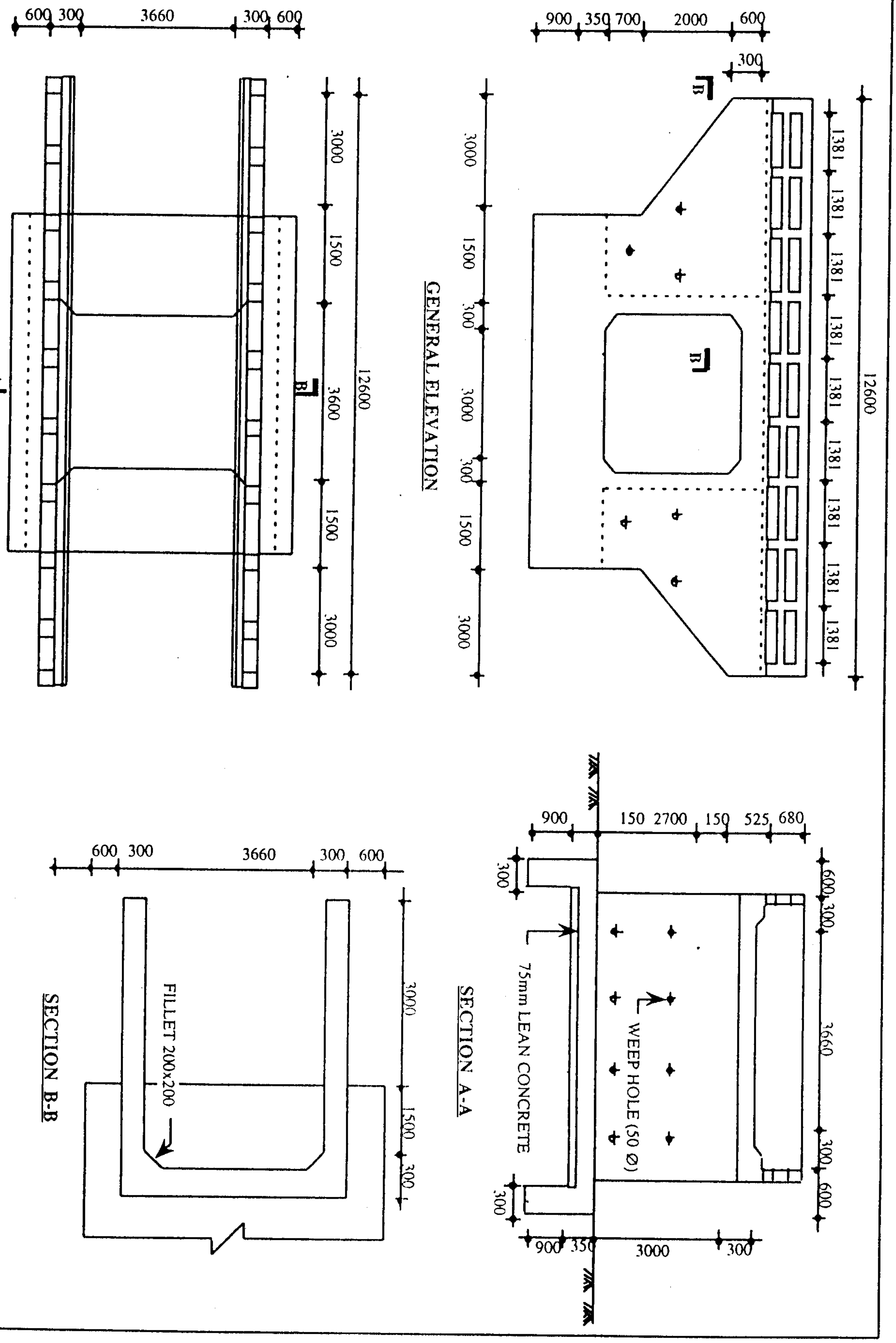


Fig. 11.1 A Typical Box Culvert

Return type wing walls have been designed for box culverts having height above 2.5m. For box culverts with height of wall 2.5m and less, splayed type wing walls have been provided.

11.3 LOADS AND FORCES FOR DESIGN

The following loads and forces shall be considered for design of reinforced concrete box culverts:

1) Concentrated Vertical Load from Wheel

Concentrated vertical load from wheel may be computed as

$$W = \frac{PI}{E}$$

where, W = Wheel load in kN per meter width of slab

I = Impact factor

P = Wheel load in kN

E = Effective width of dispersion of load

The reaction at the foundation from this load is taken to be uniformly distributed over the entire area of the bottom slab.

For distribution of wheel loads to the top slab of the culvert the distribution width in feet 'E' shall be (4+0.06S) but shall not exceed 7.0 ft or 2.13m; where S is the effective span length in feet. The width of top slab strip used for distribution of concentrated wheel loads may be increased by twice the box height and used for distribution of loads to bottom slab.

2) Uniform Vertical Load

The live load from a standard truck and the weight of the wearing coat and deck slab are to be taken as uniform vertical load. The foundation reaction from this load is also taken to be uniform. The lane load is to be considered in lieu of the concentrated wheel load.

3) Weight of Walls

The weights of the all side walls are assumed to cause uniform reaction at foundation.

4) Lateral Earth Pressure

The earth pressure computed according to Coulomb's theory as described in Chapter 9 is to be applied to both the end walls at the box culvert.

5) Pressure from Contained Water

The box is assumed to be full with the water level at the top of the opening. A triangular distribution of water pressure is to be considered. No excess hydrostatic head has been considered for design.

The loading cases are shown in Fig. 11.2.

Three loading conditions have been considered as critical for design of box culverts. These are :

a) Culvert empty with full live load

The loads to be considered in this case are : (1) weight of top slab, (2) weight of walls, (3) wheel or lane load on top slab, and (4) maximum earth pressure on walls

b) Culvert empty with no live load

The loads to be considered in this case are : (1) weight of top slab, (2) weight of walls, and (3) maximum earth pressure on walls.

c) Culvert full with full live load

The loads to be considered in this case are : (1) weight of top slab, (2) weight of walls, (3) wheel or lane load on top slab, (4) minimum earth pressure on walls and maximum horizontal pressure from water in the culvert.

It has been found that in most cases of box sizes adopted in this Manual loading condition as in (a) above governs.

11.4 STRUCTURAL ANALYSIS

Structural analysis of the barrels of a box culvert has been performed considering the box section as a monolithic rigid frame structure. Distances to the geometric centers of supports/walls have been taken as the span lengths for individual members of the frame. Fillets have not been considered in the design. The analysis has been performed with a computer programme using the moment distribution method of structural analysis. As a result of the analysis moment and shear at pertinent locations were found and used for the structural design of sections. The effect of axial forces in the sections was found to be negligible and therefore not considered in the design of sections. Maximum bearing pressure on the soils underneath the culverts was also calculated and required bearing capacity of soil has been recommended. Wing walls have been analysed considering the end part as cantilever and the other part as fixed on two sides.

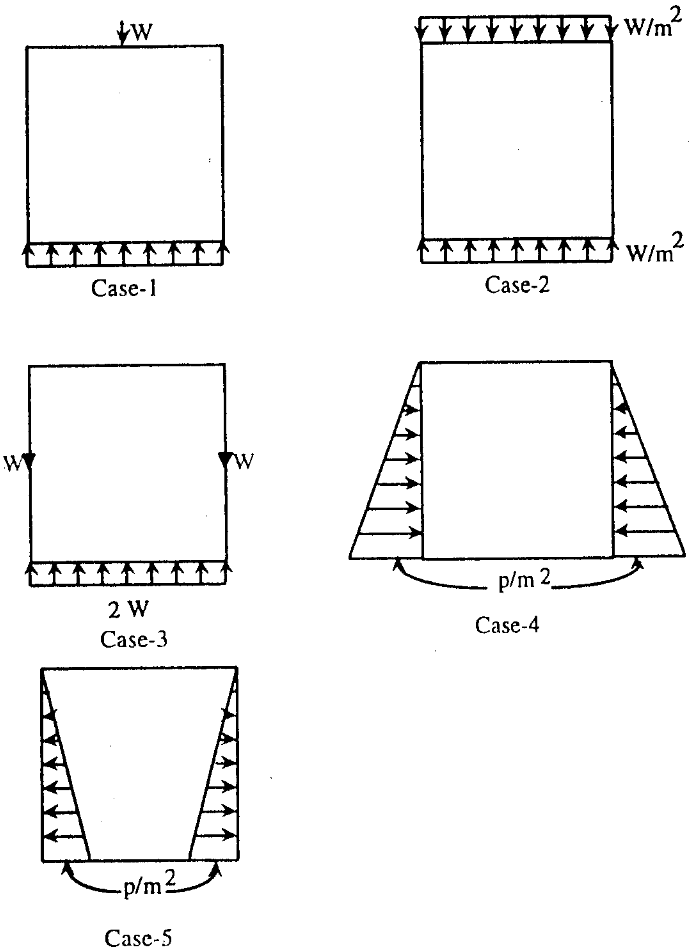


Fig. 11.2 Loading Cases for Box Culverts

11.5 STRUCTURAL DESIGN

Structural design of box culvert has been performed following AASHTO specifications and using the moments and shears at pertinent locations. The following stipulations of AASHTO have been considered during design :

1) Minimum reinforcement shall be provided at all cross-sections subject to flexural tension, including the inside face of walls in accordance with the following :

   - At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete. This requirement may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis based on the AASHTO loading combinations.

2) Shrinkage and expansion reinforcement shall be provided near the inside surfaces of walls and slabs in accordance with the following :

- Reinforcement for shrinkage and expansion stresses shall be provided near the exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be atleast 1/8 square inch per foot in each direction. The spacing of shrinkage and temperature reinforcement shall not exceed three times the wall or slab thickness, or 18 inches.

3) To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottom of all slabs.

11.6 SOIL PRESSURE UNDER THE CULVERT

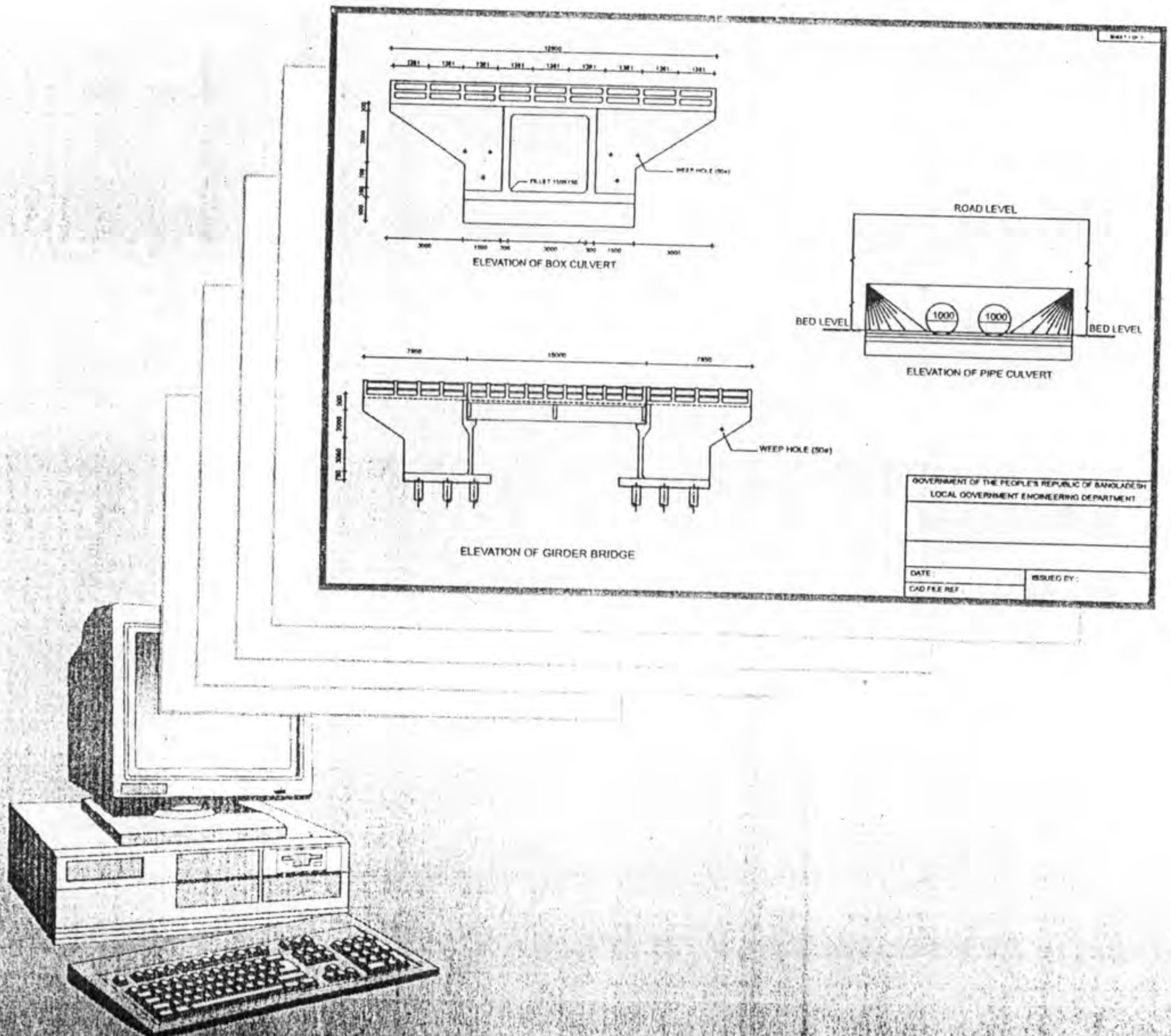
The most severe soil pressures under the bottom slab of box culverts dictate the requirement of soil bearing capacity. The required allowable soil bearing capacity for each box culvert has been presented together with the drawing. The allowable bearing capacity of soil underneath the box culvert for a particular site should be calculated following the procedure as described in Chapter 4. If it is adequate, the standard design may be adopted. In case the bearing capacity requirement exceeds the allowable bearing capacity of the soil, an alternative design should be considered or methods of improving the bearing capacity should be explored. The simplest method of improving bearing capacity of soil may be sand piling or providing a sand bed after removal of top weak soils.

SECTION V SELECTION OF A STANDARD DESIGN

CHAPTER 12: Selection of a Standard Design for Road Structure

CHAPTER 13: Selection of a Standard Design for Defined Channel

CHAPTER 14: Selection of a Standard Design for Undefined Channel



Selection of a Standard Design for Road Structure

Chapter 12 explains the flowchart techniques used to describe the step-by-step procedures for structure selection. The flowchart presented in this chapter briefly shows the steps to be followed for both defined and undefined channel. These steps are discussed elaborately in the subsequent chapters. This chapter also describes the Structure Reference Code (SRC) which can uniquely identify a drawing in the library of standard structures.

CHAPTER 12

Selection of a Standard Design for Road Structure

12.1 GENERAL

Approaches for selection of a standard design for road structures are distinct for defined and undefined channels. The procedures for selection of road structure suitable for these two types of channels have been presented in the form of flow charts. By following through the different steps of the flow charts in this section the user should be able to determine:

i) Where the structure should be located;
ii) What should be the width of waterway;
iii) What should be the depth of foundation;
iv) What should be the span;
v) What type of road structure (e.g. girder bridge, slab culvert, box culvert, pipe culvert, etc.) should be used; and
vi) What should be the type of foundation.

12.2 FLOW CHARTS

Flow charts have been drawn in accordance with the common convention in computer science as shown in Fig. 12.1. Closed rectangular boxes have been used for steps where an action leading to assignment of values or selection of a particular course is contemplated; open rectangular boxes have been used for comments; diamond shaped boxes have been used where logical decisions are necessary; small circles have been used for connectors and large circles to represent instance of another sub-flow chart. It may so happen that users may not be familiar with the action indicated within the closed rectangular box, therefore, reference have been made to appropriate article number of the Manual where the details of the procedure have been described. The Road Structures Software is based on the flow charts presented in this section.

The flow chart in Fig. 12.2 briefly shows the steps involved in selection of a road structure. It starts with a question : whether the channel is defined or undefined ? The 'Yes' side of the chart follows steps required for determining the width of waterway and depth of foundation for a defined channel. The 'No' side leads to the procedures for determining size of opening for an undefined channel. Steps in either of the above channel conditions are same but the methods to carry out those steps are distinct. One has to determine and fix the flow through the channel before deciding upon waterway

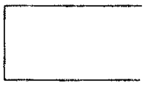
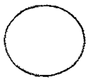
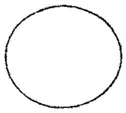
Symbol	Name	Purpose
	Closed Rectangular Box	Actions or Processes
	Open Rectangular Box	Comments
	Diamond	Logical Decisions
	Small Circle	Connectors
	Large Circle	Sub-flow Charts

Fig. 12.1 Flow Chart Symbols

and depth of scour. Both these parameters are important in fixing the dimensions and type of the structure to be selected. However, methods for determining water flow through the channels, fixing dimensions and type of the structure is different for the two types of channels.

Selection procedures of a standard design for a defined channel and that for an undefined channel with their corresponding flow charts are described in detail in Chapter 13 and Chapter 14. At various stages in the flow path the user is referred back to articles of the Manual for detail. The flow chart finally ends with the selection of roadway width, waterway width, span, type of structure, and height of the abutment.

12.3 STRUCTURE REFERENCE CODE

A list of available road structures is presented in Annexure. Each structure can be identified by a unique structure reference code (SRC). SRC is composed of a total of 13 characters of which the first four characters represent the category of structure and the last eight characters, separated from the category by a backslash (\), represents type and various dimensions of the structure. Soft copies of all structure drawings form the drawing library of the road structures software. Drawing library is organized in such a manner that the sub-directory name corresponds to the first four characters and the filename of each drawing corresponds to the last eight characters of SRC.

Category of Structure represented by first four characters of SRC can be BRDG for Bridges; ABUT for Abutments; or CULV for Culverts. The first of the last eight characters, can be either of G, S, R, B, O, P, and U, is to identify the type of the structure. Category and type of all available structures are listed in Table 12.1.

Fig. 12.2
Flow Chart for Selection of Road Structure

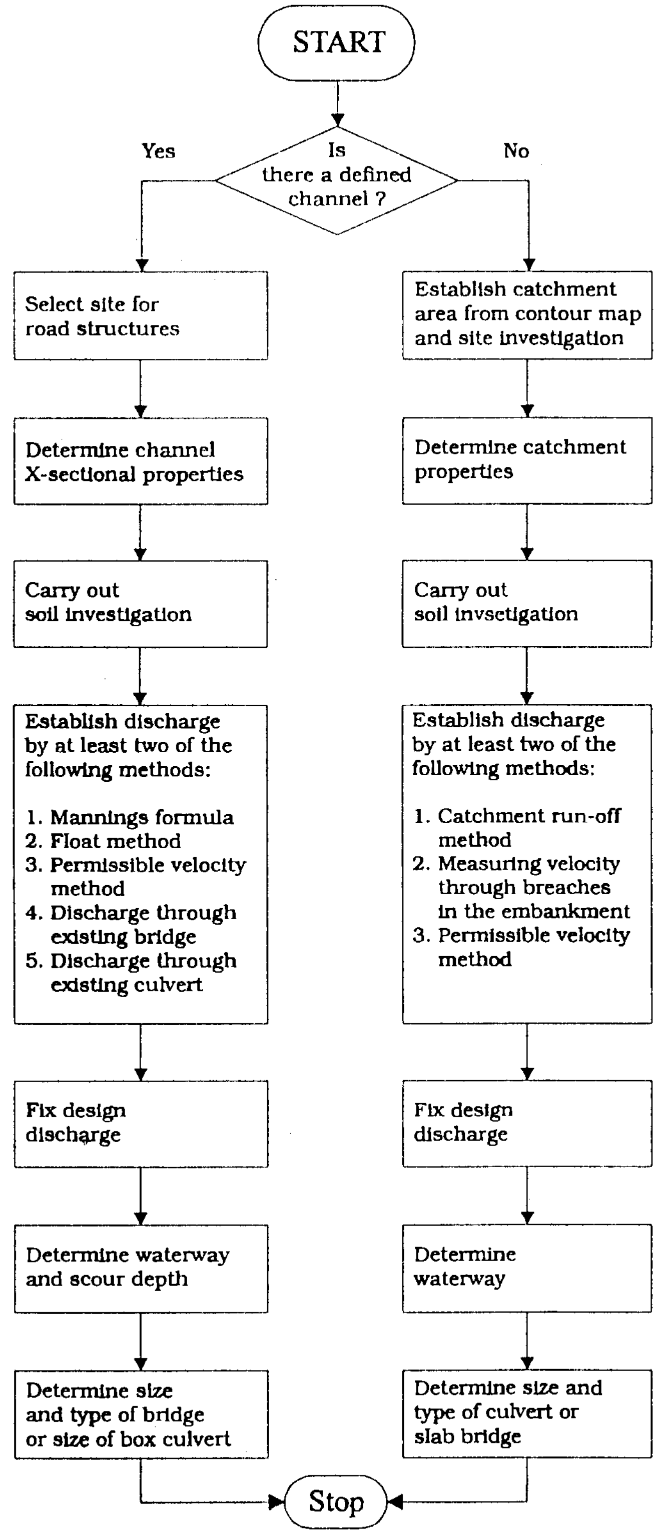


Table 12.1 : Category and type of all available structures

	SRC	Category	Type
1	BRDG\G	Bridge	Girder
2	ABUT\R	Abutment	Full Depth (RCC)
3	ABUT\S	Abutment	Stub (RCC)
4	ABUT\B	Abutment	Open (Brick)
5	ABUT\O	Abutment	Open (RCC)
6	CULV\S	Culvert	Slab
7	CULV\P	Culvert	Pipe
8	CULV\B	Culvert	Box
9	CULV\U	Culvert	U-Drain

Table 12.2 summarizes the meaning of the last eight characters of SRC for each of the structure in Table 12.1. The first numeric column of Table 12.2 holds the character positions relative to the last eight characters of SRC.

Table 12.2 : Meaning of the last eight characters of SRC

	BRDG	ABUT	ABUT	ABUT	ABUT	CULV	CULV	CULV	CULV
1	G	R	S	B	O	S	P	B	U
2	FP	FP	FP	FP	FP	FP	Cell	Cell	Null
3	Load	CW	CW	Null	CW	Load	Null	Load	CW
4	Span	Range	Null	Range	Range	Span	Dia	Span	Span
5	-do-	Null	Pile	Null	Null	-do-	-do-	-do-	-do-
6	-do-	Height	Null	Height	Height	-do-	Null	Height	Height
7	Null	-do-	Null	-do-	-do-	Null	Null	-do-	-do-
8	Sheet	Sheet	Sheet	Sheet	Sheet	Sheet	Sheet	Sheet	Sheet

In SRC existence of Footpath (FP), applicable for girder bridges, slab culverts and all abutments, has been represented by either 0 (without footpath) or 1 (with footpath). Data item Cell can be any value from 1 to 3 for box culverts and 1 to 4 for pipe culverts, to represent single (1), double (2), triple (3), and quadruple (4) cell. Data item Load, applicable for girder bridges, slab culverts and box culverts, can be any value from 1 to 3 to represent H10 (1), H15 (2), and H20 (3) loading. Range, applicable for all abutments except stubs, means span range of super-structure for which the abutment has been designed for 1 in place of range will mean lower span range (1.5m to 12m) and a 2 will mean upper span range (12.5m to 20m). Pile can be either 0 to mean precast-pile or 2 to mean cast-in-situ pile. In place of Span and Height the digits (two or three as continued with -do-) will mean span (diameter for pipe culverts) or height in decimeter. Sheet is meant to represent drawing sheet number. Null will be represented by a zero (0).

An Example of the structure coding system is shown in Fig. 12.3.

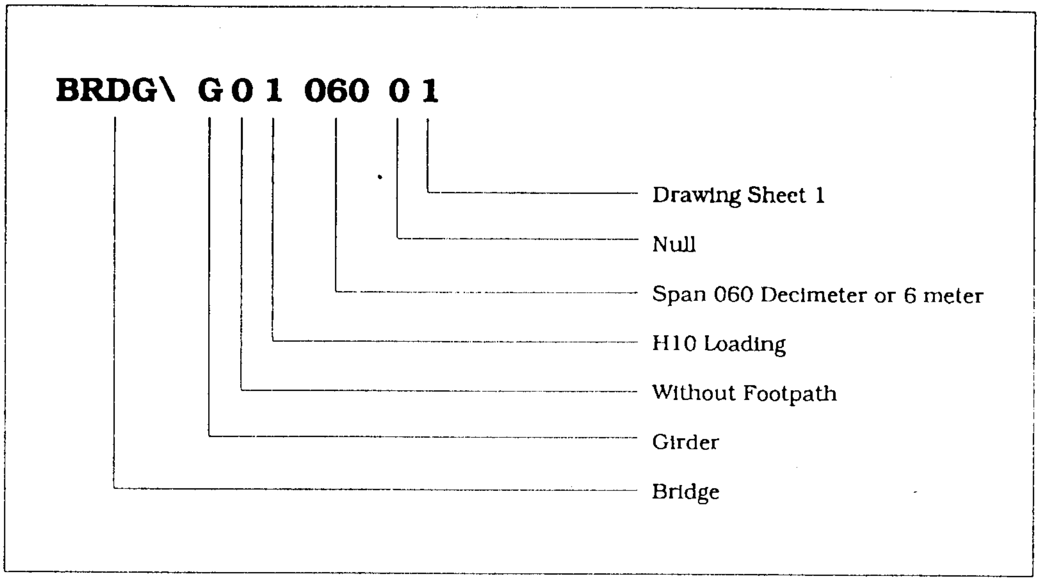


Fig. 12.3 Structure Coding System

Selection of a Standard Design for Defined Channel

Chapter 13 presents a flowchart and explains the steps to be followed in the selection of a standard design for a defined channel. Facts to be considered in site selection; determination of cross-sectional properties; establishing discharge; fixing design discharge; linear waterway and scour depth; depth of foundation and freeboard; and loading are discussed in this chapter. The structure selection procedure for defined channel has been further clarified by illustrated examples.

CHAPTER 13

Selection of a Standard Design for Defined Channel

13.1 GENERAL

A defined channel is a channel that is large enough to contain the peak flow of the waterway. In such a channel, flow is contained within the raised banks for most of the year. Steps in selection of a standard design for this type of channel have been discussed in this chapter. Flow chart of the selection procedures has been presented in Fig. 13.1.

13.2 SELECTION PROCEDURES

13.2.1 Site Selection

In siting small bridges and culverts, due consideration should be given to geometrics of the approach alignment and the latter should essentially govern the selection of site unless there are any special problems of design. Site selection procedure have been discussed in Art (5.4).

13.2.2 Determination of Cross-Sectional Properties

Three cross-sections, one at upstream, one at site location and the other at down stream should be measured and plotted. Upstream and downstream sections should be taken within 150m-250m distance from the site location. In a channel with rigid boundaries (bed and banks) the shape and size of the cross-section is significantly the same during a flood as after its subsidence. But a channel flowing in alluvium, will have a larger cross sectional area when in flood than that which may be surveyed and plotted after the flood has subsided. During the flood the velocity is high and therefore an alluvial channel scours its bed; but when the flood subsides, the velocity diminishes and the bed progressively silts up again. Before estimating the flood discharge of the channel from the cross-section, the probable average scour bed line should be ascertained. See Art 5.8 for more detail.

Establish the Design Flood Level (DFL). Design flood level may or may not be the Highest Flood Level (HFL). The DFL should be ascertained by local observations, supplemented by local inquiry. If water level gauges are available HFL may be obtained from there.

Fig. 13.1
Flowchart for Selection of Standard Design for Defined Channel

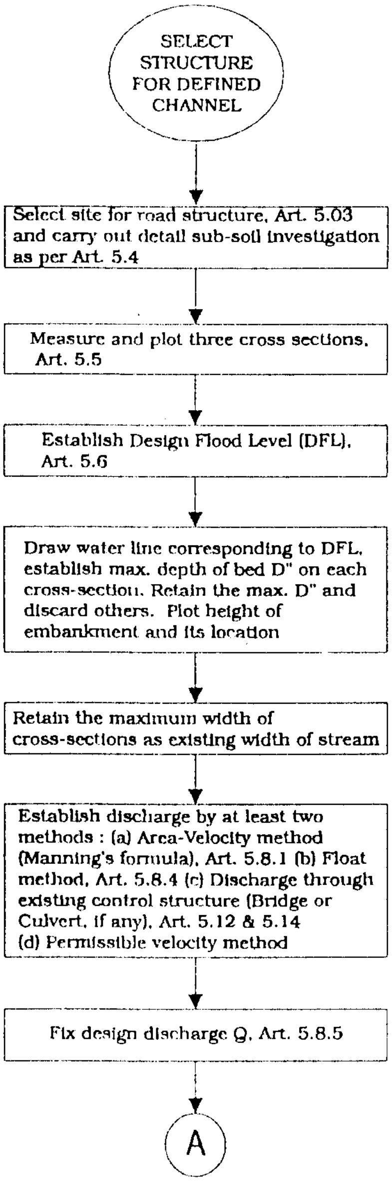


Fig. 13.1 (continued)

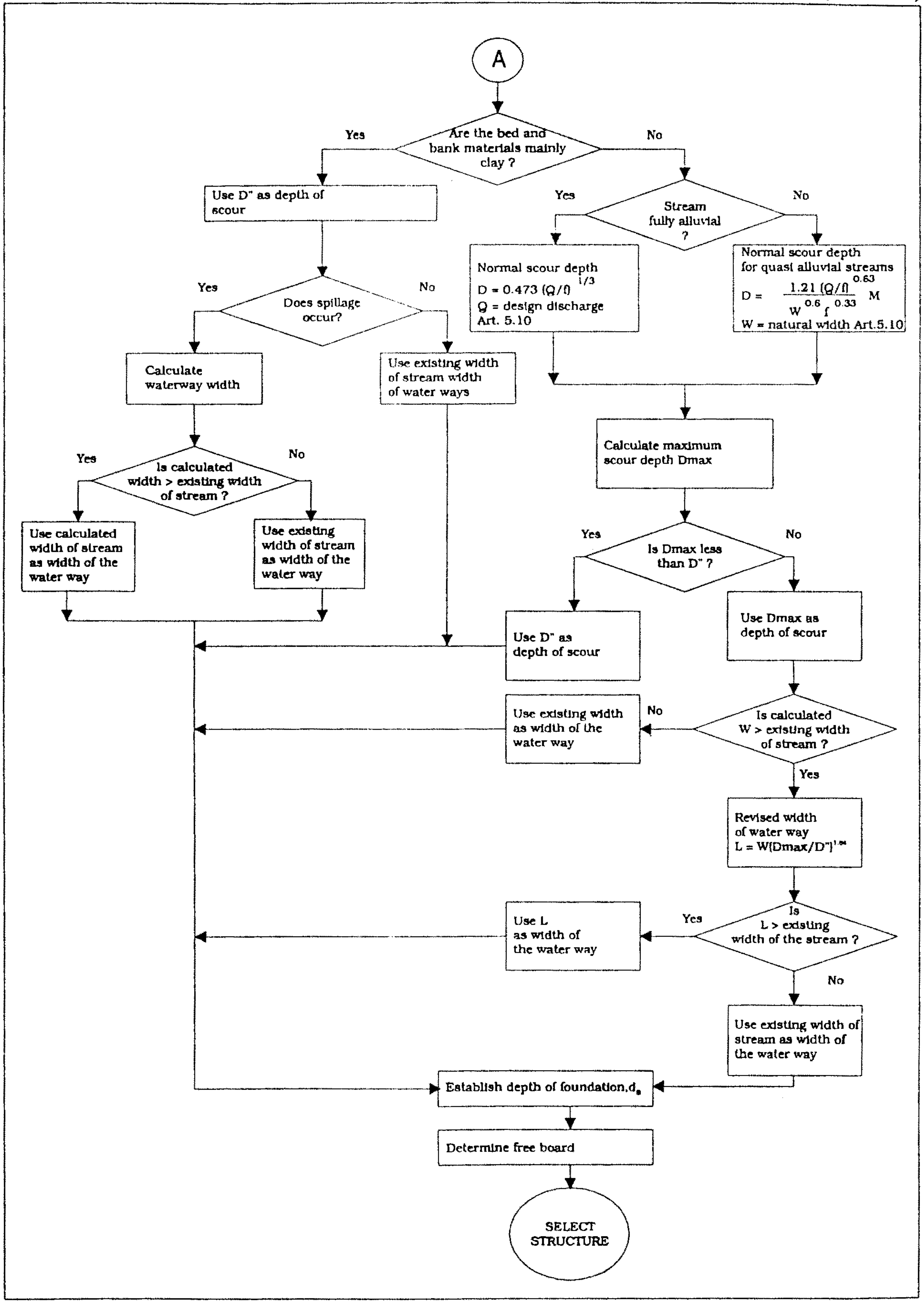


Fig. 13.1 (continued)

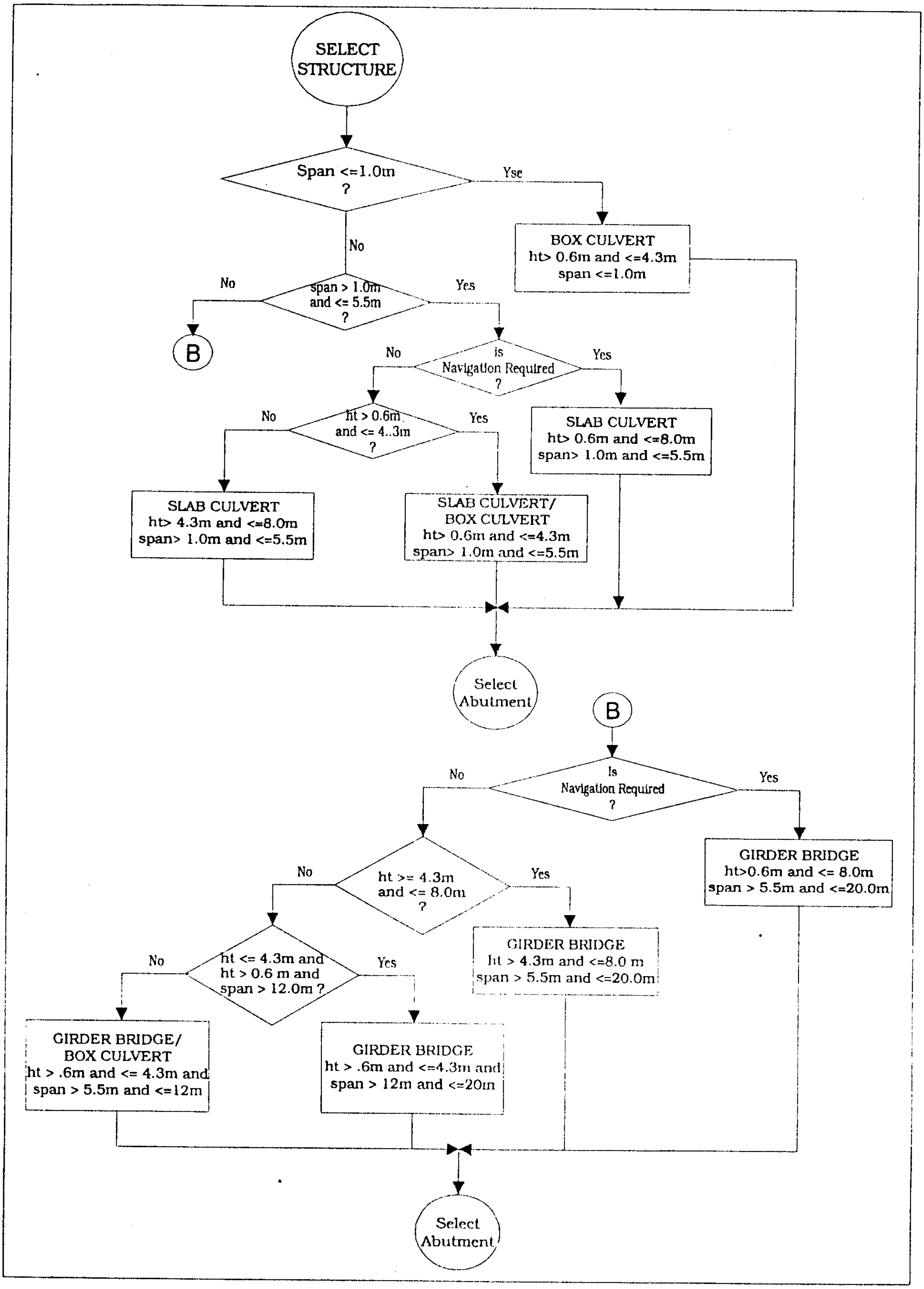
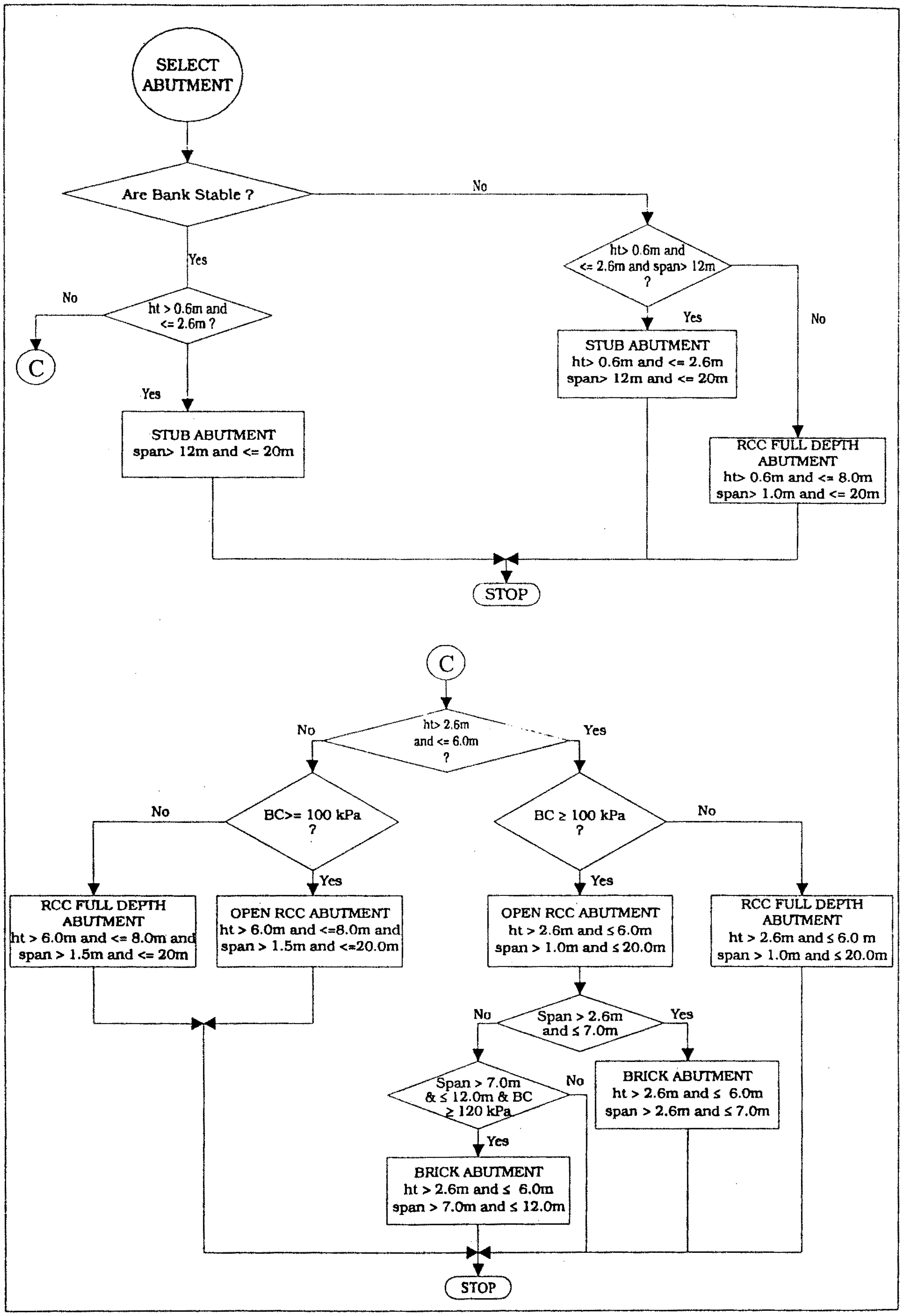


Fig. 13.1 (continued)



Draw water line corresponding to DFL and identify the left and right bank position as the intersection of DFL with the cross-section of the channel. Also plot height of embankment and its location. Determine the maximum cross-section area and maximum depth, wetted perimeter and hydraulic radius at cross-section of maximum area.

13.2.3 Establishing Discharge

At least two methods should be used in establishing discharge. Discharge can be established by Manning's formula, Float Method, or Permissible Velocity Method.

If there is an existing structure with water marks, discharge through the structure can also be calsulated. Distinct water marks on bridge piers and other structures can be easily found immediately following the flood. Sometimes these marks can be identified years afterwards but it is advisable to survey them as soon after the flood as possible. Estimating flood discharge from the flood marks on existing structure (Art 5.12 and Art 5.14) is perhaps the most reliable way, because the coefficients involved have been accurately found by experiments.

13.2.4 Fixing Design Discharge

Flood discharges calculated in various ways should be compared. The highest of these values should be adopted as the design discharge, provided it does not exceed the next highest discharge by more than 50 per cent. If it does, restrict it to that limit. See Art 5.8.5 for more details.

13.2.5 Linear Waterway and Scour Depth

The section of a channel, having rigid boundaries, is the same during the flood and after its subsidence. But it is not so in the case of channels flowing within, partially or wholly, erodible boundaries. The surface width and the normal scoured depth of channels in alluvium have to be calculated theoretically (Art 5.9 and 5.10). As regards channels which overflow their banks and create very wide surface widths with shallow side sections, judgement has to be used in fixing the linear waterway of the bridge. The bridge should span the active channel and detrimental afflux should be avoided.

A channel may flow between banks which are rigid in so far as they successfully resist erosion, but its bed may be composed of loose granular material which the current can pick up and transport. Such a channel may be called quasi-alluvial. It is not essential that the banks should be of rock to be inerodible. Natural mixtures of sand and clay may, under the influence of elements, produce material hard enough to defy erosion by the prevailing velocity in the channel. Such cases have, therefore, got to be discriminated from the fully alluvial. In any such case the width of the section, being fixed between the rigid banks, can be measured. But the scour depth corresponding to the design discharge has to be estimated theoretically (Art 5.10).

13.2.6 Depth of Foundation and Freeboard

Where navigation is required a freeboard of 2.0m (Max^m) above DFL may be maintained, otherwise a 0.5m clearance will be necessary. However, RL of the top of existing road embankment may be above this level. In that case, freeboard needs to be adjusted to the RL of road embankment. Rules of fixing the depth of foundation for various structures have been discussed in Chapter 5 - Hydraulic Consideration.

13.2.7 Loading

Structures have been designed for H20, H15 and H10 loading. For FRB Road, H20 loading is to be considered, for RR1 and RR2 roads H15 loading; and for RR3 roads H10 loading should be considered.

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PROCEDURE TO CONSULT OR ADOPT FLOW CHART FOR SELECTION OF OPTIMUM TYPE OF ROAD STRUCTURE

[FLOW CHART CALCULATIONS]

EXAMPLE CASE : 1 REGIME CHANNEL [Channels having mainly inerodible Bank and Bed]

Note: Assuming the site is visited when it becomes accessible i.e. the period between high flood and dry season.

Step No.	Start From FLOW CHART	Formulation/Course of action/Decision
1.	Is it a well defined channel?	Yes
2.	Select site for road structure	The Considerations for selection of relatively suitable sites are explained in Art. 2.3 of the Manual.
3.	Is there any existing structure with silt mark near the site	No
4.	Measure and plot three cross-sections	3 (three) cross-section should be measured : one at the selected site, one 150m up stream and another 150m down stream, the details of which are given in Art. 5.8.4. of the Manual. The cross section are plotted in Fig. 13.2.
5.	Establish Design Flood level, (DFL) (Silt mark method)	The highest flood level at three cross-sections are recorded as shown in Fig. 13.2 during peak flood period. In absence of any record, this may be obtained from local observations supplemented by inquiries in the locality, the silt marks on tree trunk, buildings or poles etc. It is better to make inquiries by talking to the people individually rather than in groups. Procedure to determine High Flood level (HFL) are explained in detail in Art. 5.6.1 of the Manual. Considering all aspects, Engineer concerned should organize the judgment on the peak flood level which occurs normally for years together during the wettest part of the year. This HFL may be considered as the Design Flood Level (DFL) for the road structure concerned.
Note: Abnormal flood level like 1988 should not be considered as DFL on the ground of economy.		

6.	Establish Bench Mark (B.M)	Assuming that the height or elevation of DFL with respect to silt mark around a school building is found to be+0.4m from the existing ground level (EGL) which is considered as B.M of that locality in absence of permanent B.M (BWDB, IWTA or SOB). This is explained in Art. 3.3.1 of the Manual.
7.	Establish temporary B.M. near each Cross-Section with respect to DFL.	By taking fly levels from that school building (step 6) Elevations of DFL are found at section 150m down stream 0.0m, at section middle of crossing +0.28m (site) respectively, and at section 150m downstream +0.43m shown in Fig. 13.2 & 13.3. DFL at respective cross-section is considered temporary B.M.
8.	Draw water line corresponding to DFL and establish maximum depth of bed: D", on each cross-section. Retain the maximum D" and discard others. Plot height of Embankment and the location of site of road structure.	Two poles are driven on each bank at each section and ELs' of DFL are marked on both poles according to records of DFL. Both ends of the poles are tied securely at EL. of DFL in such a way that the tie is horizontal without having any sag. Depths of water are then measured. The procedure is explained in Art. 3.3.3. of the Manual. Comparing D" at three cross-section, maximum D" is found at cross-section 150m downstream. D" = 3.05m. Existing new alignment of road is shown in Fig. 13.2 on the right bank of the cross-section nearby the location of site and EL. found +1.265m which needs to be connected to the approach of the road structure by slope/gradient. Similarly from other (left) side.
Note:	Poles may be driven conveniently above the line of EL. of DFL marking the EL. of DFL and subtracting from total depths; Art. 3.3.2 & 3.3.3. of the Manual.	
9.	Boundary/soil condition of the channel	As per definition of regime channel soil material for Bed and Bank are clay, which is expected to be stable and susceptible to little scour depending on the excessive flow/velocity during peak flood.
10.	Does spillage of bank occur?	A partial spillage occurs in the existing stream during peak flood period but at crossing site flow is confined within the complete embankment.
11.	Determine channel cross-sectional properties.	Profile of the cross-section of the channel along the proposed road structure i.e. mid section is shown in Fig. 13.2 and explained in detail in Art. 3.3.2 & 3.3.3 of the Manual.

12.	Determine cross-sectional area at midsection i.e. proposed location of site	Since the spillage occurs partially at the location of site i.e. mid section and the shape of cross-section is irregular, channel is divided in sub-sections as shown in Fig. 13.2 and explained in Art. 5.8.1 with Fig. 5.2 of the Manual. Calculation is carried out by Trapezoidal method and given below: Subsection : 1 (starting from left bank) A_1= 1/2 x 0.76 x 0.76 +1.27[(0.76+1.68)/2+(1.68+2.59)/2+ (2.59+2.9) + (2.9+2.74)/2 + (2.74+0.76)/2] = <u>17.5m²</u> $A_2 = 0.76\ (3.05 + 1/2*0.76) = \underline{2.6\ 2\ m^2}$ Therefore, total area : $A = A_1 + A_2 = 20.1\ m^2$ say = $\underline{20\ m^2}$
Note:	Cross-section area in case of irregular shape may also be determined by Simpson's rule.	
13.	Determine peak velocity depending on the channel Characteristics.	Since site is visited during period between high flood & dry season as mentioned in the Note at the 1st page, all site information/data and observations are based on the condition of particular time of visit. So the formulation/coarse of action/decisions are derived to simulate the condition of HFL. So, to assess the selection of appropriate structure, velocity in channel can be determined by the following formula/method: A. Manning's formula: $V = \frac{1}{n} \times R^{2/3} S^{1/2}$ Where, V= Velocity in the channel in m/s R= Hydraulic mean depth or hydraulic radius in 'm' = $\frac{A}{P}$ Where, A = Wetted area in = $20\ m^2$ P = Wetted perimeter in 'm' S = Energy slope which may be taken equal to stream bed slope : 0.001 (Fig. 13.3).

		n = Manning's co-efficient of roughness for the channel bed : 0.07 (Table 5.1 of the Manual) Determination of P (wetted perimeter) Art 5.7.2 Fig. 5.2 (a) & (b) of the Manual. $P_1 = \sqrt{0.76^2 + 0.76^2} + \sqrt{1.27^2 + (1.68 - 0.76)^2} +$ $\sqrt{1.27^2 + (2.59 - 1.68)^2} +$ $\sqrt{1.27^2 + (2.9 - 2.59)^2} + 1.27 +$ $\sqrt{1.27^2 + (2.9 - 2.74)^2} +$ $\sqrt{1.27^2 + (2.74 - 0.76)^2} = 10.415\text{m}$ $P_2 = 3.05 + \sqrt{0.76^2 + 0.76^2} = 4.123\text{m}$ $P = P_1 + P_2 = 14.54\text{m}$ say : 14m $R = A/P = \frac{20}{14} = 1.43\text{m}$ $V = \frac{(1.43)^{2/3}(0.001)^{1/2}}{0.07} = 0.573\text{m/sec}$ B. By permissible velocity method : (From Survey, Design and Construction of Small-scale Rural Infrastructure: IFFW Project Chapter 4: Hydraulic Design) Velocity in cohesive soil susceptible to scour: $V_{peak} = V_p \times S_f$ Where, V_p = Permissible velocity : ft/sec S_f = Flow factor : 0.75 From Graph - II, <u>for fairly compact bed</u> V_p = 2.40 ft/sec Correction for depth : 1.43m ≈ 4.7ft From Graph : 4-III : 1.08 ∴ $V_{peak} = (2.50 \times 1.08 \times 0.75)/3.28 \approx 0.617$ m/sec

14.	Fix design discharge	$Q = A.V$ A. By Manning formula: $Q = 20x0.573 = 11.46$ cumecs B. By permissible velocity method: $Q = 20 \times 0.617 = 12.34$ cumecs So design discharge is fixed (maximum one) : 12.34 cumecs Say, $Q_{Design} = \underline{12 \text{ cumecs}}$

Note: 1. Design discharge should be fixed by cross - checking of at least two methods.
2. Float method is the direct method of measuring the discharge which gives discharge value at the time of its occurrence i.e. during peak flood. If possible, it is suggested to apply this method and the procedure is explained in detail in Art. 5.8.4 of the Manual.

15.	Determine water way: W	The largest width of waterway at HFL is at the cross-section 150m down stream = 760 + 9150 + 2740 + 760 = 13, 410mm = 13.41m The water flow & level mainly remains within 9.15m for most of the time and within 7.62m at middle of crossing and some spillage occurs during peak flood flow. Since Bed & Bank of the stream is inerodible the regime width at the location site i.e. middle of crossing may be considered : W= 9150mm Say : W= 9.0m The linear waterway should be such that the width is sufficient to drain the design discharge without any high afflux and high velocity of flow. Using Lacey's formula : $L= W = 4.75Q^{1/2}$ Where, L = Linear water way in meter W = Regime width = 9m Q = Design discharge = 12m cumecs $L= W = 4.75 \times \sqrt{12} \approx 16.5m>W=9m$

		Art 5.9.1 of the Manual says that the above equation is for streams whose width is large compared to depth of flow, therefore may be somewhat conservative for small streams. So, the scour depth (D) corresponding to the design discharge is determined by the following equation: $D= 0.473\ (Q/f)^{1/3}$ Where, Q = Design discharge = 12 cumecs f = Lacey's Silt factor: Table 5.4 of the Manual = 0.5 where d_m=0.081mm $D=0.473(12/0.5)^{1/3}= 1.364m < D'' = 3.05m$ So, maximum scour depth is fixed : 3.05m Since D is less than D" the width of crossing may be modified to : $L = W.\ (D/D'')^{1.64} = 16.5\ (1.364/3.05)^{1.64}$ $= 4.41m < 9m$ Therefore, linear width of water way is fixed as : W = 9.0m
16.	Define linear width of channel at middle crossing (location of site)	Let A&B be the points joining the location of abutments of both sides of the channel at mid-crossing equal to length as follows: 9000mm L=W= 9.0m Shown in Fig. 13.4. So span of road structure may be considered : 9.0m
17.	Establish depth of foundation for shallow/open foundation	Art 5.15 of the Manual suggests that clay soils are relatively scour resistant. Therefore, there is likely to be very little scour for the stream concerned. The depth of foundation for this case should be at least 1.0m below stream bed level which is modified for shallow/open foundation as follows:

		Depth of foundation (d_s)= Established scour depth of foundation = (D")+1m $d_{s1} = 3.05+1.0 = 4.05$m Say 4.0m Elevation at depth 4m below DFL is : 3.72m at line JK, which is shown in Fig. 13.4.
18.	Add Free Board	If the navigational clearance is not required, it is suggested to provide Free Board as vertical clearance above DFL equal to 750mm as recommended in Art. 6.2.3 of the Manual. So, the EL. of bottom of girder because of free board = +1.03m at line LM as shown in Fig. 13.4.
19.	Add navigational clearance	Art. 6.2.4 of the Manual suggests that considering the local perspective navigational clearance may be provided for the channel to be bridged with span 12m to 20m. The maximum clearance 2.0m may be considered sufficient. So, the navigational clearance = 2.0m from DFL EL. of bottom of Girder = + 2.28m at line OP as shown in Fig. 13.4. *Since the span of bridge is 9.5m, navigational clearance is neglected.
20.	Fix appropriate combination of the structure	A. Span: From hydrological conditions and assessment: Span = 9.5m (Modification of L & W) Fig. 13.4. * Modification of linear water way of the stream is made because of the following reasons : i. To provide adequate water way opening during peak flood. ii. For selection of Appropriate Combination of structure i.e sub & super structure.

		B. Abutment with open/shallow foundation: In consideration to the scour depth i.e. D" 3.05m, the depth of foundation comes to ds_1 = 4.0m below DFL at line JK and the EL. = -3.72 1. Abutment with free board EL. of Free Board at line LM= + 1.03m 2. Abutment height with shallow/open foundation: a. In case of Free Board: 1.03+3.72=4.75m
		C. Abutment with deep/pile foundation: 1. Depth of foundation i.e. top of pile cap is fixed at depth below DFL is : d_{s2}=1.5m at line EF in consideration to the followings: a. Relatively steep slope at left bank. b. In order to avoid major disturbance to the formation and stability of slope. c. In view of getting 'N' value above 6. Increased 'N' value gives fixity of pile at a lower depth thereby reducing the slenderness effect and total length of pile and achieving more contribution of shaft and base resistance. 2. EL. at a depth of top of pile at line EF = -1.22m 3. Abutment height: a. In case of Free Board: 1.22+1.03=2.25m A. Depth of bore holes: Art 4.3.1 of the Manual suggests that depth of bore holes should be 6 to 10 m in case of culvert. In practice, bore holes are done upto depth of 21m from the existing ground level at location of site.

		B. Nos. of Bore holes: Art. 4.3.2 of the Manual recommends that for small span road bridges, minimum two boreholes of 100mm diameter , one at each abutment should be made. By checking and comparing sub-soil information of two bore holes (at left & right bank), soil properties and strength parameters at EL. -3.72m (line JK) at left bank seem to be weaker and have less values. Art. 4.6 of the Manual recommends that the Unified Soil Classification System should be followed and in this respect Table 4.2 and 4.12 are given to follow. C. Assessment of Bearing Capacity 1. Bearing capacity of cohesive soil can be calculated from the unconfined compression strength (q_u) determined by laboratory test of undisturbed soil samples which is more reliable. However, in view of remoteness of site, the assessment is carried out and values are determined in-situ condition: 2. From Bore hole The soil at EL. -3.72m from boring sample (visual inspection) is identified as cohesive. Criteria for determining type of soil are given Table 4.2 & 4.12 and Art. 4.6 of the Manual. a. Assuming that the Bore hole log on both banks are carried out and 'N' value at the left bank is found which is '10' b. As explained in Art 4.8.1.2 of the Manual, unconfined compressive strength : $q_u = kN$ Where, k = Proportionality factor = 16.78 (average value for Bangladesh)

Rod penetration depths are 178mm & 120mm at left & right bank respectively. Correlating with chart shown in Fig.4.13 of the Manual, value of cohesion at left bank (rod penetration depth 178mm) is found (minimum one):

$$c = 68 \text{ KN/m}^2$$

Again putting the value of 'c' in Bearing Capacity formula in Eqn.C-2. above, Allowable Bearing Capacity obtained as follows:

$q_a = 3 \times 68 = 204 \text{ KN.m}^2 > 200\text{KN/m}^2$ (standard)

4. Final assumption of Bearing Capacity:

Comparing the two values of Allowable Bearing Capacity by two methods above, finally it is assumed that Allowable Bearing Capacity of soil for open foundation at EL. -3.72m should be minimum one i.e.
qa = 204 KN/m^2 which is quite satisfactory.

From hydrological & hydraulic considerations above:

A. Span = <u>9.5m</u> i.e (Range 1: upto 12m)

1. Options for spans:

a. Loading : H20, H15 and H10

b. Footpath : With or without

B. Abutment = <u>5.0m</u> height for open foundation

1. Options for abutments:

a. Brick abutment.
b. Abutment full depth (R.C.C) without pile foundation.

When q_u is expressed in KPa or KN/m^2

$q_u = 16.78 \times 10 = 167.8$ KPa

Soil conditions for the structure to be built on the banks of the flowing channel is that Cohesion (C) of the soil is to be obtained at worst possible condition i.e. at saturated submerged state.

but $C = q_u/2$

$$c = \frac{167.8}{2} = 83.9 \text{ KN/m}^2$$

The ultimate bearing capacity of cohesive soil is given below:

$q_{ult} = cN_c$
where,
c = Cohesion in KN/m^2
N_c= Bearing Capacity factor for cohesive soil = 9

$q_{ult} = 9c$

Considering safety factor 3 (Art. 7.11)

$q_a = 3c$ .... (Eqn. C - 2)

where,

q_a = Allowable bearing capacity of soil in KN/m^2

$q_a = 3 \times 83.9 = 251.7$ $KN/m^2 > 200$ KN/m^2 (Standard)

3. From Field observations:

   a. Information on field observations are given in Fig. 13.3.

   b. According to Art. 4.8.1.1 rod penetration is carried out with 5/8" (16mm) dia rod and static weight 55 Kg (120 lbs). It is observed that the uniformity in penetration values at location exists, when dug to the level EL.-3.72m, which indicates uniformity in soil condition.

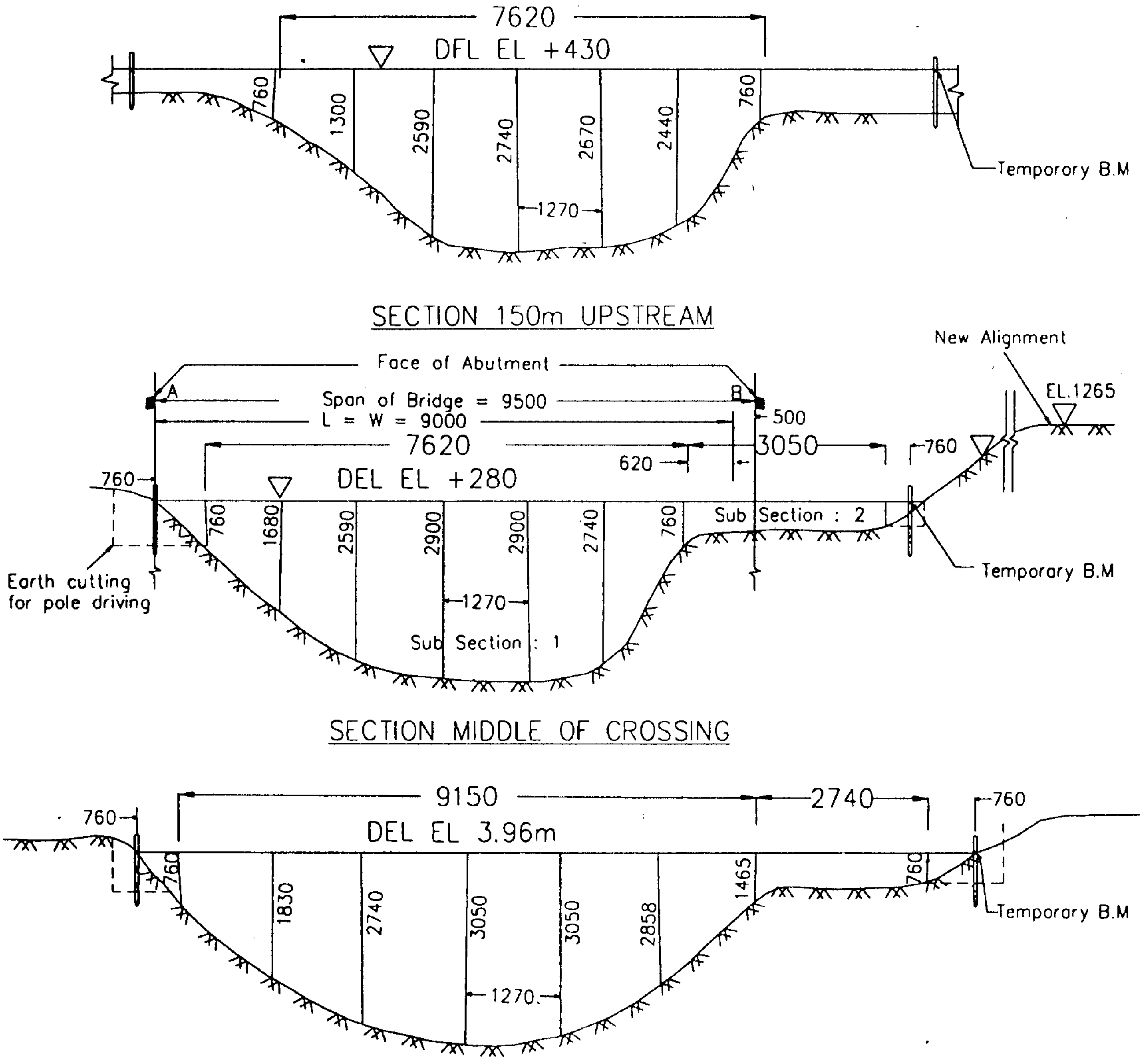
		C. Abutment = <u>3.5</u> height for deep foundation (pile foundation) 1. Options for abutment: a. Abutment full depth (R.C.C) with pilefoundation. b. Abutment full depth (R.C.C) without pile foundation.

Note: 1. Stub abutment can not be considered, because in that case span may be increased excessively beyond Linear Waterway during DFL 12m and pile foundation will have to be provided even at a height of the approaching new alignment on both sides.
2. H10 Loading is neglected in view of carriage way width : 3.66m.

23.	Select cost-effective & appropriate structure with Structure Reference Code (SRC)	In consideration to the followings the structure is selected: A. If the soil condition permits and the height of abutment happens to be within 6m, brick abutment may be adopted. B. Carriage way width : 3.66m C. Loading : H 20 with footpath D. Satisfactory bearing capacity for open foundation at EL.-3.72m i.e. 204 $KN/m^2 > 120\ KN/m^2$ (standard) E. Economy is achieved with brick abutment. So, the selected structure is: **<u>Span : 9.5 BRDG/G 130950</u>** **<u>Abutment: 5.0m ABUT\B0010501</u>** **(Brick abutment)**

* In view of stability of the structure and maximum tow pressure derived from computations of brick abutment, the safe bearing capacity of bearing stratum of soil should be considered greater than 120 KN/m^2 for span within 12.0m.

EXAMPLE CASE : 1



SECTION 150m DOWNSTREAM

Notes :

1. Assumptive field records for the proposed rood structure on an well defined stream:– Exercising the Example : Case 1
2. All dimensions are in "mm"

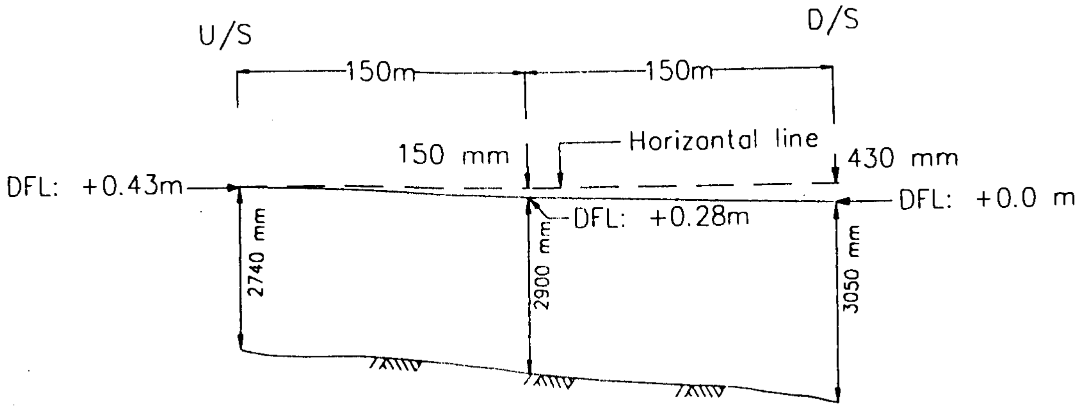
Legend : DFL = Design Flood Level

Fig. 13.2

EXAMPLE CASE : 1

ASSUMPTIVE FIELD OBSERVATIONS FOR EXERCISE OF EXAMPLE, CASE 1

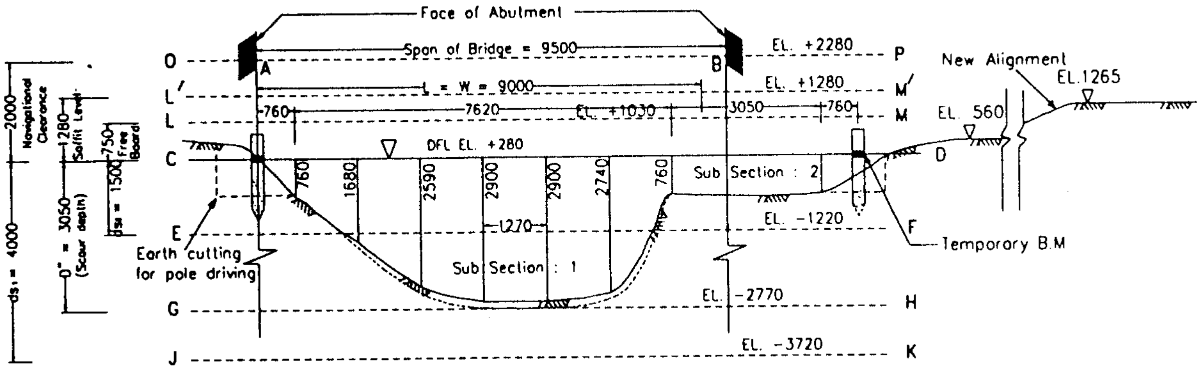
1. Bed and bank materials made of clay (trace silt)
2. Stream bed showed very little sign of silting or scouring
3. Stream bed is weedy with deep pools ($n = 0.07$)
4. Rod penetration by 16 mm dia rod on the left bank of stream at EL. −3.72 m is 178 mm



Notes :
1. Stream bed profile at stream centre
2. Design Flood Level (DFL) is determined by "silt mark method"

Fig. 13.3

EXAMPLE CASE : 1



SECTION MIDDLE OF CROSSING

Notes :

1. Cross section is at the location of Bridge site
2. All dimensions are in "mm"

A. EL. of scour (D") line below DFL : +280−3050 = −2770 at line GS

B. In case of shallow/open foundation
   1. ds_1 = depth of foundation below DFL = 3050+1000 = 4050 say 4000 at line JK
   2. Elevation at foundation level : +280−4000 = −3720 at line JK

C. In case of deep/pile foundation
   1. ds_2 = Depth of foundation below DFL : 1500 at line EF
   2. Elevation at foundation level i.e top of pile cap : +280−1500 = −1220 at line EF

D. In case of provision of Free Board
   1. Height above DFL = 750 at line LM
   2. Elevation of Soffit Level : +280+750 = +1030 at line LM

* D'. Modified EL. for soffit level for Girder Bridge (Loading H20 with side walk)
   1. Height above DFL = 1280 at line L'M'
   2. Modified Elevation of Soffit Level = +3720+1280 = +5000 at line L'M'

** E. In case of provision of navigational clearance
   1. Height above DFL = 2000 at line OP
   2. Elevation of Soffit Level = +280+2000 = +2280 at line OP

* Modification becomes necessary for Appropriate Combination of Structure.
** Navigational Clearance is neglected fat this width of channel.

Fig. 13.4

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PROCEDURE TO CONSULT OR ADOPT FLOW CHART FOR SELECTION OF OPTIMUM TYPE OF ROAD STRUCTURE

[FLOW CHART CALCULATIONS]

EXAMPLE CASE : 2 REGIME CHANNEL [Channels having mainly inerodible bank and erodible bed : quasialluvial stream]

Note: Assuming the site is visited when it becomes accessible i.e. the period between high flood and dry season.

Step No.	Start From FLOW CHART	Formulation/course of action/decision
1.	Is it a well defined channel?	Yes, but the stream is quasialluvial i.e bed is subject to moderate degree of scouring.
2.	Select site for road structure	The Considerations for selection of relatively suitable sites are explained in Art. 2.3 of the Manual.
3.	Is there any existing structure with silt mark near the site	No
4.	Measure and plot three cross-sections	3 (three) cross-section should be measured : one at the selected site, one 150m up stream and another 150m down stream, the details of which are given in Art. 5.8.4. of the Manual. The cross section are plotted in Fig. 13.5.
5.	Establish Design Flood level, (DFL) (Silt mark method)	The highest flood level at three cross-sections are recorded as shown in Fig. 13.5 during peak flood period. In absence of any record, this may be obtained from local observations supplemented by inquiries in the locality, the silt marks on tree trunk, buildings or poles etc. It is better to make inquiries by talking to the people individually rather than in groups. Procedure to determine High Flood level (HFL) are explained in detail in Art. 5.6.1 of the Manual. Considering all aspects, Engineer concerned should organize the judgment on the peak flood level which occurs normally for years together during the wettest part of the year. This HFL may be considered as the Design Flood Level (DFL) for the road structure concerned.

Note:	Abnormal flood level like 1988 should not be considered as DFL on the ground of economy.	
6.	Establish Bench Mark (B.M)	Assuming that the height or elevation of DFL with respect to silt mark around a school building is found to be+0.4m from the existing ground level (EGL) which is considered as B.M of that locality in absence of permanent B.M (BWDB, IWTA or SOB). This is explained in Art. 3.3.1 of the Manual.
7.	Establish temporary B.M. near each Cross-Section with respect to DFL.	By taking fly levels from that school building (step 6) Elevations of DFL are found at section 150m down stream 0.0m, at section middle of crossing +0.28m (site) respectively, and at section 150m downstream +0.43m shown in Fig. 13.5 & 13.6. DFL at respective cross-section is considered temporary B.M.
8.	Draw water line corresponding to DFL and establish maximum depth of bed: D", on each cross-section. Retain the maximum D" and discard others. Plot height of Embankment and the location of site of road structure.	Two poles are driven on each bank at each section and ELs' of DFL are marked on both poles according to records of DFL. Both ends of the poles are tied securely at EL. of DFL in such a way that the tie is horizontal without having any sag. Depths of water are then measured. The procedure is explained in Art. 3.3.3. of the Manual. Comparing D" at three cross-section, maximum D" is found at cross-section 150m downstream. D" = 3.05m. Existing new alignment of road is shown in Fig. 13.5 on the right bank of the cross-section nearby the location of site and EL. found +1.265m which needs to be connected to the approach of the road structure by slope/gradient. Similarly from other (left) side.
Note:	Poles may be driven conveniently above the line of EL. of DFL marking the EL. of DFL and subtracting from total depths; Art. 3.3.2 & 3.3.3. of the Manual.	
9.	Boundary/soil condition of the channel	As per definition quasialluvial stream (Art. 5.9.2 of the Manual): banks of the stream are high, well defined and rigid (rocky or somewhat other natural hard soil such as clay that can not be affected by the prevailing current), but the bed is alluvial i.e. erodible.

10.	Does spillage of bank occur?	A partial spillage occurs in the existing stream during peak flood period but at crossing site flow is confined within the complete embankment.
11.	Determine channel cross-sectional properties.	Profile of the cross-section of the channel along the proposed road structure i.e. mid section is shown in Fig. 13.5 and explained in detail in Art. 3.3.2 & 3.3.3 of the Manual.
12.	Determine cross-sectional area at midsection i.e. proposed location of site	Since the spillage occurs partially at the location of site i.e. mid section and the shape of cross-section is irregular, channel is divided in sub-sections as shown in Fig. 13.5 and explained in Art. 5.8.1 with Fig. 13.6 of the Manual. Calculation is carried out by Trapezoidal method and given below: Subsection : 1 (starting from left bank) $A1= 1/2 \times 0.76 \times 0.76$ $+ 1.27[(0.76+1.68)/2 + (1.68+2.59)/2 + (2.59+2.9) + (2.9+2.74)/2 + (2.74+0.76)/2]$ $= 17.5 m^2$ $A_2 = 0.76 (3.05 + 1/2*0.76) = 2.6\ 2\ m^2$ Therefore, total area : $A = A_1 + A_2 = 20.1\ m^2$ say $= \underline{20\ m^2}$
Note:	Cross-section area in case of irregular shape may also be determined by Simpson's rule.	
13.	Determine peak velocity depending on the channel Characteristics.	Since site is visited during period between high flood & dry season as mentioned in the Note at the 1st . page, all site information/data and observations are based on the condition of particular time of visit. So the formulation/coarse of action/decisions are derived to simulate the condition of HFL. So, to assess the selection of appropriate structure, velocity in channel can be determined by the following formula/ method: A. Manning's formula: $V= \frac{1}{n} \times R^{2/3} S^{1/2}$

Where,

V= Velocity in the channel in m/s

R= Hydraulic mean depth or hydraulic radius in 'm' = $\frac{A}{P}$

Where, A = Wetted area in = 20 m^2

P = Wetted perimeter in 'm'

S = Energy slope which may be taken equal to stream bed slope : 0.001 (Fig 13.6)

n= Manning's co-efficient of roughness for the channel bed : 0.04 (Table 5.1 of the Manual)

Determination of P (wetted perimeter)
Art 5.7.2 Fig. 5.2 (a) & (b) of the Manual.

$$P_1 = \sqrt{0.76^2 + 0.76^2} + \sqrt{1.27^2 + (1.68 - 0.76)^2} +$$

$$\sqrt{1.27^2 + (2.59 - 1.68)^2} +$$
$$\sqrt{1.27^2 + (2.9 - 2.59)^2} + 1.27 +$$
$$\sqrt{1.27^2 + (2.9 - 2.74)^2} +$$
$$\sqrt{1.27^2 + (2.74 - 0.76)^2} = 10.415\text{m}$$

$P_2 = 3.05 + \sqrt{0.76^2 + 0.76^2} = 4.123$m

$P = P_1 + P_2 = 14.54$m say :14m

$$R = A/P = \frac{20}{14} = 1.43\text{m}$$

$$V = \frac{(1.43)^{2/3}(0.001)^{1/2}}{0.04} = 1.0\text{m/sec}$$

B. From Survey, Design and Construction of small scale rural infrastructure: IFFW project, Chapter 4: Hydraulic Design critical velocity is the velocity which maintain a regime channel. It varies with depth of flow, (herein afterscour depth : D"= 3.05m) the type of soil in the stream Bed and the silt load being carried by water.

Assuming the stream bed is formed predominaufly with non cohesive soil i.e silt, the Design velocity is normally 1.25 times the critical velocity on the characteristics of stream (Art. 4.41):

$V_{design} = 1.25 \times V_c \times 0.85$

		Where, Vc = Critical velocity for the stream from Graph 4-1 = 3.3 ft /sec for Depth of stream = 3.05m = 10ft in silt bed Therefore, V_{design} : 1.25 x 3.3 x 0.85 = 3.50 ft/sec = 1.07 m/s
14.	Fix design discharge	Q = A.V A. By Manning formula: Q = 20 x 1.0 = 20.0 cumecs B. By Critical velocity method: Q = 20 x 1.07 = 21.4 cumecs So Design Discharge is fixed (maximum one) : 20.4 cumecs Say, Q_{Design} = 20.5 cumecs
Note: 1. Design discharge should be fixed by cross - checking of at least two methods. 2. Float method is the direct method of measuring the discharge which gives discharge value at the time of its occurrence i.e. during peak flood. If possible, it is suggested to apply this method and the procedure is explained in detail in Art. 5.8.4 of the Manual.		
15.	Determine Water way: W	The largest width of waterway at HFL is at the cross-section 150m down stream = 760 + 9150 + 2740 + 760 = 13, 410mm = 13.41m The water flow & level mainly remains within 9.15m for most of the time and some spillage occurs during peak flood flow. The regime width at the location site i.e. middle of crossing is: W= 760 + 7620 + 3050 + 760 = 12190mm But the linear waterway should be such that the width is sufficient to drain the <u>design discharge</u> without any high afflux and high velocity of flow. So, the linear waterway of the channel is modified to as follows:

$L = W = 13.0m$

Using Lacey's formula :

$L = W = 4.75Q^{½}$

Where,

L = Linear water way in meter = 13.0m
W = Regime width = 13.0m
Q = Design discharge = 20.5m cumecs

$L = W = 4.75 \times \sqrt{20.5} \approx 21.5m > W = 9m$

Art 5.9.1 of the Manual says that the above equation is for streams whose width is large compared to depth of flow, therefore may be somewhat conservative for small streams. So, the scour depth (D) corresponding to the design discharge is determined by the following equation

$D = 0.473\ (Q/f)^{1/3}$

Where,

Q = Design discharge = 20.5 cumecs
f = Lacey's Silt factor: Table 5.4 of the Manual = 0.6
where d_m=0.12mm

$D = 0.473(20.5/0.6)^{1/3} = 1.535m < D" = 3.05m$

For quasialluvial streams whose banks are rigid, the normal scour depth for the natural unobstructed width of the channel may be calculated by using the equation as follows

$D = (1.21\ Q^{1/3}) / (W^{0.6} \times f^{0.33})$

$= 2.06m < D" = 3.05m$

So, the maximum scour depth is considered = 3.05m

16.	Define linear width of channel at middle crossing (location of site)	Let A&B be the points joining the location of abutments of both sides of the channel at mid-crossing equal to length as follows: 405 (left bank) + 760 (left bank) + 7620 + 3050 (right bank) + 760 (right bank) 405 (right bank) = 13000mm So, L=W= 13.0m Shown in Fig.13.7 So span of road structure may be considered = 13.0m
17.	Establish depth of foundation for shallow/open foundation	Art 5.15 of the Manual suggests that clay soils at both banks are relatively scour resistant. Therefore, there is likely to be very little scour for the stream concerned. The depth of foundation for this case should be at least 1.0m below stream bed level which is modified for shallow/open foundation as follows: Depth of foundation (d_s)= Established scour depth of foundation = (D")+1m d_{s1} = 3.05+1.0 = 4.05m Say 4.0m Elevation at depth 4m below DFL is :-3.72m at line JK, which is shown in Fig 13.7.
18.	Add Free Board	If the navigational clearance is not required, it is suggested to provide Free Board as vertical clearance above DFL equal to 750mmas recommended in Art. 6.2.3 of the Manual. So, the EL of bottom of girder because of free board = +1.03m at line LM as shown in Fig. 13.7.
19.	Add navigational clearance	Art. 6.2.4 of the Manual suggests that considering the local perspective navigational clearance may be provided for the channel to be bridged with span 12m to 20m. The maximum clearance 2.0m may be considered sufficient. So, the navigational clearance = 2.0m from DFL

		EL. of bottom of Girder = + 2.28m at line OP as shown in Fig.13.7. *Since the span of bridge is 13.0m, navigational clearance is to be provided.
20.	Fix appropriate combinations of the structure	A. Span: From hydrological conditions and assessment: Span = 13.0m From Fig. 13.7. B. Abutment with open/shallow foundation: In consideration to the scour depth i.e. D" 3.05m, the depth of foundation comes to ds_1 = 4.0m below DFL at line JK and the EL. = -3.72 1. Abutment with free board EL. of Free Board at line LM= + 1.03m 2. Abutment height with shallow/open foundation: a. In case of Free Board: 1.03+3.72=4.75m
		C. Abutment with deep/pile foundation: 1. Depth of foundation i.e. top of pile cap is fixed at depth below DFL is : d_{s2}= 1.5m at line EF in consideration to the followings: a. Relatively steep slope at left bank. b. In order to avoid major disturbance to the formation and stability of slope. c. In view of getting 'N' value above 6. Increased 'N' value gives fixity of pile at a lower depth thereby reducing the slenderness effect and total length of pile and achieving more contribution of shaft and base resistance. 2. EL. at a depth of top of pile at line EF= -1.22m 3. Abutment height:

a. In case of Free Board: 1.22+1.03=2.25m

A. Depth of bore holes:

Art 4.3.1 of the Manual suggests that depth of bore holes should be 6-10m in case of culvert. In practice, bore holes are done upto depth of 21m from the existing ground level at location of site.

B. Number. of Bore holes:

Art. 4.3.2 of the Manual recommends that for small span road bridges, minimum two boreholes of 100mm diameter, one at each abutment should be made.

By checking and comparing sub-soil information of two bore holes (at left & right bank), soil properties and strength parameters at EL. -3.72m (line JK) at left bank seem to be weaker and have less values.

Art. 4.6 of the Manual recommends that the Unified Soil Classification System should be followed and in this respect Table 4.2 and 4.12 are given to follow.

C. Assessment of Bearing Capacity

1. Bearing capacity of cohesive soil (sandy clay) can be calculated from the unconfined compression strength (q_u) determined by laboratory test of undisturbed soil samples which is more reliable.

   However, in view of remoteness of site, the assessment is carried out and values are determined in-situ condition:

2. From Bore hole
   The soil at EL. -3.72m from boring sample (visual inspection) is identified as cohesive in the main. Criteria for determining type of soil are given Table 4.2 & 4.12 and Art. 4.6 of the Manual.

a. Assuming that the Borehole log on both banks are carried out and 'N' value on the left bank is found less which is '10'

b. As explained in Art. no. 4.8.1.2.

unconfined compressive strength : q_u

q_u = kN accordingly to Equation 4.7 of the Manual.

Where, k = Proportionality factor = 16.78 (average value for Bangladesh)

When q_u is expressed in KPa or KN/m^2

$q_u = 16.78 \times 10 = 167.8$ Kpa

Soil conditions for the structures to be constructed on the banks of the following channel is such that Cohesion (C) of the soil is to be obtained at worst possible condition i.e. at saturated submerged state.

but $C = q_u/2$

$$C = \frac{167.8}{2} = 83.9\ KN/m^2$$

The ultimate bearing capacity of cohesive soil is given below:

$q_{ult} = cN_c$

where,
c= Cohesion in KN/m^2
N_c= Bearing Capacity factor for cohesive soil = 9

$q_{ult} = 9c$
Considering safety factor 3 (Art. 7.11)

$q_a = 3c$ .... Equ. C-2

where,
q_a = Allowable bearing capacity of soil in KN/m^2

$q_a = 3 \times 83.9 = 251.7\ KN/m^2 > 200\ KN/m^2$ (Standard)

3. From Field observations:

   a. Information on field observations are given in Fig. 13.6.

   b. According to Art. 4.8.1.1 rod penetration is carried out with 5/8" (16mm) dia rod and static weight 55 Kg (120 lbs). It is observed that the uniformity in penetration values at location exists, when dug to the level EL.-3.72m, which indicates uniformity in soil condition.

   Rod penetration depths are 178mm and 116mm at left & right bank respectively. Correlating with chart shown in Fig. 4.13 of the Manual value of cohesion at left bank (rod penetration depth 165mm) is found (minimum one):

$$C = 69 \text{ KN/m}^2$$

   Again putting the value of 'C' in Bearing Capacity formula in Equ. C-2 above, Allowable Bearing Capacity obtained as follows:

   $q_a = 3 \times 69 = 207 \text{ KN.m}^2 > 200\text{KN/m}^2$ (standard)

4. Final assumption of Bearing Capacity:

   Comparing the two values of Allowable Bearing Capacity by two methods above, finally it is assumed that Allowable Bearing Capacity of soil for open foundation at EL. -3.72m should be minimum one i.e.
   $qa = 207 \text{ KN/m}^2$ which is quite satisfactory.

   From hydrological & hydraulic considerations above:

A. Span = <u>13.0m</u> i.e. (Range 2: From 12.0m to 20.0m)

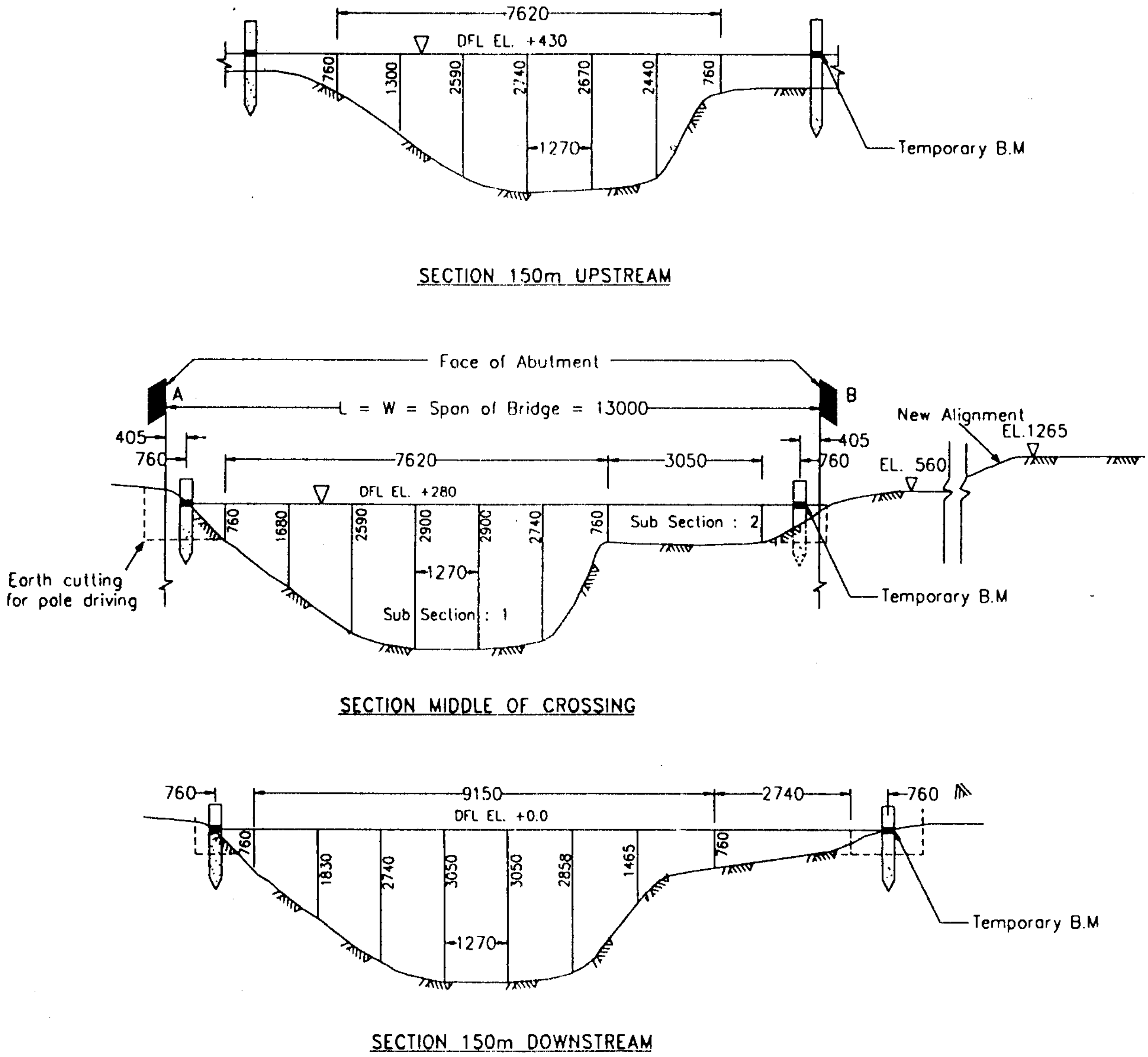
   1. Options for spans:

   a. Loading : H20, H15 and H10
   b. Footpath : With or without

		In consideration to navigational clearance & bearing capacity ≥ 200* KN/m^2 B. Abutment = 6.0m height for open foundation 1. Options for abutments: a. Brick abutment. b. Abutment full depth (R.C.C) without pile foundation. C. Abutment = 3.5 height for deep foundation (pile foundation) 1. Options for abutment: a. Abutment full depth (R.C.C) with pile foundation. b. Abutment full depth (R.C.C) without pile foundation.
Note: 1. Span of bridge 13.0m with free board only is neglected in view of provision of navigational clearance which is appropriate. 2. Stub abutment can not be considered, because in that case span may be increased excessively and pile foundation will have to be provided even at a height of the approaching new alignment on both sides in order to provide nagivational clearance. 3. H-10 Loading is neglected in view of carriage way width : 3.66m.		
23.	Select cost-effective & appropriate structure with Structure Reference Code (SRC)	In consideration to the followings the structure is selected: A. If the soil condition permits and the height of abutment happens to be within 6m, brick abutment may be adopted. B. Carriage way width : 3.66m C. Loading : H-20 with footpath D. Satisfactory bearing capacity for open foundation at EL.-3.72m i.e. 207 KN/m^2> 200 KN/m^2 (standard) E. Economy may be achieved with brick abutment but since span is greater than 12.0m brick abutment cannot be provided.

		So, the selected structure is: **<u>Span : 13.0m BRDG/G 131300</u>** **<u>Abutment: 6.0m ABUT\01120601</u>** **(RCC Open abutment)**

EXAMPLE CASE : 2



Notes :

1. Assumptive field records for the proposed road structure on on well defined stream:– Exercising the Example : Case 2
2. All dimensions are in "mm"

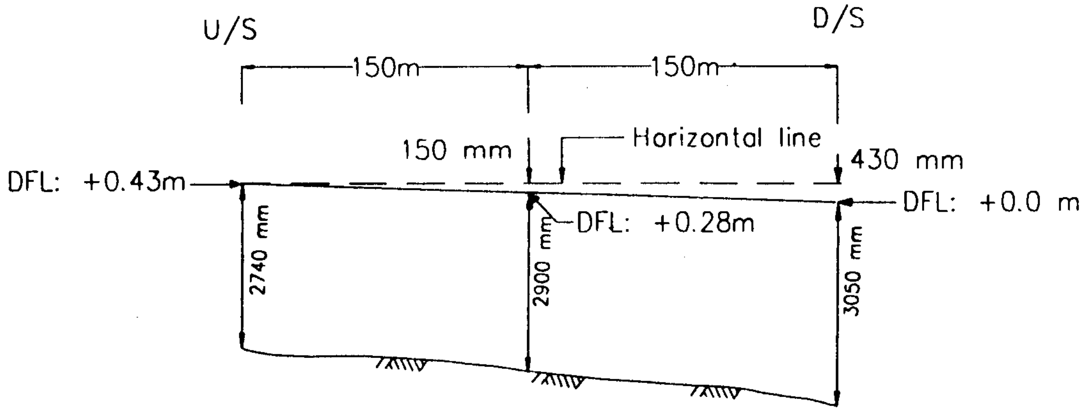
Legend : DFL = Design Flood Level

Fig. 13.5

EXAMPLE CASE.: 2

ASSUMPTIVE FIELD OBSERVATIONS FOR EXERCISE OF EXAMPLE, CASE 2

1. Banks are inerodible and Bed is erodible
2. Stream bed showed moderate scouring
3. Stream is flowing through cultivated areas associated with Mature field crops (n = 0.04)
4. Rod penetration by 16 mm dia rod on the left bank of stream at EL. −3.72 m is 165 mm

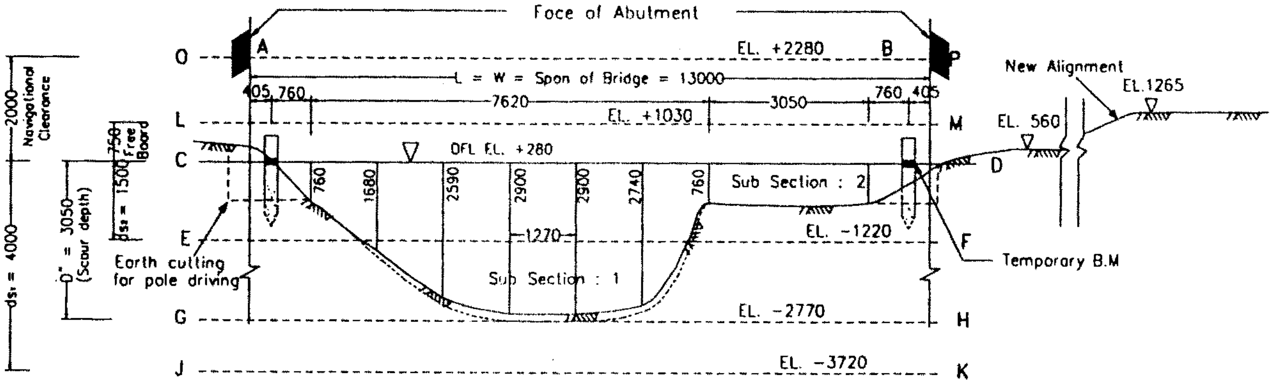


Notes :

1. Stream bed profile at stream centre
2. Design Flood Level (DFL) is determined by "silt mark method"

Fig. 13.6

EXAMPLE CASE : 2



SECTION MIDDLE OF CROSSING

Notes :

1. Cross section is at the location of Bridge site
2. All dimensions are in "mm"

A. EL. of scour (D") line below DFL : +280−3050 = −2770 at line GS

B. In case of shallow/open foundation
   1. ds_1 = depth of foundation below DFL = 3050+1000 = 4050 say 4000 at line JK
   2. Elevation at foundation level : +280−4000 = −3720 at line JK

C. In case of deep/pile foundation
   1. ds_2 = Depth of foundation below DFL : 1500 at line EF
   2. Elevation at foundation level i.e top of pile cap : +280−1500 = −1220 at line EF

D. In case of provision of Free Board
   1. Height above DFL = 750 at line LM
   2. Elevation of Soffit Level : +280+750 = +1030 at line LM

E. In case of provision of navigational clearance
   1. Height above DFL = 2000 at line OP
   2. Elevation of Soffit Level = +280+2000 = +2280 at line OP

Fig. 13.7

Selection of a Standard Design for Undefined Channel

Chapter 14 presents another flowchart and explains the steps to be followed in the selection of a standard design for an undefined channel. Facts to be considered in determination of catchment properties, establishing discharge, fixing design discharge, type and size of structure are discussed in this chapter. The structure selection procedure for undefined channel has been further clarified by an illustrated example.

CHAPTER 14

Selection of a Standard Design for Undefined Channel

14.1 GENERAL

An undefined channel is a channel where no definite stream exists and the flow is caused by a natural depression in the terrain. Steps in selection of a standard design for this type of channel have been discussed in this chapter. The Flow chart of the selection procedures is presented in Fig. 14.1.

14.2 SELECTION PROCEDURES

14.2.1 Determination of Catchment Properties

A contour maps or aerial photos of the area may be obtained from BWDB, SOB or other sources. From this map and from interviewing local people, the general slope of the terrain, the general direction of flow, the drainage areas feeding the crossing site and water levels during high floods should be established. The percentage of the total drainage area drained by the proposed structure and not by other existing or planned structures for the catchment has to be calculated. In addition, the allowable head-up of water level should be fixed so as to avoid undue submergence at the upstream site.

14.2.2 Establishing Discharge

At least two methods should be used in establishing discharge. Discharge can be established by Catchment Run-off Method, Direct Method (measuring the velocity where possible, e.g. existing breaches in the embankment), or Permissible Velocity Method.

14.2.3 Fixing Design Discharge

The same rule discussed in Art 13.2.4 applies for fixing the design discharge for an undefined channel. However, discharge obtained in this way should be multiplied with the percentage of flow (Art 14.2.1) that will be drained through the structure.

Fig. 14.1
Flowchart for Selection of Standard Design for Undefined Channel

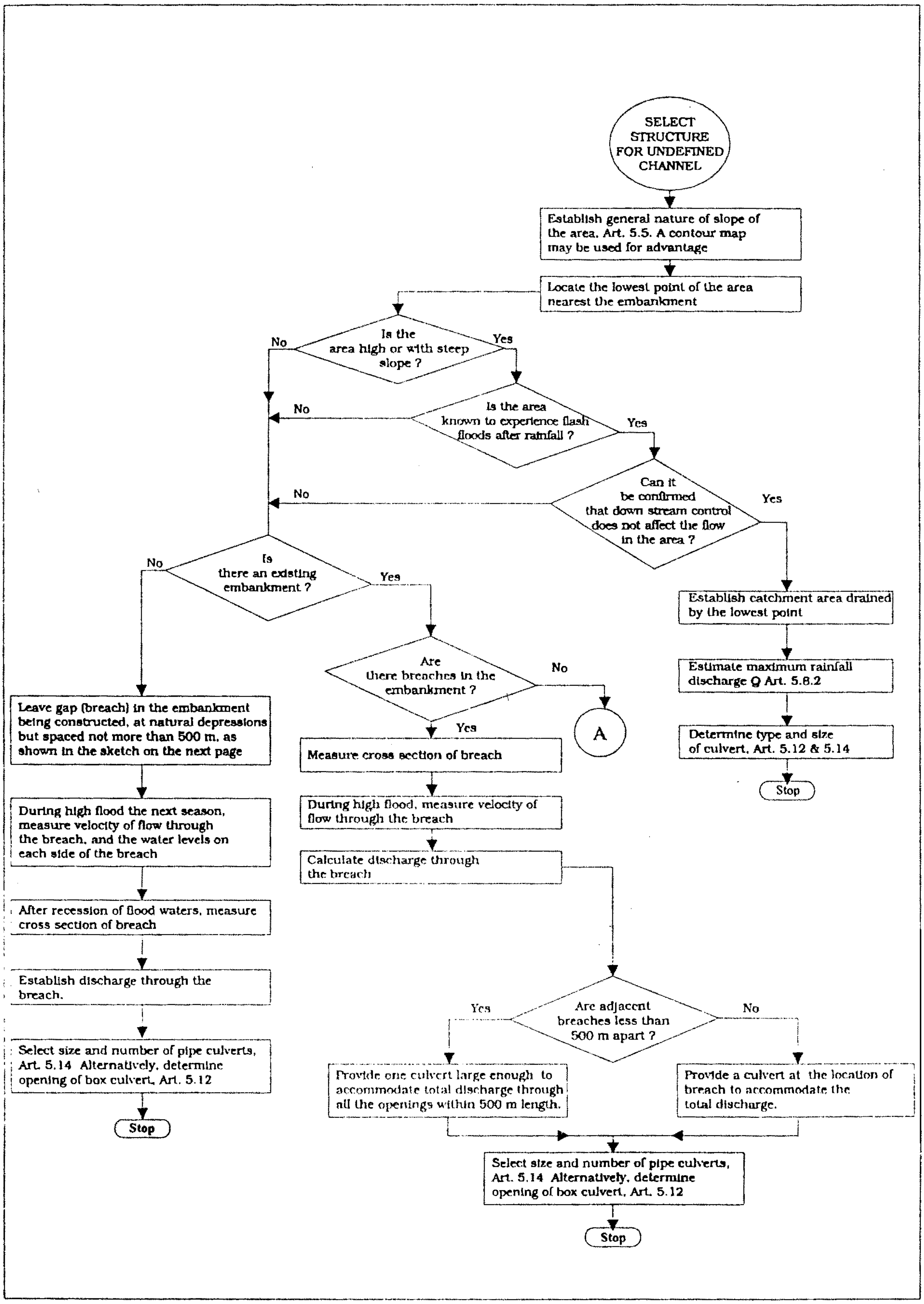
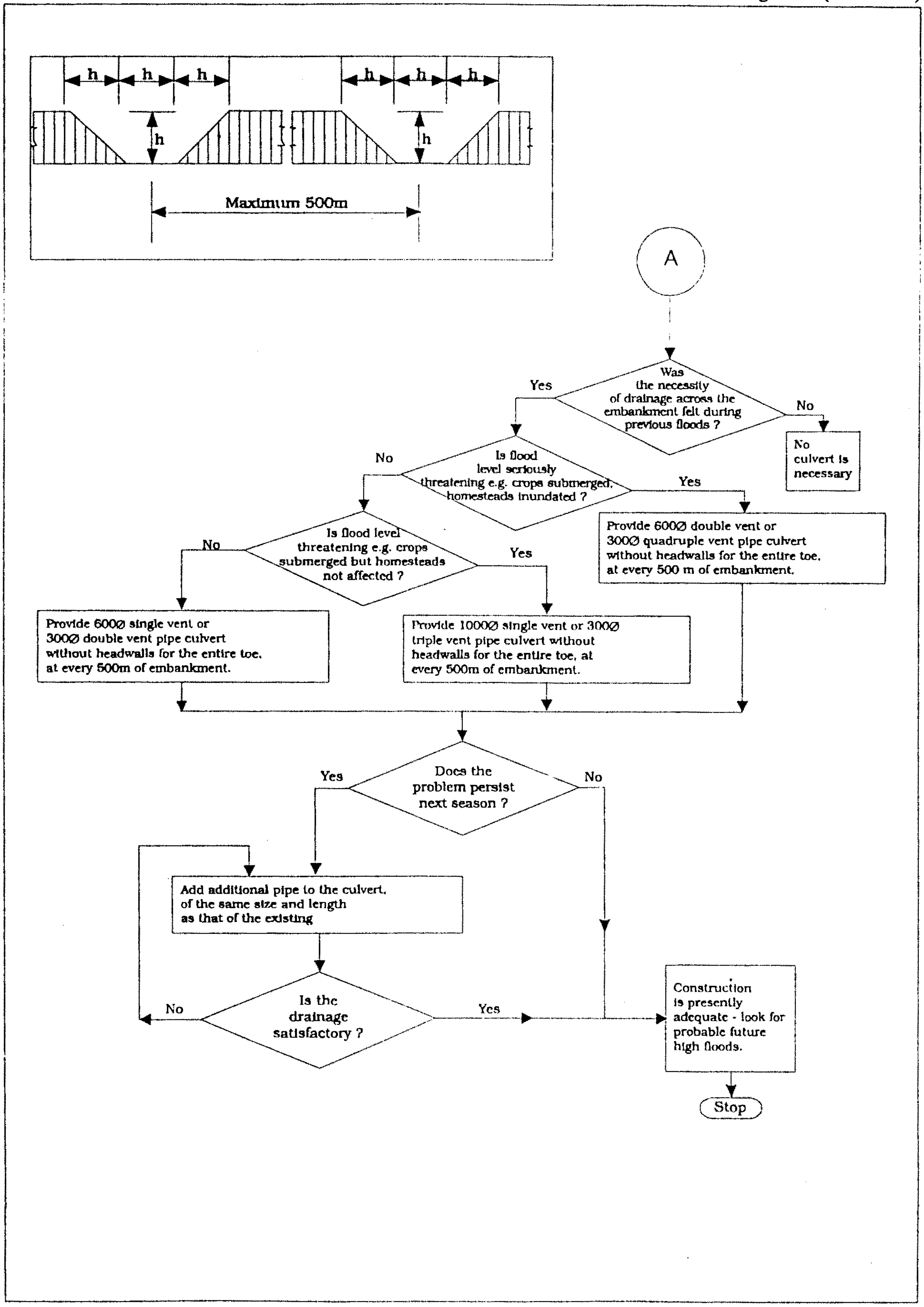


Fig. 14.1 (continued)



14.2.4 Type and Size of Structure

For small to medium size discharge u-drains, pipe culverts or box culverts are to be provided. For medium to large size discharges slab culvert or bridges with stub abutments or open foundations might be appropriate. In the determination of size of pipe culvert full flow condition should be assumed (Art. 5.14). Size of box culverts and bridges should be determined in accordance with Art. 5.12.

PROCEDURE TO CONSULT OR ADOPT FLOW CHART FOR SELECTION OF OPTIMUM TYPE OF ROAD STRUCTURE

[FLOW CHART CALCULATIONS]

EXAMPLE CASE : 3 ILL - DEFINED CHANNEL [Art. 5.11 of the Manual]

Note: Assuming the site is in the Flat Region of Sylhet and the structure is to be constructed bridging up the existing embankment .

Step No.	Start From FLOW CHART	Formulation/Course of Action/Decision
1.	Establish general nature of slope of the Catchment Area	Since the site is situated in Sylhet district i.e Eastern Part of Bangladesh, flash flood occurs which are not affected by water level in major rivers or downstream control and therefore may be related to rainfall intensity and catchment area characteristics as suggested in Art. 5.8.2 of the Manual. So, the natural depression of the catchment area and the water path are established by spot levels making a "contour map". Field informations/records are furnished in the catchment area in Fig. 14.2.
Notes : A. The aerial size of the catchment area in question is 1295 hectres (approx.) B. Rainfall intensity has been taken from One-Hour Rainfall (mm) Map in Bangladesh National Building Code - 1993 (attached as Fig. 14.3) also shown Table 14.1 Rainfall Intensity		
2.	Locate the lowest point of the area nearest to the embankment.	Such a point is located on the Contour Map of the area and marked by 'A' at EL.280.
3.	Is the area high or steep slope?	Yes
4.	Is the area known to experience flash flood after rainfall?	Yes
5.	Can it be confirmed that downstream control dose not affect the flow in the area ?	Yes
6.	Establish catchment area drained by the lowest point 'A'.	From natural depression of the Topography, area in which the whole run-off takes place to the lowest point 'A' is approximately 1294 hectres i.e 1294×10^4

7.	Select suitable method for estimating "rainfall discharge"	Art. 5.2 of the Manual suggests that hydrological considerations for this type of area are related to rainfall intensity and surface drainage characteristics. Hence the catchment run-off method is preferably suitable : Art. 5.8.2 of the Manual .
8.	Establish rainfall intensity and surface drainage characteristics.	A. Rainfall Intensity : It is obtained from One-Hour Rainfall (mm) Map in Bangladesh National Building Code - 1993 (Fig. 14.3). It is observed that the area falls in the Rainfall Region **95 mm** B. Surface layer of Soil : Sandy soil. C. Vegetation : Area covered with heavy brush D. Slope of the ground : Considering points B & A of the Fig. 14.2 : S = 100/ (3*1760*3)x100 = 0.63 % E. P = Factor, depends on the followings :- I. Porosity of soil ii. Area, shape and size of the catchment area. iii. Vegetation cover iv. Surface storage i.e existence of lake marshes etc. v. Initial state of wetness of soil In this case, P = 0.10 Art. 5.8.2 of the Manual : Using the Fig. 5.4 of the Manual for calculating run-off "q" in m^3 / sec for One Hour Rainfall (I_0) of 1 (one) cm and incorporating the values in 8 above we get: **q = 3.15 m^3/ sec** Since the catchment area falls in Rainfall Region of **95 mm** Therefore, Design discharge : **= 3.15* 9.5 ≅ 30 cumecs**

Note : Procedural steps of consulting for determining run-off "q" for One - Hour Rainfall (I_0) of 1 (one) cm are furnished in Art.5.8.2 of the Manual .

<table>
<tr><td>9.</td><td>Is the Discharge greater than 10 cumecs</td><td>Yes</td></tr>
<tr><td colspan="3">Note : It is safe and economic to provide Pipe Culverts if the Design Discharge is below 10 cumecs in view of evacuation of the upstream stored water at such rate that is no danger of abnormal rise in upstream water level.</td></tr>
<tr><td>10.</td><td>Select the suitable structure</td><td>Multiple cells / vents Box Culvert may be provided depending on the field conditions in the main on ground of the following considerations :

A. Hydrological Considerations :

In this case, design has to be based on an increased velocity of flow through the culvert. To create this increased velocity the design must provide for heading up at the inlet end of the culvert. Economy in the Design being the primary consideration, the correct practice is to design Box Culvert on the assumption that water at inlet end may head up to a 'Predetermined Level" above the inlet opening. The surface level of the headed up water at the upstream end has to be so fixed that road bank should not be overstepped, nor any property in the flood plain is damaged..

B. Considerations from Soil Condition :

i . Floors and curtain walls are generally provided in Box Culverts to protect the channel bed from erosion of soil.

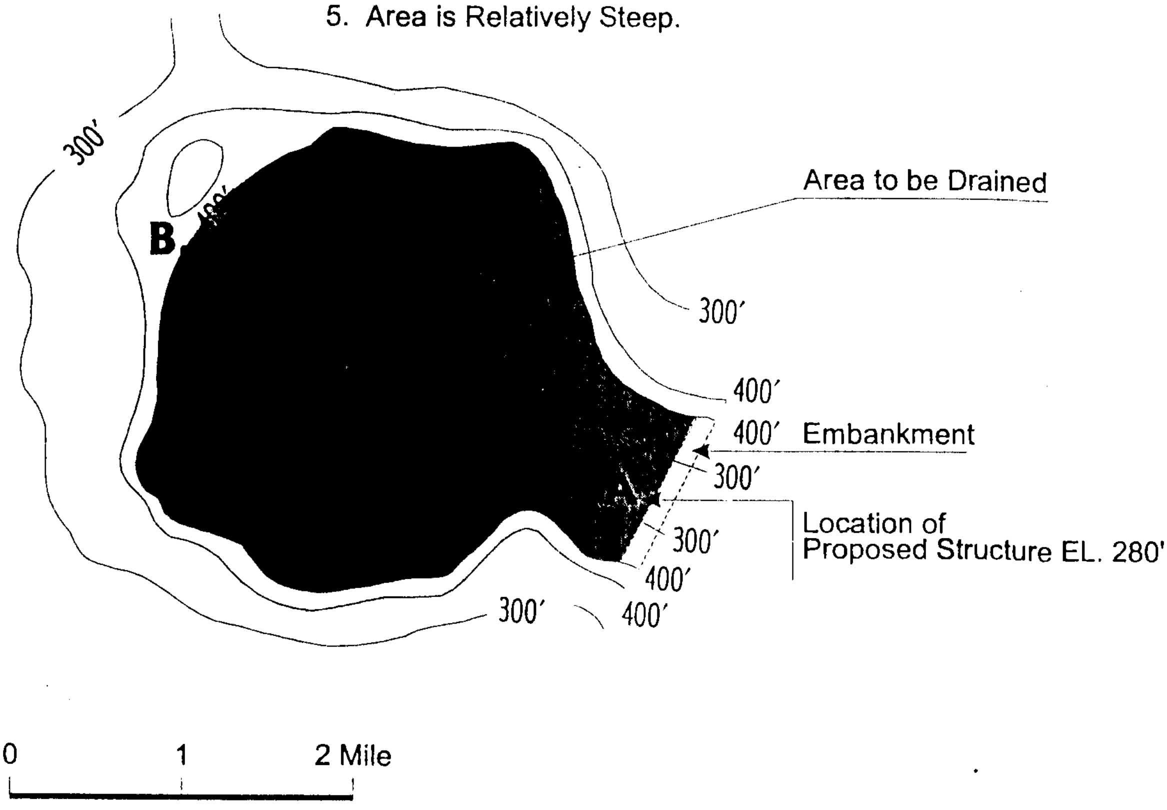
ii. Where the displacement of soil because of installation of foundation is the every possibility and in case of expansive soil which may be very common in the Ill - defined drainage area, curtain wall will control the direction of expansion by allowing the soil to expand into cavities built in the foundations.</td></tr>
<tr><td colspan="3">Note : A. Hydrological considerations : For fixing the "predetermined safe level" field conditions should be adhered to with due attention.
B. Soil Conditions : These conditions are absolutely dependent on the particular site.</td></tr>
</table>

EXAMPLE CASE: 3

Assuming the Region to be Sylhet

Field Observations

1. Sandy Soil, Covered Heavy Brush.
2. The Area Falls in the Railfall Region
3. One Hour Rainfall, 1o = 95 mm According to One-Hour
4. Area of Catchment = 1294 x 10 4 m 2 (Approx.)
5. Area is Relatively Steep.

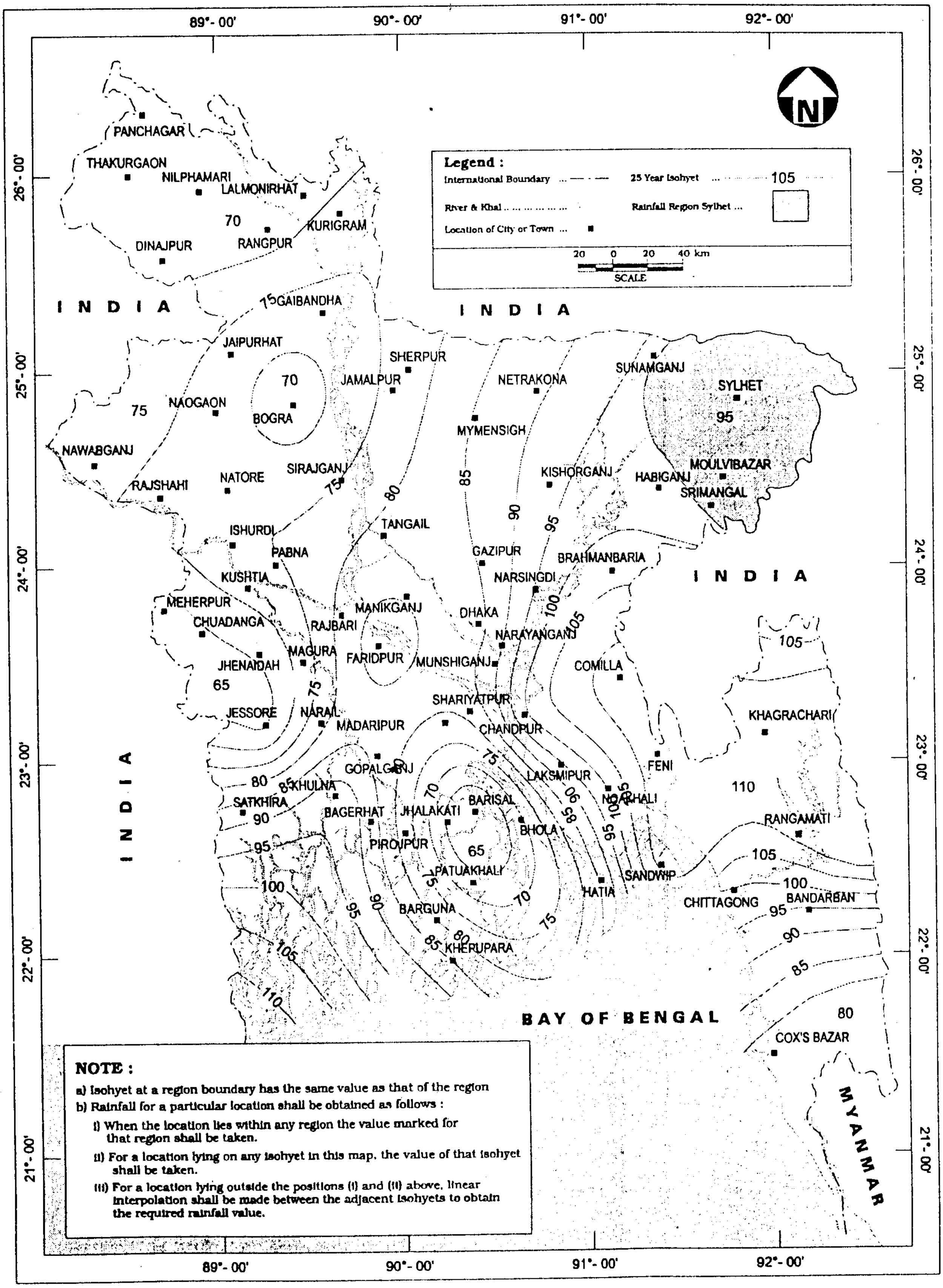


Notes:

1. Spot Levels are Taken and Contour Map is Prepared in "ft" Scale.
2. One-Hour Rainfall (mm) is Attached as Annex-B.

FIG. 14.2 Figure Showing the Field Records (Contour Map) for an Ill-Defined Channel.

Fig. 14.3 One-Hour Rainfall (mm)



* Reproduced From Fig. S1 of the Final Draft of BNBC-1993

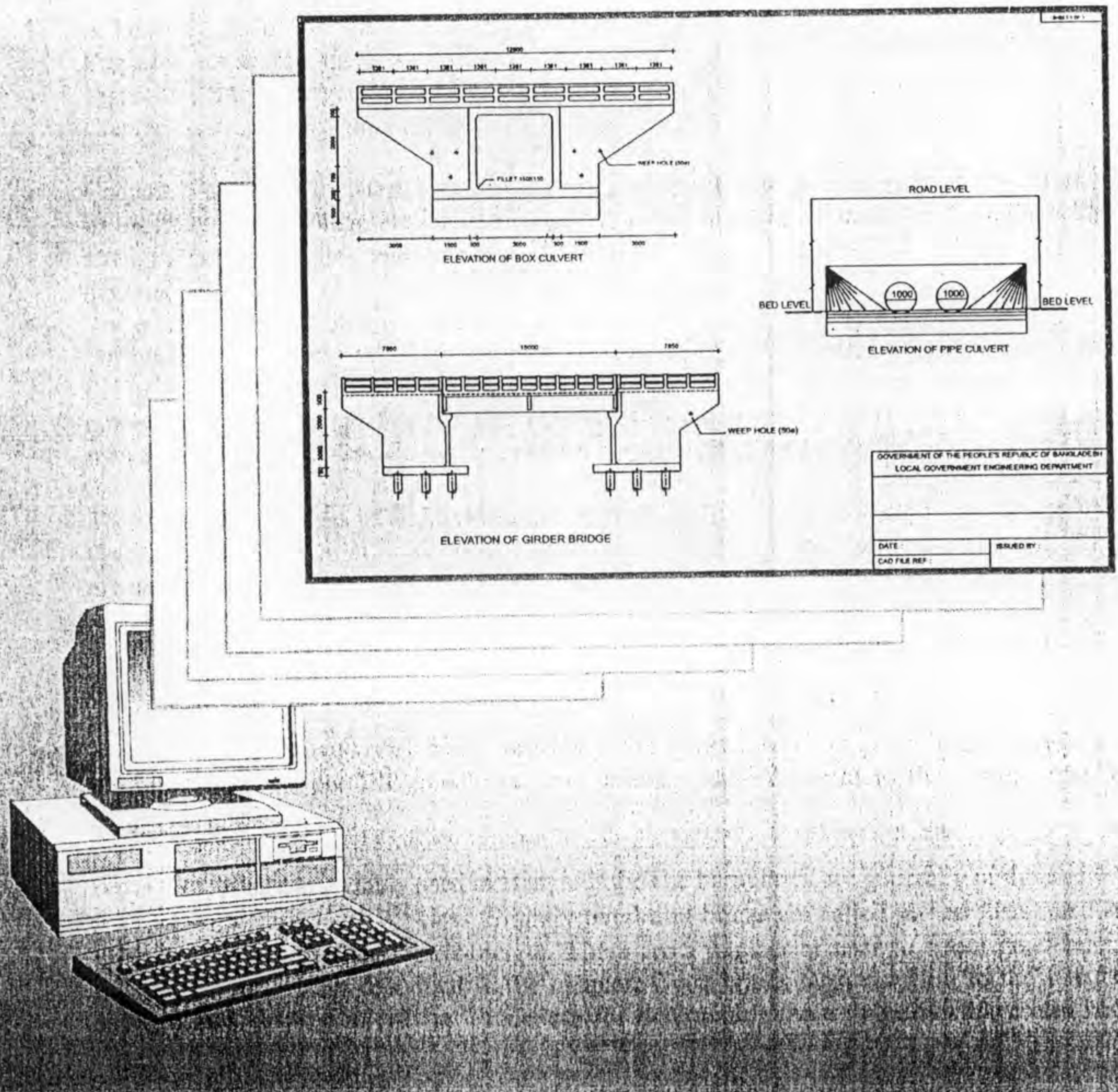
Table: 14.1 Rainfall Intensity

Districts	Rainfall (cm/hour)	Districts	Rainfall (cm/hour)
Bagerhat	8.7	Madaripur	8.4
Bandarban	9.5	Magura	7.3
Barguna	8.0	Manikganj	7.5
Barisal	6.5	Maulibazar	9.5
Bhola	7.0	Meherpur	6.5
Bogra	7.0	Munshiganj	9.0
Brahmanbaria	10.0	Mymensingh	8.5
Chandpur	10.0	Naogaon	7.2
Chittagong	10.0	Narail	7.5
Chuadanga	6.5	Narayanganj	9.0
Comilla	11.0	Natore	7.0
Cox's Bazar	8.0	Nawabganj	7.5
Dhaka	8.5	Netrokona	8.9
Dinajpur	7.0	Nilphamair	7.0
Faridpur	7.5	Noakhali	10.0
Feni	11.0	Norsingdi	8.7
Gaibandha	7.5	Pabna	7.4
Gazipur	8.5	Panchagarh	7.0
Gopalganj	8.4	Patuakhali	7.2
Habigonj	9.5	Pirojpur	7.7
Jamalpur	7.7	Rajbari	7.8
Jessore	6.5	Rajshahi	7.2
Jhalakati	6.5	Rangamati	11.0
Jhenaidaha	6.6	Rangpur	7.0
Joypurhat	7.4	Satkhira	8.7
Khagrachhari	11.0	Shariatpur	8.3
Khulna	8.5	Sherpur	7.8
Kishorganj	9.3	Sirajganj	7.5
Kurigram	7.0	Sunamganj	9.5
Kushtia	7.2	Sylhet	9.5
Lakshimpur	9.5	Tangail	8.1
Lalmonirhat	7.0	Thakurgaon	7.0

SECTION VI MATERIAL SPECIFICATIONS AND BILL OF QUANTITIES

CHAPTER 15: Material Specifications

CHAPTER 16: Bill of Quantities



Material Specifications

In construction of a road structure, various types of materials are used. This chapter deals with different materials used for construction, their specifications as per standard and their properties. The materials mentioned are – cement, reinforcing steel, coarse aggregates, fine aggregates, bricks, water, admixtures and timber.

A knowledge of the standard of these materials and their properties is essential for an Engineer to accept these materials for use in the construction. Specification for coarse aggregate, fine aggregate and their tests for selection specification has been presented in the chapter.

Cement is a sensitive material used in the construction. Knowledge of its properties, duration of use, handling, storage etc. are discussed. Reinforcing steel available in the market, their strength and properties and standard to be used as per AASHTO M31 have also been discussed.

For good concrete mix the Engineer must be sure about the aggregates, then exercise rigidly the proportioning of the mix, care in mixing, consistency in water cement ratio, placing and compacting and curing. For the test of strength of concrete, several methods have been mentioned which may be adopted to confirm the strength of the concrete.

There are several types of additives in the market. The manufacturers claim them to do some functions like fungicidal, water proofing and or accelerating or retarding on concrete. These are not standardised.

Work materials used are wooden boards, sawn timber, bamboo poles, tubular metal poles, steel sheets, etc. These should be of standard dissemble properties.

Stone chips and gravels shall have abrasion value as per AASHTO guidelines and aggregate crushing value as per BS guidelines. The gradings shall conform to the requirements of concerete mix.

Burnt clay bricks are widely used in Bangladesh. The burnt brick are generally of three grades – 1st class, 2nd class and 3rd class. Slightly overburnt bricks are called picked jhama and are used in making aggregates for concrete. There are a number of tests to select good bricks. These tests have been described in the chapter. Good bricks must conform to the standard strength and absorption criteria. Procedure for determining these properties have also been discussed.

CHAPTER 15
Material Specifications

15.1 GENERAL

The effective use of any structural material requires the knowledge of its physical and mechanical properties and the conditions under which the chosen material is to operate in a built structure.

The construction of road structures requires different materials which include cement, reinforcing steel, coarse aggregates, fine aggregates, bricks, water, admixtures and timber. Each material must possess qualities that conform to certain specified standards in order that the resulting construction is sound and of desired quality. The following sections describe very briefly the important properties of common construction materials, tests for determining the properties and methods for obtaining desirable quality of materials. The limiting requirements regarding physical and mechanical properties of materials are also specified in this chapter.

15.2 AGGREGATES

Aggregates are important component of concrete. They give body to the concrete, reduce shrinkage and effect economy. Earlier, aggregates were considered as chemically inert materials but now it has been recognised that some of the aggregates are chemically active and also that certain aggregates exhibit chemical bond at the interface of aggregates and paste. In this respect Neville (1981) mentions that "it is possible to look on aggregate as a building material connected into a cohesive whole by means of the cement paste, in a manner similar to masonry construction. In fact, aggregate is not truly inert and its physical, thermal, and sometimes also chemical properties influence the performance of concrete".

The mere fact that the aggregates occupy at least three-quarters of the volume of concrete, means that they have a significant effect on the various characteristics and properties of concrete. Knowledge of the properties of the aggregates which constitue the majority of the volume of the concrete is essential for an understanding of the concrete. Aggregates are divided into two categories from size consideration :

i) Coarse aggregate and

ii) Fine aggregate

Aggregate larger than 4.75mm is considered as coarse aggregate and aggregate smaller than 4.75mm is considered as fine aggregate. The shape of aggregates is an important characteristic because it affects the workability of the concrete. But it is difficult to really measure the shape of irregular bodies like aggregates which are derived from different rocks (and also from bricks in Bangladesh). Again the crushing method, hand crushing or machine crushing, also influences the shape of aggregates.

15.2.1 Fine Aggregate

Fine aggregate shall conform to the requirements of AASHTO M6-81 (1986). Fine aggregate shall consist of natural sand for Road Structures presented in this Manual.

15.2.1.1 Properties of Sand

Sand shall not have (i) silt or other fine materials more than 5%; and (ii) organic content more than 5%. The fineness modulus (F.M.) of sand shall not be less than 1.8.

When the sand is subject to the five alternations of the sodium sulfate soundness test, the weighted loss shall not exceed 10 mass percent.

Sand shall be well graded from coarse to fine and when tested by means of laboratory sieves shall generally conform to the following requirements (AASHTO M-6) :

Sieve	Mass percent passing
9.50 mm (3/8 in.)	100
4.75 mm (No. 4)	95-100
1.18 mm (No. 16)	45-80
0.300 mm (No. 50)	10-30
0.150 mm (No. 100)	2-10

Sieves 2.36mm (No. 8), 0.600 (No. 30) may be used in addition to those supplied above, to determine F.M. of sand.

15.2.1.2 Fineness Modulus of Sand

The fineness modulus (F.M) of sand is defined as the sum of the cumulative percentages retained by the standard sieves 9.5mm, 4.75mm, 2.36mm, 1.18mm, 0.600mm, 0.300mm and 0.150mm divided by 100. Local river sands in most areas of Bangladesh are fine grained and have F.M. approximately between 1.0 and 1.5, lower than the desirable minimum value of about 1.80 for concrete. Sylhet sands on the other hand are relatively coarse grained and have F.M. approximately between 2.0 and 2.75 (sometimes 3.0). However, Sylhet sands are poorly graded and lack in the fine grains; they are also costly. In order to obtain the desired gradation and reduce the cost of the material, local river sand should be blended with Sylhet sand in a suitable proportion so that the desired minimum F.M. of 1.80 is obtained.

The number of parts, R, of local fine sand to be mixed with 1 part of coarse Sylhet sand, in order to obtain a sand of desired fineness, should be determined by trial mixing. The following approximate formula for R may be used for the first trial :

$$R = \frac{F_c - F}{F - F_f}$$

in which F_c and F_f are the F.M. of coarse and fine sand respectively, and F is the required F.M. of the mix (minimum 1.80).

15.2.1.3 Bulking of Sand

The volume of aggregates, particularly sand, varies with the moisture content of the material. The increase in volume of sand with moisture content is known as bulking. Since in Bangladesh in many construction works proportioning of concrete mixes is usually based on volumetric measurements, it is important that the moisture content considered for the mix design be actually maintained in batching a concrete mix.

Typical bulking curves of sand are shown in Fig. 15.1. The volume of sand increases with increasing moisture content, reaching a maximum volume at about 4 to 6 per cent moisture content. With further increases of moisture content, the volume decreases gradually. The finer the sand, the more pronounced is the bulking phenomenon. The maximum bulking may amount to as much as 40 per cent for a fine sand and 25 per cent for a somewhat coarse sand. However, bulking tests on the particular sand to be used in concrete should be done in the laboratory and the curve showing the effect of moisture on bulking of the particular sand should be drawn instead of depending upon the sample curves shown in Fig. 15.1. In terms of fineness modulus (F.M.) clean river sands of Bangladesh may be somewhat arbitrarily grouped into coarse, medium and fine sand as below :

Kind of Sand	F.M.
Coarse sand	2.00 - 2.75
Medium sand	1.50 - 2.00
Fine sand	1.00 - 1.50

It may be noted that fineness modulus gives an idea of coarseness or fineness of material but gives no idea of grading.

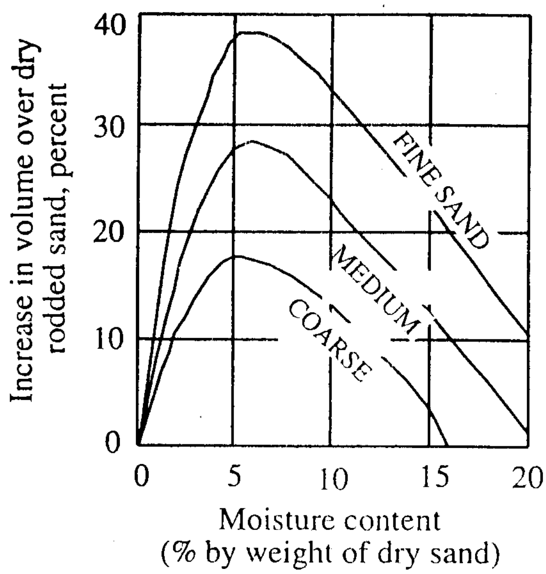


Fig. 15.1 Effect of Moisture on Bulking of Sand

15.2.1.4 Tests on Sand

a) Field Tests

The sand may be subjected to the following field tests in order to check its suitability for being used in mortar or concrete :

i) Rub a little sand between fingers. Stains left on the fingers will indicate the presence of undesirable clay impurities. Accurate mass content of silt and clay (fraction passing 0.075 mm sieve) may be determined by wet sieving in the laboratory.

ii) Stir a sample of sand vigorously in a glass of water and allow it to rest. The content of clay or silt present in it would settle over the sand in a distinct layer.

b) Laboratory Tests

Whenever there is reason to doubt the quality and properties of sand, representative samples of the material should be subjected to laboratory tests before the sand is approved for use. Common laboratory tests of sand are :

i) Sieve analysis for the determination of particle size distribution (AASHTO : T-27).

ii) Determination of clay lump by wet sieving (AASHTO : T-112).

iii) Determination of the total amount of silt, clay and water soluble materials by washing on 0.075 mm sieve (AASHTO : T-11).

iv) Determination of organic impurities (AASHTO : T-21).

15.2.1.5 Corrective Measures

Corrective measures should be taken if results of the tests in section 15.2.1.4 indicate values outside specified limits. Two measures are given below :

i) For obtaining desired gradation and fineness modulus, blending with other good quality samples may be done so that the composite sample conforms to the desired specifications.

ii) For removal of the fine particles and undesirable substances, the fine aggregate should be washed throughly and vigorously prior to use in concrete.

15.3 COARSE AGGREGATE

Coarse aggregate for Portland cement concrete for road structures will be stone chips and gravels which will conform to requirements of AASHTO : M80 and will be well-graded.

15.3.1 Stone Chips

Stone chips and gravels shall have Los Angeles abrasion value as per AASHTO T96-83 (ASTM C131), not more than 40% and aggregate crushing value as per BS: 812 Part 3, not more than 30%. The gradings shall generally conform to the requirements of concrete mix.

15.3.2 Recommended Tests for Coarse Aggregates

The following tests are recommended as routine tests on coarse aggregates :

i) Test for grading (sieve analysis) by AASHTO : T-27 method

ii) Test for silt and clay by AASHTO : T-11 method

iii) Test for total moisture content by any drying method or by AASHTO : T-265 method.

iv) Test for absorption of water on saturated surface dry (SSD) basis by AASHTO : T-85 method which covers determination of specific gravity (sp. gr.) and absorption of coarse aggregate. The specific gravity may be expressed as bulk specific gravity or apparent specific gravity by this method.

v) Test for unit weight and bulk density by AASHTO : T-19 method.

In addition to the above, the following tests are recommended as special tests on coarse aggregates :

i) Test for Los Angeles Abrasion Value by AASHTO:T-96 method.

ii) Test for Aggregate Crushing Value and 10 per cent fines value by BS : 812, Part 3 method.

iii) Soundness of Aggregate by use of Sodium Sulfate or Magnesium Sulfate as per AASHTO : T-104 method.

The tests mentioned as special tests on coarse aggregates should be carried out at least once for each source of supply and subsequently when warranted by changes in the quality of aggregates.

15.4 CEMENT

Ordinary Portland cement is to be used for all concrete and mortar mixes in the construction of road structures. The cement shall conform to the requirements of ASTM C150 or AASHTO M85 satisfying the following requirements:

Setting Time	Gillmore Method	Vicat Method
Initial setting time :	Minimum 60 minutes	Minimum 45 minutes
Final setting time :	Maximum 600 minutes	Maximum 375 minutes

Compressive Strength (Standard Mortar Cube)	
3 days :	12.4 MN/m^2
7 days :	19.3 MN/m^2
28 days :	27.6 MN/m^2

Tensile Strength (Standard Mortar Briquette)			Remarks
3 days	:	1.0 MN/m^2	Tensile strength requirements have not been included in AASHTO : M85 - 84 (14th Edition 1986)
7 days	:	1.9 MN/m^2	
28 days	:	2.4 MN/m^2	

15.4.1 Tests for Cement

a) Field Tests

i) A good cement will retain its shape if a lump is made by hand.

ii) If a sample of good cement is taken in a glass of water it will sink immediately.

iii) The cement will give a feeling of silky smoothness, free from lumps, when rubbed between fingers.

b) Laboratory Tests

The following laboratory tests are recommended :

i) Test for initial and final setting time by AASHTO T131 (ASTM C191) method.

ii) Test for compressive strength by AASHTO T106 (ASTM : C109) method.

iii) Test for tensile strength by AASHTO T132 (ASTM : C190) method, and

iv) Test for fineness by AASHTO T128 (ASTM C184) method.

15.4.2 Storage of Cement

Ordinary Portland cement starts setting as soon as it comes in contact with water or moisture. It is, therefore, very important that it is protected from dampness during storage prior to its use in construction. The following recommendations regarding storage of cement should be followed, as far as practicable :

i) The walls of the store house in which the bags are stored should preferably be of waterproof masonry construction. The roof should be leakproof.

ii) The windows (if there are any) should be few and small, and kept tightly shut preventing external moisture entering the building.

iii) The floor should be dry.

iv) The bags should not be piled against the wall; a space of 300 mm (12 inch) all around should be left between the exterior walls and the cement bags.

v) Cement bags should not be placed directly on the floor to avoid absorption of moisture.

vi) The bags should be placed close together in the pile to reduce circulation of air as much as possible.

vii) The bags should not be piled more than about 15 bags high. The width of pile should be less than about 3m (10 ft). The bags may be arranged in header and stretcher fashion to avoid collapse of the stack.

viii) For extra safety, particularly during monsoon, the pile of cement bags should be enclosed completely in a polyethylene sheet or covered with a tarpaulin.

ix) Each consignment should be stacked separately so that the older cement may be identified readily and used earlier.

If storage conditions are not satisfactory, cement stored for long periods is generally adversely affected. Table 15.1 gives a general idea of the reduction in strength of concrete made with stored cement.

Table 15.1 Reduction in Strength of Concrete Made with Stored Cement

Period of Storage	Approximate Relative Strength at 28 Days (percent)
Fresh	100
3 months	80
6 months	70
1 year	60
2 years	50

15.5 REINFORCEMENT

Reinforcing steel deformed bars for concrete reinforcement shall conform to AASHTO M31 (ASTM A615). Reinforcing steel having minimum yield strength 275 MN/m^2 shall be used. Specifications of reinforcing steel as per AASHTO M31-86 (ASTM A615-84a) are shown in Table 15.2, 15.3 and 15.4.

Reinforcing bars generally available in the market are made either from steel billets or from scrap steel. Bars made from scrap steel may some times exhibit lower strength than that of bars made from billet. Actual diameters of many bars available in the market are less than their stated diameters. Care must, therefore, be exercised in procuring reinforcing steel from local markets.

Table 15.2 : Deformed Bar Designation Numbers, Nominal Masses, Nominal Dimensions, and Deformation Requirements

Bar Designation No. B/	Nominal Mass, kg/m	Nominal Dimensions A/			Deformation Requirements, mm		
		Diameter, mm	Cross Sectional Area mm^2	Perimeter mm	Maximum Average Spacing	Minimum Average Height	Maximum Gap (Chord of 12.5% of Nominal Perimeter)
10	0.785	11.3	100	35.5	7.9	0.45	4.4
15	1.570	16.0	200	50.3	11.2	0.72	6.3
20	2.355	19.5	300	61.3	13.6	0.98	7.7
25	3.925	25.2	500	79.2	17.6	1.26	9.9
30	5.495	29.9	700	93.9	20.9	1.48	11.7
35	7.850	35.7	1000	112.2	25.0	1.79	14.0
45	11.775	43.7	1500	137.3	30.6	2.20	17.2
55	19.625	56.4	2500	177.2	39.4	2.55	22.2

A/ The nominal dimensions of a deformed bar are equivalent to those of a plain round bar having the same mass per meter as the deformed bar.

B/ Bar designation numbers approximate the number of millimeters of the nominal diameter of the bar.

(Ref. Table 1 : AASHTO M31-86)

Table 15.3 : Tensile Requirements of Reinforcement

	Grade 300 A/	Grade 400
Tensile strength, min, MPa	500	600
Yield strength, min, MPa	300	400
Elongation in 200 mm, min, % :		
Bar No.		
10	11	9
15, 20	12	9
25	-	8
30	-	7
35	-	7
45, 55	-	7

A/ Grade 300 bars are furnished only in sizes 10 through 20. (Ref. Table 2 : AASHTO M31-86)

Table 15.4 : Bend Test Requirements A/

Bar Designation No.	Pin Diameter for Bend Tests d = nominal diameter of specimen	
	Grade 300	Grade 400
10,15	$4d^b$	4d
20	$5d^b$	5d
25	-	6d
30,35	-	8d

A/ Test bends 180 deg.
d^b = nominal diameter of specimen
(Ref. Table 3 : AASHTO M31-86)

15.6 CONCRETE

Fresh concrete or plastic concrete is a freshly mixed material which can be moulded into any shape. The relative quantities of cement, aggregates and water mixed together control the properties in the wet state as well as in the hardened state.

Concrete mixes must fulfill the following requirements :

i) When freshly mixed, the mass should be workable.

ii) When the mass has hardened, it should posess strength and durability adequate for the purpose for which it is intended.

iii) Cost of the final product should be a minimum, consistent with acceptable quality.

15.6.1 Factors Affecting Quality of Concrete

The properties of concrete depend on the following factors :

i) Proportion of the ingredients,

ii) Gradation and properties of aggregates,

iii) Consistency (water-cement ratio) of the mix at the time of placing,

iv) Efficiency of mixing, placing and compacting, and

v) Curing.

15.6.2 Mix Design

Proportioning either by weight or by volume may be used depending on the facilities available. However, the volumetric proportioning is more common in this country.

The selected strength of concrete may be achieved with the following mix ratio by volume:

For precast and cast-in-situ piles the ratio between Cement : Fine aggregates: Coarse aggregates will be 1 : 1.5 : 3.

For other structures it will be 1 : 2 : 4.

If the above mentioned mix ratios do not satisfy the strength requirements, a richer mix should be used. Trial design mixes should be made to find the required mix ratio for the minimum strength requirement.

The water cement ratios shall preferably be within the range of 0.55 to 0.45. For concrete in or over saltwater the maximum water cement ratio shall be 0.45. The water-cement ratio shall be adjusted by trials as necessary to produce concrete of required strength with the consistencies listed in Table 15.5.

Table 15.5 : Type of work and Required Consistencies

Type of Work		Nominal Slump (mm)	Maximum Slump (mm)
1.	Formed elements :		
	Section over 300 mm thick	25 - 75	125
	Section 300 mm thick or less	25 - 100	125
2.	Cast-in-place pile	125 - 200	225
3.	Concrete placed under water	125 - 200	225
(Reference : Table 8.3, P-481, AASHTO - 1992)			

15.6.3 Water-Cement Ratio

In engineering practice, the strength of concrete at a given age and cured at a prescribed temperature is assumed to depend primarily on two factors only : the water-cement ratio and the degree of compaction. When concrete is fully compacted its strength is taken to be inversely proportional to the water-cement ratio (Neville, 1981). Both the durability and the compressive strength of concrete made of a given concrete mix increase with a decrease in the water-cement ratio. However, the use of less water than that required for the desired workability would result in porous and weak concrete. Excess water, on the other hand, may result in bleeding or segregation of concrete. For a non-vibrated concrete, water-cement ratio of about 0.55 with hand mixing and about 0.45 with machine mixing may be used. This corresponds to about 22 to 30 litres (5 to 6.5 gallons) of water for a 50kg (112 lb) bag of cement. Water to be used in concrete shall be reasonably clear, free from oil, acid, alkalies, salt and organic matters and shall be of comparable quality with drinking water. A decrease in more than 10% mortar strength resulting from the use of a water when compared with similar mortar made with distilled water, will make the water unfit for use.

15.6.4 Slump Test

The consistency of fresh concrete to permit proper handling and placing of the mixture is usually determined by the slump test (ASTM C143). The procedure of performing the slump test is briefly summarised below.

A standard galvanised metal mould which is 200mm (8 in.) in diameter at the base, 100mm (4 in.) in diameter at the top and 300mm (12 in.) in height is placed on a non-absorptive surface.

The sample needs to be representative of the batch. For concrete samples containing aggregates larger than 38mm (1.5 inches) in size, pieces of aggregate exceeding 38mm (1.5 inches) are to be removed by wet screening. The mould is to be moistened and placed on a flat, moist, non-absorptive surface, where the operator is to hold it firmly in place by standing on the foot pieces of the mould while it is being filled with concrete. The mould is filled in three layers, each approximately one-third of the volume of the mould. The vertical dimensions from the bottom of the mould to the tops of the layers should be approximately 63mm (2.5 inches), 150mm (6 inches) and 300mm (12 inches). In placing each scoopful of concrete, the scoop is to be moved around the top edge of the mould as concrete slides from it to insure symmetrical distribution of concrete within the mould. Each layer is to be rodded with 25 strokes of a 16mm (0.625 inch) rod, 600mm (24 inches) length and hemispherically tipped at the lower end. The strokes are to be distributed uniformly over the cross section of the mould and should just penetrate into the underlying layer. The bottom layer should be rodded throughout its depth. After the top layer has been rodded, the surface of the concrete is struck off so that the mould is exactly filled and the spilled concrete is cleared from the surface. The mould is immediately removed from the concrete by raising it slowly and carefully in a vertical direction. The slump should be measured to the nearest 6mm (one fourth of an inch) immediately thereafter by determining the difference between the height of the mould and the average height of the top surface of the concrete after subsidence.

Slump specimens which break or slough off laterally give incorrect results and need to be retested with a fresh sample. After the slump measurement has been completed the side of the concrete frustum is to be tapped gently with the tamping rod. The behaviour of the concrete under this treatment is a valuable indication of its cohesiveness, workability and placeability. Fig. 15.2 shows the slump testing apparatus.

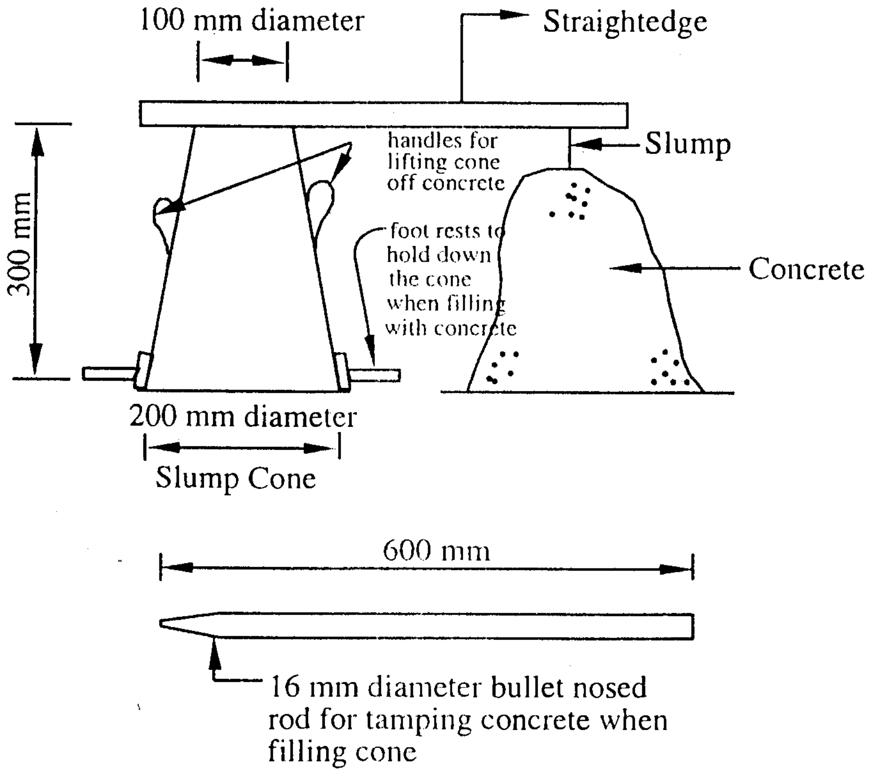


Fig. 15.2 Slump Test Apparates

A well-proportioned, workable mix will slump gradually to lower elevations and retain its original identity; however, a poor mix will crumble, segregate, and fall apart (USDI Concrete Manual, 1990).

Ranges of nominal slump and maximum slump are recommended for the concrete mix to be used for road structures in Table 15.5. In general, the lowest possible slump that permits adequate workability in a given casting should be used.

15.6.5 Admixtures

Often, instead of using a special cement, it is possible to change some of the properties of the cement in hand by the use of a suitable additive. A great number of proprietary products is available: their effects are described by the manufacturers but the full details of the action of many of these additives, known as admixtures, are yet to be determined, and the performance of any one admixture should be carefully checked before it is used. Admixtures may be classified according to the purpose for which they are used in concrete; the approach of ASTM Standard C494-79 can be used. There are different admixtures, namely, accelerating admixture, retarding admixture, or retarder, water-reducing admixture, and air-entrainment additive. There are also additives for other purposes, such as air dètrainment, fungicidal action, water-proofing, etc., but these are not sufficiently standardized. (Neville, 1981).

The only admixture that is commonly used in Bangladesh is a permeability reducing agent known as "pudlo", a white powdered like substance. It is used in a proportion of 1 to 2 percent of cement, in mortars for plastering, and in concrete expected to retain water.

In recent years for high strength concrete in some river bridge constructions imported brands of plasticizers were used in Bangladesh.

15.6.6 Materials for Formwork

Common materials used in formwork for concrete are listed below together with their desirable properties :

i) Wooden boards : Usually wooden boards of 25 to 40 mm (1 to 1.5 in.) thickness are used in formwork depending on the nature of the structure. Such boards should be free of cracks and large holes that will allow cement mortar and water to flow out.

ii) Sawn timber: These are used in conjunction with wooden boards as framing, shoring and bracing for the formwork.

iii) Bamboo poles: These are generally used as props to support the formwork of beams and floors. Bamboo poles are also widely used for building scaffoldings for construction.

iv) Balli poles: Timber balli poles, sal or garjan, 150 to 200 mm (6 to 8 in.) in diameter are sometimes used as props where construction loads are heavy or the structures are high.

v) Tubular metal poles : Tubular metal poles of various standard sizes may be used. However, because of the large initial investment necessary, few construction contractors use them in Bangladesh.

vi) Steel sheets : Steel sheets of thickness varying between 24 and 26 gauge may be used as a facing material for wooden boards where superior surface finish of concrete is required.

15.7 BRICKS

Burnt clay bricks are widely used in Bangladesh for two distinct construction purposes; as building blocks in walls, columns, pavements etc. and for the preparation of aggregates for concrete. Bricks are generally manufactured by hand moulding of soft clay and burnt in intermittent kilns. However, in large cities machine made bricks manufactured by the extrusion process may be available.

Bricks manufactured in the same lot and burnt in the same batch may vary in quality and strength. In local terminology, these are graded as 1st class, 2nd class and 3rd class. In addition, slightly overburnt bricks are called picked jhama and are commonly used for making aggregates for concrete.

15.7.1 Characteristics of Good Building Bricks

The following features characterize good building bricks;

i) Uniform colour, shape and size

ii) No distortion and cracks

iii) Good strength and durability

15.7.2 Field Tests for Good Bricks

i) When a brick is struck with another, it should give a clear metallic sound and should not break easily.

ii) It is not possible to indent a good brick by scratching with finger nail.

iii) A good brick does not break when dropped from a height of 1.2m (4 ft.) on a hard floor.

iv) Surfaces of good bricks should be smooth having square edges, and free from cracks and voids.

v) Colour, shape, size and structure of bricks should be uniform, and

vi) When two bricks are formed into a 'T' and dropped from a height of 1.2m (4 ft.), they should not break.

15.7.3 Bangladesh Standard for Bricks, BDS 208: 1980

BDS 208 classifies common burnt clay bricks into three grades - A, B and C. The classification is based on properties such as crushing strength, water absorption, efflorescence, workmanship and appearance,. The standard dimension of a brick is specified as 240 x 115 x 70 mm (9.5 x 4.5 x 2.75 in.). Methods of tests for evaluating the various properties are detailed in the standard. Sampling methods and acceptability requirements of brick lots are also specified.

Tests for compressive strength are to be performed on twelve halved brick specimens capped with cement-sand mortar. The test is performed under an axial load applied at a uniform rate of 13.8 MN/m^2 (2000 psi) per minute. Tests for water absorption are to be performed on six whole brick specimens by immersing them in water at room temperature for 24 hours. The limiting values of crushing strength and water absorption for various grades of bricks are given in Table 15.6.

Table 15.6 : Limiting Values of Crushing Strength and Absorption (BDS 208: 1980)

Grade	Minimum Crushing Strength, MN/m^2 (psi)		Maximum Water Absorption, Percent
	Mean of 12 specimens	Individual brick	
A	27.6 (4000)	20.7 (3000)	12
B	17.2 (2500)	13.8 (2000)	12
C	10.3 (1500)	08.3 (1200)	16

Profuse efflorescence which is the result of deposition of salts on brick surface, deteriorates the appearance and strength of structures. BDS 208 gives simple methods of testing for efflorescence in the laboratory and in the field. The degree of efflorescence is, however, denoted only qualitatively, such as 'nil', 'slight', 'moderate', 'heavy' and 'serious' For masonry construction, bricks with 'nil' or 'slight' efflorescence only should be selected.

15.7.4 Recommended Strength and Absorption Criteria

Experience indicates that it is difficult to achieve at field the quality as specified for grade A bricks in BDS 208: 1980 for manually made common building clay bricks. Again the 24 hour absorption of most bricks having satisfactory crushing strength exceeds 12 per cent, the upper limit recommended by the above standard for grades A and B bricks. Considering the above difficulties and recognising that most of the locally produced bricks are still manually made, the following criteria as regards crushing strength and water absorption are recommended for the construction of road structures:

i) The minimum average crushing strength of bricks for masonry work shall not be less than 17.2 MN/m^2 (2500 psi) for 12 halved brick specimens and the minimum for individual brick shall be 13.8 MN/m^2 (2000 psi). This corresponds to strength requirement of grade B bricks.

ii) The average crushing strength of picked jhama bricks (halved bricks, 12 specimens) shall not be less than 20.7 MN/m^2 (3000 psi). The minimum for individual brick shall not be less than 17.2 MN/m^2 (2500 psi).

iii) The 24 hours water absorption of bricks satisfying the strength requirements should not exceed 15 per cent by weight.

In summary it is mentioned that the following shall be the major criteria of bricks (BDS 208 : 1980) for construction of road structures covered in this Manual.

Type of Test Crushing Strength (halved bricks of 12 brick samples)	Crushing Strength
Mean of 12 bricks	17.2 MN/m^2 (2500 psi)
Minimum for individual brick	13.8 MN/m^2 (2000 psi)
Water absorption	15% maximum
Efflorescence	Slight or nil
Standard dimension	240 mm x 115 mm x 70 mm
Variation in dimension	6 mm in length
	5 mm in breadth
	1.5 mm in height

15.7.5 Reference

1. American Association of State Highway and Transportation Officials (AASHTO), Part 1 and Part 2 (1986).

2. American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (Fifth Edition, 1992).

3. American Society for Testing and Materials (ASTM).

4. Concrete Manual, U.S. Department of Interior Water and Power Resources Service, 8th Edition,1990, Published by S.K. Jain, Delhi-32.

5. Neville, A.M. (1981), Properties of Concrete, Third Edition, Pitman Publishing Press, London.

6. Road Structures Manual of LGEB, 1989.

Summary 16

Bill of Quantities

A Bill of Quantities is a contractual document that contains the standard specifications for items of work, their estimated quantities, unit rates and total cost of a road structure construction project. It provides a basis for tendering. This chapter contains the standard specification of items of works related to the construction of road structures and describes the steps to be followed to estimate the cost of construction.

CHAPTER 16
Bill Of Quantities

16.1 GENERAL

The Bill of Quantities is a contractual document that contains the standard specifications of items of work, their estimated quantities, unit rates and total cost. The Bill of Quantities provides a basis for tendering. Because most contracts are awarded as a result of competitive tendering and having a Bill of Quantities common to all tenderers ensures comparability of tenders. A Bill of Quantities allows preparation of a cost estimate prior to construction of the project. The actual cost will not, however, be known until the project is completed. The actual cost should not be far from the estimated cost. This chapter contains the standard specification of items of works related to the construction of road structures and describes the steps to be followed to estimate the cost of construction.

16.2 COST ESTIMATE

The following steps are adopted for the preparation of a cost estimate for a road structure project.

i) After selecting the structure type and size of road structure as furnished in Chapter 6, Section V, the estimated quantities for each item of work for the selected structure have been extracted from the corresponding table.

ii) The Schedule of Items of work to be involved for any type of road structure has been prepared and presented in Table 16.1. Additional items of work are to be identified, which are relevant to the selected structure but not included in the tabulated quantity estimates of standard design, quantity estimate for those additional items considering site conditions has also been prepared.

iii) Using the estimated quantities for various items and the corresponding approved rates of LGED, the total cost of those items has been computed. Cost for mobilization, demobilization and clearance of the site have been included as items of work.

iv) Summing up the cost of different items to arrive at the total cost of the project, 5 percent of the estimated cost has to be added as contingency to meet the cost of unforeseen items or other miscellaneous expenditure which do not fall under any item of work.

v) ±10% deviation may be acceptable over the stipulated cost estimate.

Table 16.1
Schedule of Items

Road : **District :**

Structure No. : **Location :**

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
1.	Mobilization of men, materials and equipment, preparation of site, clearing of all existing debris, jungles and obstruction, construction of appurtenances, etc. for commencing the work as per direction of the Engineer.	L.S.			
2.	Providing and maintaining the Engineers' office (during construction), a temporary shed (3mx3mx2.6m) at site made of C.I. sheet roofing with bullah/ bamboo supports and tarja walls, floor with single brick flat soling at a minimum 150mm high plinth level from G.L., with minimum one door (2mx0.8m) and one window (1.22mx0.9m) with wooden shutters including furnishing of three wooden chairs and one wooden table (1.22mx0.9m) as per direction of the Engineer.	L.S.			
3.	Providing a diversion with approaches at a place away from the construction site for the movement of pedestrian/ existing traffic including cost of all materials, labour, equipment and maintaining the same till the newly constructed bridge/culvert is open to traffic, as directed by the Engineer.	L.S.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
4.	Dismantling of existing structures of any type including removal of foundations (down to the required depth) and superstructure, stacking the removed debris and materials in a safe place, supply of all labour, equipment, taking all precautionary measures, etc. complete as per direction of the Engineer. (The salvaged materials are Govt. property).	L.S.			
5.	Making earthen ring/ cross bundh of required height and width to prevent water from entering in the working area for any type of foundation with earth arranged and carried by the contractor including bullah/ bamboo palisading and double tarja mat/ drum sheets walling as and where necessary and maintaining the same till completion of work for which the ring/ cross bundh is made including supply of all materials, labour and equipment and removal after completion of work as per direction of the Engineer. (70% of payment will be made after completion of the construction of work. The balance amount will be paid after full removal of the same).	Each			
6.	(2.2.1) Earth work in excavation of foundation trenches in all kinds of soil upto the required depth and width, providing the layout, construction of centre line brick/concrete reference pillars, preparation of bed, bailing out of water, removal of spoils to a safe				

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	distance, back filling of sides of trenches upto original level, shoring if necessary, etc. complete as per drawings and directions of the Engineer.	Cu.m.			
7.	Bailing out water/ de-watering of working areas, including for supply, operation, and maintenance of 2 nos. water pumps of not less than 60 l/s capacity each, for bailing out of water as required. Pumps to be present on site for all work carried out below prevailing ground level and between 1st April and 30th November each year.	L.S.			
8.	Sand filling under culvert base or in any other foundation in layers with sand (FM 0.80) free from dust and impurities, including cost of all materials, watering, ramming, consolidating, dressing etc. complete in all respects as per drawing and directions of the Engineer.	Cu.m.			
9.	(5.1) Single layer brick flat soling in foundation & other places designated by the Engineer with first class bricks, filling the joints with sand (FM 0.80), @ 0.012 cum of sand per sqm over 75mm thick sand cushion, watering, levelling, dressing including cost of all materials, labour etc. complete as per drawings and direction of the Engineer.	Sq.m.			
10.	(3.14) Brick on edge pavement in single layer of Herring bone bond for side walk and where necessary as directed by Engineer with 1st class or picked jhama bricks over 25mm thick				

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	sand (minimum F.M- 0.80)cushion including carrying bricks, filling the interstices with sand (minimum F.M- 0.80) @ 0.12m^3 per m^2 , of road surface etc. all complete as per drawing and direction of the Engineer.	Sq.m.			
11.	(4.01) Cement concrete work in proportion (1:3:6) in foundations with cement, coarse sand (FM 1.80) and 20mm down graded stone chips including supplying of all materials, labour and equipment, curing etc. all complete as per drawings, specifications and directions of the Engineer.	Cu.m.			
12.	(4.03) First class brick work in cement mortar (1:3) with Portland cement sand (minimum FM 1.50) upto ground level (GL) in abutment/ wing wall/ retaining wall/ headwall etc. including raking out joint, curing etc. all complete, including immersing of bricks in water before use of bricks, including carrying, supply and cost of all materials and labour etc. all complete as per direction of the Engineer.	Cu.m.			
13.	(4.04) Same as item no. 12 and in cement mortar (1:3) with Portland cement and sand (minimum FM 1.5) and in special cases where side shoring is necessary including supply, carrying and cost of all materials and labour for side shoring as per direction of the Engineer.	Cu.m.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
14.	(4.05) First class brick work in cement mortar (1:4) with Portland cement and sand (minimum FM 1.50) above GL in abutment/wing wall/retaining wall/ head wall etc. including raking out joint, curing etc. all complete, including immersing of bricks for at least 6 (six) hours in supply carrying and cost of all materials and labour including scaffolding if necessary etc. all complete as per direction of the Engineer.	Cu.m.			
15.	(4.30) Supply and fabrication of M.S. deformed/ high yield deformed bar for all types of R.C.C works as per drawings including hooking, bending, binding with 22 BWG G.I wire supported or spaced by mortar blocks (1:3) of cement and sand or metal chair, separators and hangers, lapping including supplying, carrying and cost of all materials and straightening removing rust if any cranking labours etc. all complete as per direction of the Engineer. (Measurement will be based on standard weight of 490 lb/ cft Chairs, laps and separators will not be measured for payment. The cost of these will be included in the unit rate.)				
	a) M S Deformed grade 40 bar, fy = 276 Mpa (40000 Psi)	Kg.			
	b) High yield deformed/ Twisted grade 60 bar fy = 414 Mpa (60000 Psi)	Kg.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
16.	(4.17) Supply and install metal shoes of approved designed for RC pre-cast piles with steel plates, angles, MS bars, etc. welded to proper size and shape with necessary arrangement for embedding in pre-cast concrete piles including cost of all materials and labour etc. complete as per drawing, specifications and directions of the Engineer.	Each			
17.	Mobilization for pre-cast RCC test piles for driving with pile driving set winch etc. upto the desired & required depth and length to determine the length of the service piles etc. complete as per direction of the Engineer. (The driving cost of the test pile is not included in this item and will be paid as per rate of the service piles).	L.S			
18.	(4.26) Driving pre-cast RC piles down to required depth in all types of soil by approved means using derrick and mechanical winch, handling and keeping the piles in position, use of water jetting system where necessary with suitable jetting equipment, fitting and fixing steel caps etc. complete as per drawings and directions of the Engineer. (Payment will be made for length of pile driven in soil from the bottom of the pile cap to top of the pile shoe).				
	a) Size 300mm x 300mm				
	i) 0 to 5m depth	L.m.			
	ii) 5m to above	L.m.			
	b) Size 350mm x 350mm				
	i) 0 to 5m depth	L.m.			
	ii) 5m to above	L.m.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
19.	Boring and casting of reinforced concrete cast-in-situ piles upto bottom of pile cap and or top of permanent/ temporary steel casing if required, including necessary arrangement for staging, drilling, driving bentonite circulation, placing of reinforcement and casting concrete with cement, sand (FM 1.8)and 20mm down graded crushed stone chips in proportion not less than 1:1.5:3 and a minimum ultimate cylinder crushing strength of concrete at 28 days will be 21 Mpa (3000 psi) (excluding cost of reinforcement and its fabrication / permanent steel casing). Providing temporary steel casing upto required depth and pulling it out as per standard practice etc. complete in all respects as per drawings, specifications & directions of the Engineer. Extra cement if required to attain the strength to be provided by the contractor at his own cost.				
	a) Boring				
	i) 600mm dia	L.m.			
	ii) 500mm dia	L.m.			
	b) Concreting				
	i) 600mm dia	Cu.m			
	ii) 500mm dia	Cu.m			
20.	Breaking the head of cast-in-situ bored pile/ pre-cast pile upto required length by any means and removing the dismantled materials such as concrete to a safe distance including scrapping				

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	and removing concrete from MS rods, bending for anchorage in pile cap, carrying, all sorts of handling, stacking the same properly after clearing, levelling and dressing the site and clearing the river bed etc. complete as per direction of the Engineer-in-Charge. (Measurement will be given for the actual pile head volume to be broken).	Cu.m			
21	Earthwork in temporary coffer dams/ artificial island for convenience of driving pre-cast piles and drilling operation and concreting for cast-in-situ bored piles in all types of soil, including cutting, carrying, throwing and filling in layers not exceeding 150mm in thickness, with all leads & lifts and including close bamboo palasiding with matured barrack bamboo of minimum diameter of 75mm including driving at least one thired of its length below G.L including carrying of all materials, fitting fixing with gazal, nail, battens at 0.5m c/c etc. with double tarza walling including fixing with G.I wire etc. all complete as per direction of the Engineer (materials to be obtained from excavation of temporary diversion channel/ foundation excavation etc. as directed; coffer dams to be subsequently removed by placing as embankment or other parmanent fill) 70% of payment will be made after the construction of artificial island/				

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	coffer dam. The balance amount will be paid after full removal of the same. If barge is used instead of artificial island same payment will be made .	Nos.			
22.	Supplying best quality steel pipes for bored pile casing of 6mm thickness and diameter as per drawings, carrying, handling, fitting, fixing in position, etc. complete as per specifications and direction of the Engineer. a) 600mm dia b) 500mm dia	 L.m L.m			
23.	Static load test of cast-in-situ/ pre-cast concrete piles by the application of superimposed loads on the pile head, preparation of all arrangements including staging, supplying loads with approved means and keeping the full load in place for 24 hours minimum, measuring of settlements by calibrated gauges and subsequent removal of loads, staging and other temporary works, etc. complete. The test load shall not be less than double the design load of the pile and the applied load shall not be removed before 24 hours or until the rate of settlement is less than 0.25mm (0.01 inch) per half an hour, or as directed by the Engineer (in case of pre-cast piles, the item will be provisional and will be executed only as per direction of the Engineer). Applied Load : a) Upto 60 Ton b) Above 60 Ton	 Each Each			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
24.	(4.10) Reinforced Cement Concrete works (1:2:4) with Portland cement, coarse sand (minimum FM 1.80) and 20mm downgraded crushed stone chips including shuttering (watertight), mixing with mixer machine, casting compacting with vibrator, curing etc. all complete including breaking of chips and cost of all materials their carrying & labour but excluding cost of reinforcement & its fabrication etc. complete as per specifications, drawings and direction of the Engineer (no plastering to be allowed over RCC work, ultimate cylinder crushing strength of concrete at 28 days shall be 21 Mpa (3000 psi), Extra cement, if required to attain the strength, to be provided by the contractor at his own cost)				
	a) [4.10.2 (ii)] Foundation and footing.	Cu.m.			
	b) [4.10.2 (iii)] Bottom slab of Box Culverts	Cu.m.			
	c) [4.10.2 (iii)] Apron of Culvert	Cu.m.			
	d) [4.10.2 (iv)] Vertical members (Box wall with fillet)	Cu.m.			
	e) [4.10.2 (iv)] Drop wall of box culvert (cut-off wall)	Cu.m.			
	f) [4.10.2 (v)] Top slab of box and masonry Culvert	Cu.m.			
	g) [4.10.2 (vii)] Pile cap, bed block, wheel guard/ curb, side walk/ foot-path.	Cu.m.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	h) [4.10.2 (viii)] Wing wall, retaining wall,abutment, column, pier, pier cap, extended bored pile above casing, tie beam, breast walls etc.	Cu.m.			
	i) [4.10.2 (ix)] Girders, cross girder of bridge upto 10m span.	Cu.m.			
	j) [4.10.2 (x)]Girder, cross girders of bridge beyond 10m span. (Girder will be measured upto the bottom of deck slab)	Cu.m.			
	k) [4.10.2 (xi)] Deck slab of bridge upto 10m span	Cu.m.			
	l) [4.10.2 (xii)] Deck slab of bridge beyond 10m span	Cu.m.			
	m) [4.10.2 (xiii)]Railing & Rail post	Cu.m.			
24.1	[4.10.1] Same as item no. 24 but with brick chips for U-type culvert top, bottom slab & drop wall as per drawing & direction of the Engineer.	Cu.m			
	i) [4.10.1 (ii)] Bottom slab	Cu.m			
	b) [4.10.1 (iv)] Drop Wall	Cu.m			
	ii) [4.10.1 (v)] Top slab	Cu.m			
25.	(4.11) Same as item no. 24 but with the mix minimum of 1:1.5:3.0 in proportion using coarse sand of F.M-2.50 and achieving cylinder crushing strength 24 Mpa (3500 psi) for pre. Cast R.C.C piles.	Cu.m.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
26.	(4.42) Providing wearing course (1:1.5:3.0) on deck slab of bridge or culvert with cement, sand (Minimum F.M-1.8) and 6mm down graded boulder chips including breaking chips, mixing concrete, laying, compacting and curing including cost of all materials, labour and transportation to the site etc. all complete as per direction of the Engineer.	Cu.m.			
27.	Providing weep holes in abutments, wing walls and spouts in deck slab with required dia PVC pipe including supplying, cutting to proper sizes placing them in proper position in the formwork and binding with GI wire and keeping in position during concreting including iron grating spouts as per drawings, and directions of the Engineer. a) Weep holes 50mm dia b) Spout 75mm dia	 L.m. L.m.			
28.	Supplying, fitting and fixing bearings in exact position as per drawings, specification and direction of the Engineer including cost of all materials, labour, welding, carrying, testing by BUET etc. all complete. (The set shall mean all holding down bolts, dowel bars, rubber bearing consisting of one or more vulcanised natural rubber pads bonded to metal plates to form a Sandwich arrangement, etc. to complete the support of a girder at each end).				

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	a) Elastomeric/ Neoprene bearing size as follows:				
	i) 300 x 250 x 52mm	Set			
	ii) 300 x 200 x 46mm	Set			
	iii) 300 x 225 x 39mm	Set			
	iv) 300 x 200 x 37mm	Set			
	v) 250 x 225 x 52mm	Set			
	vi) 250 x 175 x 46mm	Set			
	vii) 250 x 175 x 39mm	Set			
	viii) 250 x 150 x 37mm	Set			
	b) Roller bearing with required number of roller, plates and other fittings complete as per drawings and specification.				
	i) ------ dia of roller				
	c) Hinge plate/ free plate bearing with lead plate and other fitting complete as per drawings and specifications.	Set			
	i) M.S plate of thickness --------	Set			
29.	[4.15] Providing expansion joint including supply of 75mm x 75mm x 6mm M.S angle and 12mm thick M.S plate over expansion joints as per drawing and design & providing 16mm dia anchor bar welded to it @ 1000mm including carrying cost of materials and placing in position on both sides of joints at the edge of entire section of slab etc. all complete as per direction of the Engineer.	L.m.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
30.	Back filling of abutment with 50:50 best quality picked jhama brick khoa & sand of min. F.M. 1.00 of 450mm width, in layers of 150mm thickness free from dust, impurities etc. including compacting using steel or concrete drop hammer (durmus) watering & dressing and including supply & cost of all materials, carrying and labour, arranging and supplying of steel/ concrete hammer and other tools required to work site etc. all complete as per direction of the Engineer . Payment to be made for the compacted volume only for a compaction of 90% of the maximum dry density.	Cu.m.			
31.	(2.1.2) Earth work in bridge/culvert approaches with earth of approved quality, arranged and supplied by the contractor, spreading the same to obtain 150mm compacted layers, breaking clods down to 75mm in each layer, watering/drying as necessary, compacting by minimum 5kg weight 'durmuz' or mechanical equipment to reach a minimum of 85% standard proctor density and dressing as per longitudinal gradient and side slope, with initial lead of 20m and lift up to 2m and maintaining the embankment true to profile upto the end of maintenance period including cost of the earth, loading, unloading, carrying, etc. complete as per drawings and directions of the Engineer.	Cu.m.			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
32.	Supplying, fitting and fixing in position RC pipe with collar or approved joints including cost of materials, labour, carrying, handling, etc. complete as per drawings and directions of the Engineer.				
	i) [4.19 (i)] 300mm dia (inner)	L.m.			
	ii) [4.19 (iii)] 600mm dia (inner)	L.m.			
	iii) [4.19 (iv)] 900mm dia (inner)	L.m.			
33.	Providing Nosing with MS angle size 75mmx75mm X 6mm welding with anchor bars using electrodes, including cost of all materials, carrying, local handling, fitting, fixing and incidentals necessary to complete the work as per drawings and directions of the Engineer.	L.m.			
34.	Welding at the splicing point of main reinforcement at 3 points (each point being 100mm in length) and each alternate contact point of spiral binder tie rod with the main vertical reinforcement of the bored piles using electrodes including the cost of all materials, labour, tools & equipment the cost of power etc. complete as per drawings, specifications and directions of the Engineer.				
	a) Sprial (contact point with the main vertical rod)				
	i) 600mm dia piles	Each Spot			
	ii) 500mm dia piles	Each Spot			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	b) Main Reinforcement (lapping welding of main reinforcement each 100mm length) i) 600mm dia piles ii) 500mm dia piles	 Each point Each point			
35.	Supplying and placing of two layers of gunny bagged rip-rap filled with mixture of sand (FM 1.00), cement (8:1) along slopes of abutments, banks of river/ khal, tamping the bags in place, curing by sprinkling water over the bags including the cost of all materials, labour and incidentals necessary to complete the work as specified and directed by the Engineer.	Sq.m.			
36.	[2.3.11] Turfing on embankment top & slope with good quality turf not less than 225mm in size including supply of turf, placing anchoring turfs with pegs, watering until grass is fully grown, and for all leads and lifts etc. all complete as per direction of the Engineer (payment to be made only when grass is fully grown)	Sq.m.			
37.	Hanging of sign board of approved size as per direction of the Engineer.	L.S			
38.	Supplying, fitting and fixing 300x300x 20mm thick marble Name plate at left hand wheel guard on each side, one in English and one in Bengali including cost of all materials, labour, from work, engrave neatly the approved inscriptions given below etc. complete as per drawings and direction of the Engineer.	Each			

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
39.	(4.12) 6mm thick plaster in cement mortar (1:4) with Portland cement and coarse sand (minimum FM 1.80) in railing bar and rail post only including curing etc. all complete as per design including supply, carrying and cost of all materials and labour etc. all complete as per direction of the Engineer.	Sq.m.			
40.	(4.08) Flash pointing to brick work in cement (1:2) with Portland cement and sand (minimum FM 1.5) including raking the joints, curing, scaffolding if necessary including carriage of materials etc. all complete as per direction of the Engineer.	Sq.m.			
41.	(4.07) Rule pointing to brick work in cement (1:2) with Portland cement and sand (minimum FM 1.5) including raking the joints, curing, scaffolding if necessary including carriage of materials etc. all complete as per direction of the Engineer.	Sq.m.			
42.	(5.66.2) Three coats of white washing/colour washing/ marking the post & rail of structure including cleaning and sand papering the surface and necessary scaffolding etc. all complete as per direction of the Engineer.	Sq.m			
43.	(4.31) Dismantling damaged structural works in bridge and removing the debris to safe distance, stacking properly for measurement etc. all complete as per direction of the Engineer. a) C.C b) R.C.C. c) Brick work	 Cu.m Cu.m Cu.m			

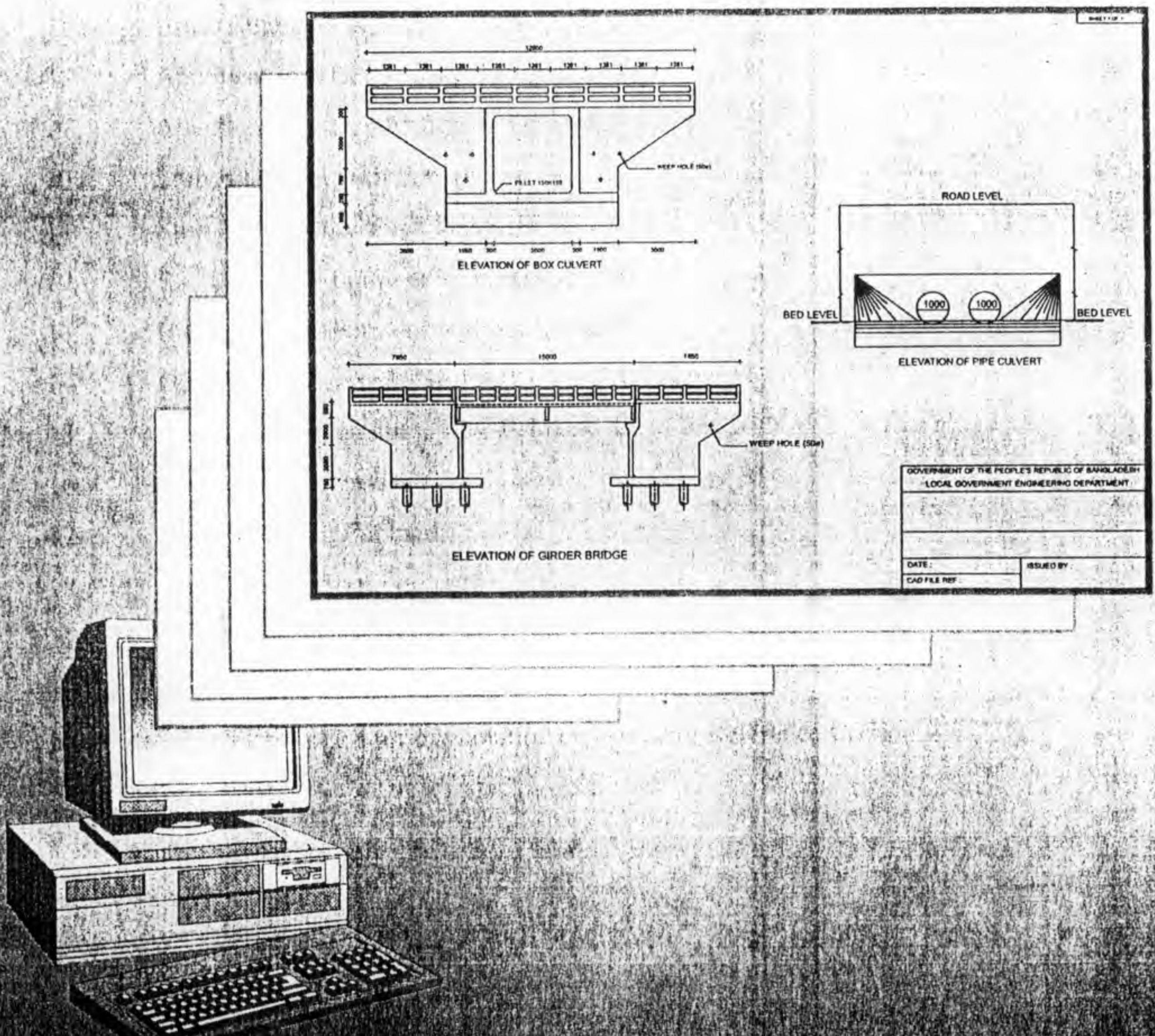
Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
44.	(4.32) Dismantling steel structure bridge for reuse (as far possible) by any means, removing the debris to a safe distance, carrying the salvaged steel members to the storeyard of the dept. or anywhere as directed etc. all complete as per direction of the Engineer.	Tonne			
45.	(4.33) Dismantling existing timber bridge and timber work including selecting, sorting and stacking properly in measurable stacks after carrying those to a safe distance etc. all complete as per direction of the Engineer (measurement for payment to be made for quantity of salvaged wood)	Cu.m			
46.	(4.27) Construction of 2m wide temporary bamboo diversion bridge with 100mm average dia pucca borak bamboo in 4(four) lines of posts longitudinally & required lines of posts 2m c/c transversely with beams and struts & every post with double bracings & each post driven to a least 0.75m depth including half split bamboo decking including supplying nuts bolts etc., fitting, fixing & supply, carrying & cost of all materials & labour etc. all complete as per direction of the Engineer.	L.m			
47.	(4.28) Construction of 0.25m width bamboo sanko with 100mm dia pucca borak bamboo posts of required length @ 2.5m c/ c driven at least 1m below G.L. of stream, with 75mm dia full bamboo decking beams, bracings, diagonals, with 100mm dia bamboo				

Table 16.1
Schedule of Items (Continued)

Item No.	Description of Items	Unit	Quantity	Rate (in Taka) in Figures and Words	Amount (in Taka)
1	2	3	4	5	6
	including fitting, fixing to complete satisfaction including supply of nails, nuts, bolts etc. all materials, carrying & labour etc. all complete as per direction of the Engineer.	L.m.			
48.	Demobilization and clearing the work site, restoration of the channel bed to its original profile including removal of contractor's materials, sheds, machinery, etc. from the work site after completion of the works all to the satisfaction of the Engineer.	L.S.			
49.	(5.25) Providing polythene sheeting in 2 (two) layers (0.18mm thick) on top of top slab of U-type culvert & where necessary as per drawing, specification and direction of the Engineer.	Sq.m.			
50.	(5.31.1) Sand, cement plaster 1:4, 19mm thick with portland cement and sand (minimum FM 1.5) to inside and exposed surfaces of U-type Culvert as per drawing, with neat cement finishing including rounding of edge and corners, polishing curing including cost and carriage of materials etc. complete as per specifications and direction of the Engineer.	Sq.m.			
51.	Supplying and placing of filter material behind the weep holes with gunny bags filled with 1st. class picked jhamma brick bats of size 40mm to 63mm @ 0.0284 m^3 (1 cft) in each bags as per drawing and direction of the Engineer-in-Charge.	Each			

ANNEXURE: ROAD STRUCTURES LIBRARY



SUMMARY OF STRUCTURE LIST
ROAD STRUCTURES LIBRARY

Sl. No.	Structure Description	Number of Structures
1	Pipe Culvert	12
2	U-Drain	2
3	Box Culvert	231
4	Slab Culvert	27
5	Girder Bridge with Footpath	57
6	Girder Bridge without Footpath	88
7	Brick Abutment	10
8	Abutment Full Depth with Footpath	32
9	Abutment Full Depth without Footpath	64
10	Stub Abutment with Footpath	2
11	Stub Abutment without Footpath	4
12	R.C.C. Open Abutment with Footpath	20
13	R.C.C. Open Abutment without Footpath	40
14	General Arrangement Girder Bridge with Footpath	4
15	General Arrangement Girder Bridge without Footpath	8
16	General Arrangement Slab Culvert without Footpath	8
17	Deck Slab with Footpath	2
18	Deck Slab without Footpath	3
	Total Structure: .	**614**

Note: Refer to Section 12.3 for explanation on Structure Reference Code

Structure List of Pipe Culverts

Sl. No.	Reference Code	Structure Description
1	CULV\P1003001	Pipe Culvert; Single Vent; Dia 0300mm;
2	CULV\P1006001	Pipe Culvert; Single Vent; Dia 0600mm;
3	CULV\P1010001	Pipe Culvert; Single Vent; Dia 1000mm;
4	CULV\P2003001	Pipe Culvert; Double Vent; Dia 0300mm;
5	CULV\P2006001	Pipe Culvert; Double Vent; Dia 0600mm;
6	CULV\P2010001	Pipe Culvert; Double Vent; Dia 1000mm;
7	CULV\P3003001	Pipe Culvert; Triple Vent; Dia 0300mm;
8	CULV\P3006001	Pipe Culvert; Triple Vent; Dia 0600mm;
9	CULV\P3010001	Pipe Culvert; Triple Vent; Dia 1000mm;
10	CULV\P4003001	Pipe Culvert; Quadruple Vent; Dia 0300mm;
11	CULV\P4006001	Pipe Culvert; Quadruple Vent; Dia 0600mm;
12	CULV\P4010001	Pipe Culvert; Quadruple Vent; Dia 1000mm;

Structure List of U-Drain

Sl. No.	Reference Code	Structure Description
1	CULV\U0037451	U-type Irrigation Drain; S375mm x H450mm;
2	CULV\U0062601	U-type Irrigation Drain; S625mm x H600mm;

Structure List of Box Culverts

Sl. No.	Reference Code	Structure Description
1	CULV\B1110101	Box Culvert; Single Vent; Load H10; S1.0m x H 1.0m;
2	CULV\B1113101	Box Culvert; Single Vent; Load H10; S1.3m x H 1.0m;
3	CULV\B1113131	Box Culvert; Single Vent; Load H10; S1.3m x H 1.3m;
4	CULV\B1116131	Box Culvert; Single Vent; Load H10; S1.6m x H 1.3m;
5	CULV\B1116161	Box Culvert; Single Vent; Load H10; S1.6m x H 1.6m;
6	CULV\B1120161	Box Culvert; Single Vent; Load H10; S2.0m x H 1.6m;
7	CULV\B1123161	Box Culvert; Single Vent; Load H10; S2.3m x H 1.6m;
8	CULV\B1120201	Box Culvert; Single Vent; Load H10; S2.0m x H 2.0m;
9	CULV\B1123201	Box Culvert; Single Vent; Load H10; S2.3m x H 2.0m;
10	CULV\B1126201	Box Culvert; Single Vent; Load H10; S2.6m x H 2.0m;
11	CULV\B1130201	Box Culvert; Single Vent; Load H10; S3.0m x H 2.0m;
12	CULV\B1123231	Box Culvert; Single Vent; Load H10; S2.3m x H 2.3m;
13	CULV\B1126231	Box Culvert; Single Vent; Load H10; S2.6m x H 2.3m;
14	CULV\B1130231	Box Culvert; Single Vent; Load H10; S3.0m x H 2.3m;
15	CULV\B1133231	Box Culvert; Single Vent; Load H10; S3.3m x H 2.3m;
16	CULV\B1126261	Box Culvert; Single Vent; Load H10; S2.6m x H 2.6m;
17	CULV\B1130261	Box Culvert; Single Vent; Load H10; S3.0m x H 2.6m;
18	CULV\B1133261	Box Culvert; Single Vent; Load H10; S3.3m x H 2.6m;
19	CULV\B1136261	Box Culvert; Single Vent; Load H10; S3.6m x H 2.6m;
20	CULV\B1130301	Box Culvert; Single Vent; Load H10; S3.0m x H 3.0m;
21	CULV\B1133301	Box Culvert; Single Vent; Load H10; S3.3m x H 3.0m;
22	CULV\B1136301	Box Culvert; Single Vent; Load H10; S3.6m x H 3.0m;
23	CULV\B1140301	Box Culvert; Single Vent; Load H10; S4.0m x H 3.0m;
24	CULV\B1143301	Box Culvert; Single Vent; Load H10; S4.3m x H 3.0m;
25	CULV\B1133331	Box Culvert; Single Vent; Load H10; S3.3m x H 3.3m;
26	CULV\B1136331	Box Culvert; Single Vent; Load H10; S3.6m x H 3.3m;
27	CULV\B1140331	Box Culvert; Single Vent; Load H10; S4.0m x H 3.3m;
28	CULV\B1143331	Box Culvert; Single Vent; Load H10; S4.3m x H 3.3m;
29	CULV\B1136361	Box Culvert; Single Vent; Load H10; S3.6m x H 3.6m;
30	CULV\B1140361	Box Culvert; Single Vent; Load H10; S4.0m x H 3.6m;
31	CULV\B1143361	Box Culvert; Single Vent; Load H10; S4.3m x H 3.6m;
32	CULV\B1140401	Box Culvert; Single Vent; Load H10; S4.0m x H 4.0m;
33	CULV\B1143401	Box Culvert; Single Vent; Load H10; S4.3m x H 4.0m;
34	CULV\B1143431	Box Culvert; Single Vent; Load H10; S4.3m x H 4.3m;
35	CULV\B1210101	Box Culvert; Single Vent; Load H15; S1.0m x H 1.0m;
36	CULV\B1213101	Box Culvert; Single Vent; Load H15; S1.3m x H 1.0m;
37	CULV\B1213131	Box Culvert; Single Vent; Load H15; S1.3m x H 1.3m;
38	CULV\B1216131	Box Culvert; Single Vent; Load H15; S1.6m x H 1.3m;
39	CULV\B1216161	Box Culvert; Single Vent; Load H15; S1.6m x H 1.6m;
40	CULV\B1220161	Box Culvert; Single Vent; Load H15; S2.0m x H 1.6m;
41	CULV\B1223161	Box Culvert; Single Vent; Load H15; S2.3m x H 1.6m;
42	CULV\B1220201	Box Culvert; Single Vent; Load H15; S2.0m x H 2.0m;
43	CULV\B1223201	Box Culvert; Single Vent; Load H15; S2.3m x H 2.0m;
44	CULV\B1226201	Box Culvert; Single Vent; Load H15; S2.6m x H 2.0m;

Sl. No.	Reference Code	Structure Description
45	CULV\B1230201	Box Culvert; Single Vent; Load H15; S3.0m x H 2.0m;
46	CULV\B1223231	Box Culvert; Single Vent; Load H15; S2.3m x H 2.3m;
47	CULV\B1226231	Box Culvert; Single Vent; Load H15; S2.6m x H 2.3m;
48	CULV\B1230231	Box Culvert; Single Vent; Load H15; S3.0m x H 2.3m;
49	CULV\B1233231	Box Culvert; Single Vent; Load H15; S3.3m x H 2.3m;
50	CULV\B1226261	Box Culvert; Single Vent; Load H15; S2.6m x H 2.6m;
51	CULV\B1230261	Box Culvert; Single Vent; Load H15; S3.0m x H 2.6m;
52	CULV\B1233261	Box Culvert; Single Vent; Load H15; S3.3m x H 2.6m;
53	CULV\B1236261	Box Culvert; Single Vent; Load H15; S3.6m x H 2.6m;
54	CULV\B1230301	Box Culvert; Single Vent; Load H15; S3.0m x H 3.0m;
55	CULV\B1233301	Box Culvert; Single Vent; Load H15; S3.3m x H 3.0m;
56	CULV\B1236301	Box Culvert; Single Vent; Load H15; S3.6m x H 3.0m;
57	CULV\B1240301	Box Culvert; Single Vent; Load H15; S4.0m x H 3.0m;
58	CULV\B1243301	Box Culvert; Single Vent; Load H15; S4.3m x H 3.0m;
59	CULV\B1233331	Box Culvert; Single Vent; Load H15; S3.3m x H 3.3m;
60	CULV\B1236331	Box Culvert; Single Vent; Load H15; S3.6m x H 3.3m;
61	CULV\B1240331	Box Culvert; Single Vent; Load H15; S4.0m x H 3.3m;
62	CULV\B1243331	Box Culvert; Single Vent; Load H15; S4.3m x H 3.3m;
63	CULV\B1236361	Box Culvert; Single Vent; Load H15; S3.6m x H 3.6m;
64	CULV\B1240361	Box Culvert; Single Vent; Load H15; S4.0m x H 3.6m;
65	CULV\B1243361	Box Culvert; Single Vent; Load H15; S4.3m x H 3.6m;
66	CULV\B1240401	Box Culvert; Single Vent; Load H15; S4.0m x H 4.0m;
67	CULV\B1243401	Box Culvert; Single Vent; Load H15; S4.3m x H 4.0m;
68	CULV\B1243431	Box Culvert; Single Vent; Load H15; S4.3m x H 4.3m;
69	CULV\B1310101	Box Culvert; Single Vent; Load H20; S1.0m x H 1.0m;
70	CULV\B1313101	Box Culvert; Single Vent; Load H20; S1.3m x H 1.0m;
71	CULV\B1313131	Box Culvert; Single Vent; Load H20; S1.3m x H 1.3m;
72	CULV\B1316131	Box Culvert; Single Vent; Load H20; S1.6m x H 1.3m;
73	CULV\B1316161	Box Culvert; Single Vent; Load H20; S1.6m x H 1.6m;
74	CULV\B1320161	Box Culvert; Single Vent; Load H20; S2.0m x H 1.6m;
75	CULV\B1323161	Box Culvert; Single Vent; Load H20; S2.3m x H 1.6m;
76	CULV\B1320201	Box Culvert; Single Vent; Load H20; S2.0m x H 2.0m;
77	CULV\B1323201	Box Culvert; Single Vent; Load H20; S2.3m x H 2.0m;
78	CULV\B1326201	Box Culvert; Single Vent; Load H20; S2.6m x H 2.0m;
79	CULV\B1330201	Box Culvert; Single Vent; Load H20; S3.0m x H 2.0m;
80	CULV\B1323231	Box Culvert; Single Vent; Load H20; S2.3m x H 2.3m;
81	CULV\B1326231	Box Culvert; Single Vent; Load H20; S2.6m x H 2.3m;
82	CULV\B1330231	Box Culvert; Single Vent; Load H20; S3.0m x H 2.3m;
83	CULV\B1333231	Box Culvert; Single Vent; Load H20; S3.3m x H 2.3m;
84	CULV\B1326261	Box Culvert; Single Vent; Load H20; S2.6m x H 2.6m;
85	CULV\B1330261	Box Culvert; Single Vent; Load H20; S3.0m x H 2.6m;
86	CULV\B1333261	Box Culvert; Single Vent; Load H20; S3.3m x H 2.6m;
87	CULV\B1336261	Box Culvert; Single Vent; Load H20; S3.6m x H 2.6m;
88	CULV\B1330301	Box Culvert; Single Vent; Load H20; S3.0m x H 3.0m;
89	CULV\B1333301	Box Culvert; Single Vent; Load H20; S3.3m x H 3.0m;
90	CULV\B1336301	Box Culvert; Single Vent; Load H20; S3.6m x H 3.0m;

Sl. No.	Reference Code	Structure Description
91	CULV\B1340301	Box Culvert; Single Vent; Load H20; S4.0m x H 3.0m;
92	CULV\B1343301	Box Culvert; Single Vent; Load H20; S4.3m x H 3.0m;
93	CULV\B1333331	Box Culvert; Single Vent; Load H20; S3.3m x H 3.3m;
94	CULV\B1336331	Box Culvert; Single Vent; Load H20; S3.6m x H 3.3m;
95	CULV\B1340331	Box Culvert; Single Vent; Load H20; S4.0m x H 3.3m;
96	CULV\B1343331	Box Culvert; Single Vent; Load H20; S4.3m x H 3.3m;
97	CULV\B1336361	Box Culvert; Single Vent; Load H20; S3.6m x H 3.6m;
98	CULV\B1340361	Box Culvert; Single Vent; Load H20; S4.0m x H 3.6m;
99	CULV\B1343361	Box Culvert; Single Vent; Load H20; S4.3m x H 3.6m;
100	CULV\B1340401	Box Culvert; Single Vent; Load H20; S4.0m x H 4.0m;
101	CULV\B1343401	Box Culvert; Single Vent; Load H20; S4.3m x H 4.0m;
102	CULV\B1343431	Box Culvert; Single Vent; Load H20; S4.3m x H 4.3m;
103	CULV\B2123161	Box Culvert; Double Vent; Load H10; S2.3m x H 1.6m;
104	CULV\B2123201	Box Culvert; Double Vent; Load H10; S2.3m x H 2.0m;
105	CULV\B2126201	Box Culvert; Double Vent; Load H10; S2.6m x H 2.0m;
106	CULV\B2130201	Box Culvert; Double Vent; Load H10; S3.0m x H 2.0m;
107	CULV\B2123231	Box Culvert; Double Vent; Load H10; S2.3m x H 2.3m;
108	CULV\B2126231	Box Culvert; Double Vent; Load H10; S2.6m x H 2.3m;
109	CULV\B2130231	Box Culvert; Double Vent; Load H10; S3.0m x H 2.3m;
110	CULV\B2133231	Box Culvert; Double Vent; Load H10; S3.3m x H 2.3m;
111	CULV\B2126261	Box Culvert; Double Vent; Load H10; S2.6m x H 2.6m;
112	CULV\B2130261	Box Culvert; Double Vent; Load H10; S3.0m x H 2.6m;
113	CULV\B2133261	Box Culvert; Double Vent; Load H10; S3.3m x H 2.6m;
114	CULV\B2136261	Box Culvert; Double Vent; Load H10; S3.6m x H 2.6m;
115	CULV\B2130301	Box Culvert; Double Vent; Load H10; S3.0m x H 3.0m;
116	CULV\B2133301	Box Culvert; Double Vent; Load H10; S3.3m x H 3.0m;
117	CULV\B2136301	Box Culvert; Double Vent; Load H10; S3.6m x H 3.0m;
118	CULV\B2140301	Box Culvert; Double Vent; Load H10; S4.0m x H 3.0m;
119	CULV\B2143301	Box Culvert; Double Vent; Load H10; S4.3m x H 3.0m;
120	CULV\B2133331	Box Culvert; Double Vent; Load H10; S3.3m x H 3.3m;
121	CULV\B2136331	Box Culvert; Double Vent; Load H10; S3.6m x H 3.3m;
122	CULV\B2140331	Box Culvert; Double Vent; Load H10; S4.0m x H 3.3m;
123	CULV\B2143331	Box Culvert; Double Vent; Load H10; S4.3m x H 3.3m;
124	CULV\B2136361	Box Culvert; Double Vent; Load H10; S3.6m x H 3.6m;
125	CULV\B2140361	Box Culvert; Double Vent; Load H10; S4.0m x H 3.6m;
126	CULV\B2143361	Box Culvert; Double Vent; Load H10; S4.3m x H 3.6m;
127	CULV\B2140401	Box Culvert; Double Vent; Load H10; S4.0m x H 4.0m;
128	CULV\B2143401	Box Culvert; Double Vent; Load H10; S4.3m x H 4.0m;
129	CULV\B2143431	Box Culvert; Double Vent; Load H10; S4.3m x H 4.3m;
130	CULV\B2223161	Box Culvert; Double Vent; Load H15; S2.3m x H 1.6m;
131	CULV\B2223201	Box Culvert; Double Vent; Load H15; S2.3m x H 2.0m;
132	CULV\B2226201	Box Culvert; Double Vent; Load H15; S2.6m x H 2.0m;
133	CULV\B2230201	Box Culvert; Double Vent; Load H15; S3.0m x H 2.0m;
134	CULV\B2223231	Box Culvert; Double Vent; Load H15; S2.3m x H 2.3m;
135	CULV\B2226231	Box Culvert; Double Vent; Load H15; S2.6m x H 2.3m;
136	CULV\B2230231	Box Culvert; Double Vent; Load H15; S3.0m x H 2.3m;

Sl. No.	Reference Code	Structure Description
137	CULV\B2233231	Box Culvert; Double Vent; Load H15; S3.3m x H 2.3m;
138	CULV\B2226261	Box Culvert; Double Vent; Load H15; S2.6m x H 2.6m;
139	CULV\B2230261	Box Culvert; Double Vent; Load H15; S3.0m x H 2.6m;
140	CULV\B2233261	Box Culvert; Double Vent; Load H15; S3.3m x H 2.6m;
141	CULV\B2236261	Box Culvert; Double Vent; Load H15; S3.6m x H 2.6m;
142	CULV\B2230301	Box Culvert; Double Vent; Load H15; S3.0m x H 3.0m;
143	CULV\B2233301	Box Culvert; Double Vent; Load H15; S3.3m x H 3.0m;
144	CULV\B2236301	Box Culvert; Double Vent; Load H15; S3.6m x H 3.0m;
145	CULV\B2240301	Box Culvert; Double Vent; Load H15; S4.0m x H 3.0m;
146	CULV\B2243301	Box Culvert; Double Vent; Load H15; S4.3m x H 3.0m;
147	CULV\B2233331	Box Culvert; Double Vent; Load H15; S3.3m x H 3.3m;
148	CULV\B2236331	Box Culvert; Double Vent; Load H15; S3.6m x H 3.3m;
149	CULV\B2240331	Box Culvert; Double Vent; Load H15; S4.0m x H 3.3m;
150	CULV\B2243331	Box Culvert; Double Vent; Load H15; S4.3m x H 3.3m;
151	CULV\B2236361	Box Culvert; Double Vent; Load H15; S3.6m x H 3.6m;
152	CULV\B2240361	Box Culvert; Double Vent; Load H15; S4.0m x H 3.6m;
153	CULV\B2243361	Box Culvert; Double Vent; Load H15; S4.3m x H 3.6m;
154	CULV\B2240401	Box Culvert; Double Vent; Load H15; S4.0m x H 4.0m;
155	CULV\B2243401	Box Culvert; Double Vent; Load H15; S4.3m x H 4.0m;
156	CULV\B2243431	Box Culvert; Double Vent; Load H15; S4.3m x H 4.3m;
157	CULV\B2323161	Box Culvert; Double Vent; Load H20; S2.3m x H 1.6m;
158	CULV\B2323201	Box Culvert; Double Vent; Load H20; S2.3m x H 2.0m;
159	CULV\B2326201	Box Culvert; Double Vent; Load H20; S2.6m x H 2.0m;
160	CULV\B2330201	Box Culvert; Double Vent; Load H20; S3.0m x H 2.0m;
161	CULV\B2323231	Box Culvert; Double Vent; Load H20; S2.3m x H 2.3m;
162	CULV\B2326231	Box Culvert; Double Vent; Load H20; S2.6m x H 2.3m;
163	CULV\B2330231	Box Culvert; Double Vent; Load H20; S3.0m x H 2.3m;
164	CULV\B2333231	Box Culvert; Double Vent; Load H20; S3.3m x H 2.3m;
165	CULV\B2326261	Box Culvert; Double Vent; Load H20; S2.6m x H 2.6m;
166	CULV\B2330261	Box Culvert; Double Vent; Load H20; S3.0m x H 2.6m;
167	CULV\B2333261	Box Culvert; Double Vent; Load H20; S3.3m x H 2.6m;
168	CULV\B2336261	Box Culvert; Double Vent; Load H20; S3.6m x H 2.6m;
169	CULV\B2330301	Box Culvert; Double Vent; Load H20; S3.0m x H 3.0m;
170	CULV\B2333301	Box Culvert; Double Vent; Load H20; S3.3m x H 3.0m;
171	CULV\B2336301	Box Culvert; Double Vent; Load H20; S3.6m x H 3.0m;
172	CULV\B2340301	Box Culvert; Double Vent; Load H20; S4.0m x H 3.0m;
173	CULV\B2343301	Box Culvert; Double Vent; Load H20; S4.3m x H 3.0m;
174	CULV\B2333331	Box Culvert; Double Vent; Load H20; S3.3m x H 3.3m;
175	CULV\B2336331	Box Culvert; Double Vent; Load H20; S3.6m x H 3.3m;
176	CULV\B2340331	Box Culvert; Double Vent; Load H20; S4.0m x H 3.3m;
177	CULV\B2343331	Box Culvert; Double Vent; Load H20; S4.3m x H 3.3m;
178	CULV\B2336361	Box Culvert; Double Vent; Load H20; S3.6m x H 3.6m;
179	CULV\B2340361	Box Culvert; Double Vent; Load H20; S4.0m x H 3.6m;
180	CULV\B2343361	Box Culvert; Double Vent; Load H20; S4.3m x H 3.6m;
181	CULV\B2340401	Box Culvert; Double Vent; Load H20; S4.0m x H 4.0m;
182	CULV\B2343401	Box Culvert; Double Vent; Load H20; S4.3m x H 4.0m;

Sl. No.	Reference Code	Structure Description
183	CULV\B2343431	Box Culvert; Double Vent; Load H20; S4.3m x H 4.3m;
184	CULV\B3130201	Box Culvert; Triple Vent; Load H10; S3.0m x H 2.0m;
185	CULV\B3130231	Box Culvert; Triple Vent; Load H10; S3.0m x H 2.3m;
186	CULV\B3130261	Box Culvert; Triple Vent; Load H10; S3.0m x H 2.6m;
187	CULV\B3130301	Box Culvert; Triple Vent; Load H10; S3.0m x H 3.0m;
188	CULV\B3133231	Box Culvert; Triple Vent; Load H10; S3.3m x H 2.3m;
189	CULV\B3133261	Box Culvert; Triple Vent; Load H10; S3.3m x H 2.6m;
190	CULV\B3136261	Box Culvert; Triple Vent; Load H10; S3.6m x H 2.6m;
191	CULV\B3133301	Box Culvert; Triple Vent; Load H10; S3.3m x H 3.0m;
192	CULV\B3136301	Box Culvert; Triple Vent; Load H10; S3.6m x H 3.0m;
193	CULV\B3140301	Box Culvert; Triple Vent; Load H10; S4.0m x H 3.0m;
194	CULV\B3133331	Box Culvert; Triple Vent; Load H10; S3.3m x H 3.3m;
195	CULV\B3136331	Box Culvert; Triple Vent; Load H10; S3.6m x H 3.3m;
196	CULV\B3140331	Box Culvert; Triple Vent; Load H10; S4.0m x H 3.3m;
197	CULV\B3136361	Box Culvert; Triple Vent; Load H10; S3.6m x H 3.6m;
198	CULV\B3140361	Box Culvert; Triple Vent; Load H10; S4.0m x H 3.6m;
199	CULV\B3140401	Box Culvert; Triple Vent; Load H10; S4.0m x H 4.0m;
200	CULV\B3230201	Box Culvert; Triple Vent; Load H15; S3.0m x H 2.0m;
201	CULV\B3230231	Box Culvert; Triple Vent; Load H15; S3.0m x H 2.3m;
202	CULV\B3230261	Box Culvert; Triple Vent; Load H15; S3.0m x H 2.6m;
203	CULV\B3230301	Box Culvert; Triple Vent; Load H15; S3.0m x H 3.0m;
204	CULV\B3233231	Box Culvert; Triple Vent; Load H15; S3.3m x H 2.3m;
205	CULV\B3233261	Box Culvert; Triple Vent; Load H15; S3.3m x H 2.6m;
206	CULV\B3236261	Box Culvert; Triple Vent; Load H15; S3.6m x H 2.6m;
207	CULV\B3233301	Box Culvert; Triple Vent; Load H15; S3.3m x H 3.0m;
208	CULV\B3236301	Box Culvert; Triple Vent; Load H15; S3.6m x H 3.0m;
209	CULV\B3240301	Box Culvert; Triple Vent; Load H15; S4.0m x H 3.0m;
210	CULV\B3233331	Box Culvert; Triple Vent; Load H15; S3.3m x H 3.3m;
211	CULV\B3236331	Box Culvert; Triple Vent; Load H15; S3.6m x H 3.3m;
212	CULV\B3240331	Box Culvert; Triple Vent; Load H15; S4.0m x H 3.3m;
213	CULV\B3236361	Box Culvert; Triple Vent; Load H15; S3.6m x H 3.6m;
214	CULV\B3240361	Box Culvert; Triple Vent; Load H15; S4.0m x H 3.6m;
215	CULV\B3240401	Box Culvert; Triple Vent; Load H15; S4.0m x H 4.0m;
216	CULV\B3330201	Box Culvert; Triple Vent; Load H20; S3.0m x H 2.0m;
217	CULV\B3330231	Box Culvert; Triple Vent; Load H20; S3.0m x H 2.3m;
218	CULV\B3330261	Box Culvert; Triple Vent; Load H20; S3.0m x H 2.6m;
219	CULV\B3330301	Box Culvert; Triple Vent; Load H20; S3.0m x H 3.0m;
220	CULV\B3333231	Box Culvert; Triple Vent; Load H20; S3.3m x H 2.3m;
221	CULV\B3333261	Box Culvert; Triple Vent; Load H20; S3.3m x H 2.6m;
222	CULV\B3336261	Box Culvert; Triple Vent; Load H20; S3.6m x H 2.6m;
223	CULV\B3333301	Box Culvert; Triple Vent; Load H20; S3.3m x H 3.0m;
224	CULV\B3336301	Box Culvert; Triple Vent; Load H20; S3.6m x H 3.0m;
225	CULV\B3340301	Box Culvert; Triple Vent; Load H20; S4.0m x H 3.0m;
226	CULV\B3333331	Box Culvert; Triple Vent; Load H20; S3.3m x H 3.3m;
227	CULV\B3336331	Box Culvert; Triple Vent; Load H20; S3.6m x H 3.3m;
228	CULV\B3340331	Box Culvert; Triple Vent; Load H20; S4.0m x H 3.3m;
229	CULV\B3336361	Box Culvert; Triple Vent; Load H20; S3.6m x H 3.6m;

230	CULV\B3340361	Box Culvert; Triple Vent; Load H20; S4.0m x H 3.6m;
231	CULV\B3340401	Box Culvert; Triple Vent; Load H20; S4.0m x H 4.0m;

Structure List of Slab Culverts

Sl. No.	Reference Code	Structure Description
1	GNRL\S000R201	General Arrangement Slab Culvert; Without Footpath; CW3.66; Full Depth Abutment
2	GNRL\S000R101	General Arrangement Slab Culvert; Without Footpath; CW2.44; Full Depth Abutment
3	GNRL\S000S201	General Arrangement Slab Culvert; Without Footpath; CW3.66; Stub Abutment
4	GNRL\S000S101	General Arrangement Slab Culvert; Without Footpath; CW2.44; Stub Abutment
5	GNRL\S000B201	General Arrangement Slab Culvert; Without Footpath; CW3.66; Brick Abutment
6	GNRL\S000B101	General Arrangement Slab Culvert; Without Footpath; CW2.44; Brick Abutment
7	GNRL\S000O201	General Arrangement Slab Culvert; Without Footpath; CW3.66; Open Abutment
8	GNRL\S000O101	General Arrangement Slab Culvert; Without Footpath; CW2.44; Open Abutment
9	CULV\S0101501	Slab Culvert; Load H10; S1.5m;
10	CULV\S0102001	Slab Culvert; Load H10; S2.0m;
11	CULV\S0102501	Slab Culvert; Load H10; S2.5m;
12	CULV\S0103001	Slab Culvert; Load H10; S3.0m;
13	CULV\S0103501	Slab Culvert; Load H10; S3.5m;
14	CULV\S0104001	Slab Culvert; Load H10; S4.0m;
15	CULV\S0104501	Slab Culvert; Load H10; S4.5m;
16	CULV\S0105001	Slab Culvert; Load H10; S5.0m;
17	CULV\S0105501	Slab Culvert; Load H10; S5.5m;
18	CULV\S0201501	Slab Culvert; Load H15; S1.5m;
19	CULV\S0202001	Slab Culvert; Load H15; S2.0m;
20	CULV\S0202501	Slab Culvert; Load H15; S2.5m;
21	CULV\S0203001	Slab Culvert; Load H15; S3.0m;
22	CULV\S0203501	Slab Culvert; Load H15; S3.5m;
23	CULV\S0204001	Slab Culvert; Load H15; S4.0m;
24	CULV\S0204501	Slab Culvert; Load H15; S4.5m;
25	CULV\S0205001	Slab Culvert; Load H15; S5.0m;
26	CULV\S0205501	Slab Culvert; Load H15; S5.5m;
27	CULV\S0301501	Slab Culvert; Load H20; S1.5m;
28	CULV\S0302001	Slab Culvert; Load H20; S2.0m;
29	CULV\S0302501	Slab Culvert; Load H20; S2.5m;
30	CULV\S0303001	Slab Culvert; Load H20; S3.0m;
31	CULV\S0303501	Slab Culvert; Load H20; S3.5m;
32	CULV\S0304001	Slab Culvert; Load H20; S4.0m;
33	CULV\S0304501	Slab Culvert; Load H20; S4.5m;
34	CULV\S0305001	Slab Culvert; Load H20; S5.0m;
35	CULV\S0305501	Slab Culvert; Load H20; S5.5m;

Structure List of Girder Bridges

Sl. No.	Reference Code	Structure Description
1	GNRL\G000R201	General Arrangement Girder Bridge; Without Footpath; CW3.66; Full Depth Abutment
2	GNRL\G100R201	General Arrangement Girder Bridge; With Footpath; CW3.66; Full Depth Abutment
3	GNRL\G000R101	General Arrangement Girder Bridge; Without Footpath; CW2.44; Full Depth Abutment
4	GNRL\G000S201	General Arrangement Girder Bridge; Without Footpath; CW3.66; Stub Abutment
5	GNRL\G100S201	General Arrangement Girder Bridge; With Footpath; CW3.66; Stub Abutment
6	GNRL\G000S101	General Arrangement Girder Bridge; Without Footpath; CW2.44; Stub Abutment
7	GNRL\G000B201	General Arrangement Girder Bridge; Without Footpath; CW3.66; Brick Abutment
8	GNRL\G100B201	General Arrangement Girder Bridge; With Footpath; CW3.66; Brick Abutment
9	GNRL\G000B101	General Arrangement Girder Bridge; Without Footpath; CW2.44; Brick Abutment
10	GNRL\G000O201	General Arrangement Girder Bridge; Without Footpath; CW3.66; Open Abutment
11	GNRL\G100O201	General Arrangement Girder Bridge; With Footpath; CW3.66; Open Abutment
12	GNRL\G000O101	General Arrangement Girder Bridge; Without Footpath; CW2.44; Open Abutment
13	BRDG\D0100001	Deck Slab; Without Footpath; Load H10;
14	BRDG\D0200001	Deck Slab; Without Footpath; Load H15;
15	BRDG\D0300001	Deck Slab; Without Footpath; Load H20;
16	BRDG\D1200001	Deck Slab; With Footpath; Load H15;
17	BRDG\D1300001	Deck Slab; With Footpath; Load H20;
18	BRDG\G0106001	Girder Bridge; Without Footpath; Load H10; S6.0m;
19	BRDG\G0106501	Girder Bridge; Without Footpath; Load H10; S6.5m;
20	BRDG\G0107001	Girder Bridge; Without Footpath; Load H10; S7.0m;
21	BRDG\G0107501	Girder Bridge; Without Footpath; Load H10; S7.5m;
22	BRDG\G0108001	Girder Bridge; Without Footpath; Load H10; S8.0m;
23	BRDG\G0108501	Girder Bridge; Without Footpath; Load H10; S8.5m;
24	BRDG\G0109001	Girder Bridge; Without Footpath; Load H10; S9.0m;
25	BRDG\G0109501	Girder Bridge; Without Footpath; Load H10; S9.5m;
26	BRDG\G0110001	Girder Bridge; Without Footpath; Load H10; S10.0m;
27	BRDG\G0110501	Girder Bridge; Without Footpath; Load H10; S10.5m;
28	BRDG\G0111001	Girder Bridge; Without Footpath; Load H10; S11.0m;
29	BRDG\G0111501	Girder Bridge; Without Footpath; Load H10; S11.5m;
30	BRDG\G0112001	Girder Bridge; Without Footpath; Load H10; S12.0m;
31	BRDG\G0112501	Girder Bridge; Without Footpath; Load H10; S12.5m;
32	BRDG\G0113001	Girder Bridge; Without Footpath; Load H10; S13.0m;
33	BRDG\G0113501	Girder Bridge; Without Footpath; Load H10; S13.5m;
34	BRDG\G0114001	Girder Bridge; Without Footpath; Load H10; S14.0m;

Sl. No.	Reference Code	Structure Description
35	BRDG\G0114501	Girder Bridge; Without Footpath; Load H10; S14.5m;
36	BRDG\G0115001	Girder Bridge; Without Footpath; Load H10; S15.0m;
37	BRDG\G0115501	Girder Bridge; Without Footpath; Load H10; S15.5m;
38	BRDG\G0116001	Girder Bridge; Without Footpath; Load H10; S16.0m;
39	BRDG\G0116501	Girder Bridge; Without Footpath; Load H10; S16.5m;
40	BRDG\G0117001	Girder Bridge; Without Footpath; Load H10; S17.0m;
41	BRDG\G0117501	Girder Bridge; Without Footpath; Load H10; S17.5m;
42	BRDG\G0118001	Girder Bridge; Without Footpath; Load H10; S18.0m;
43	BRDG\G0118501	Girder Bridge; Without Footpath; Load H10; S18.5m;
44	BRDG\G0119001	Girder Bridge; Without Footpath; Load H10; S19.0m;
45	BRDG\G0119501	Girder Bridge; Without Footpath; Load H10; S19.5m;
46	BRDG\G0120001	Girder Bridge; Without Footpath; Load H10; S20.0m;
47	BRDG\G0206001	Girder Bridge; Without Footpath; Load H15; S6.0m;
48	BRDG\G0206501	Girder Bridge; Without Footpath; Load H15; S6.5m;
49	BRDG\G0207001	Girder Bridge; Without Footpath; Load H15; S7.0m;
50	BRDG\G0207501	Girder Bridge; Without Footpath; Load H15; S7.5m;
51	BRDG\G0208001	Girder Bridge; Without Footpath; Load H15; S8.0m;
52	BRDG\G0208501	Girder Bridge; Without Footpath; Load H15; S8.5m;
53	BRDG\G0209001	Girder Bridge; Without Footpath; Load H15; S9.0m;
54	BRDG\G0209501	Girder Bridge; Without Footpath; Load H15; S9.5m;
55	BRDG\G0210001	Girder Bridge; Without Footpath; Load H15; S10.0m;
56	BRDG\G0210501	Girder Bridge; Without Footpath; Load H15; S10.5m;
57	BRDG\G0211001	Girder Bridge; Without Footpath; Load H15; S11.0m;
58	BRDG\G0211501	Girder Bridge; Without Footpath; Load H15; S11.5m;
59	BRDG\G0212001	Girder Bridge; Without Footpath; Load H15; S12.0m;
60	BRDG\G0212501	Girder Bridge; Without Footpath; Load H15; S12.5m;
61	BRDG\G0213001	Girder Bridge; Without Footpath; Load H15; S13.0m;
62	BRDG\G0213501	Girder Bridge; Without Footpath; Load H15; S13.5m;
63	BRDG\G0214001	Girder Bridge; Without Footpath; Load H15; S14.0m;
64	BRDG\G0214501	Girder Bridge; Without Footpath; Load H15; S14.5m;
65	BRDG\G0215001	Girder Bridge; Without Footpath; Load H15; S15.0m;
66	BRDG\G0215501	Girder Bridge; Without Footpath; Load H15; S15.5m;
67	BRDG\G0216001	Girder Bridge; Without Footpath; Load H15; S16.0m;
68	BRDG\G0216501	Girder Bridge; Without Footpath; Load H15; S16.5m;
69	BRDG\G0217001	Girder Bridge; Without Footpath; Load H15; S17.0m;
70	BRDG\G0217501	Girder Bridge; Without Footpath; Load H15; S17.5m;
71	BRDG\G0218001	Girder Bridge; Without Footpath; Load H15; S18.0m;
72	BRDG\G0218501	Girder Bridge; Without Footpath; Load H15; S18.5m;
73	BRDG\G0219001	Girder Bridge; Without Footpath; Load H15; S19.0m;
74	BRDG\G0219501	Girder Bridge; Without Footpath; Load H15; S19.5m;
75	BRDG\G0220001	Girder Bridge; Without Footpath; Load H15; S20.0m;
76	BRDG\G0306001	Girder Bridge; Without Footpath; Load H20; S6.0m;
77	BRDG\G0306501	Girder Bridge; Without Footpath; Load H20; S6.5m;
78	BRDG\G0307001	Girder Bridge; Without Footpath; Load H20; S7.0m;
79	BRDG\G0307501	Girder Bridge; Without Footpath; Load H20; S7.5m;
80	BRDG\G0308001	Girder Bridge; Without Footpath; Load H20; S8.0m;

Sl. No.	Reference Code	Structure Description
81	BRDG\G0308501	Girder Bridge; Without Footpath; Load H20; S8.5m;
82	BRDG\G0309001	Girder Bridge; Without Footpath; Load H20; S9.0m;
83	BRDG\G0309501	Girder Bridge; Without Footpath; Load H20; S9.5m;
84	BRDG\G0310001	Girder Bridge; Without Footpath; Load H20; S10.0m;
85	BRDG\G0310501	Girder Bridge; Without Footpath; Load H20; S10.5m;
86	BRDG\G0311001	Girder Bridge; Without Footpath; Load H20; S11.0m;
87	BRDG\G0311501	Girder Bridge; Without Footpath; Load H20; S11.5m;
88	BRDG\G0312001	Girder Bridge; Without Footpath; Load H20; S12.0m;
89	BRDG\G0312501	Girder Bridge; Without Footpath; Load H20; S12.5m;
90	BRDG\G0313001	Girder Bridge; Without Footpath; Load H20; S13.0m;
91	BRDG\G0313501	Girder Bridge; Without Footpath; Load H20; S13.5m;
92	BRDG\G0314001	Girder Bridge; Without Footpath; Load H20; S14.0m;
93	BRDG\G0314501	Girder Bridge; Without Footpath; Load H20; S14.5m;
94	BRDG\G0315001	Girder Bridge; Without Footpath; Load H20; S15.0m;
95	BRDG\G0315501	Girder Bridge; Without Footpath; Load H20; S15.5m;
96	BRDG\G0316001	Girder Bridge; Without Footpath; Load H20; S16.0m;
97	BRDG\G0316501	Girder Bridge; Without Footpath; Load H20; S16.5m;
98	BRDG\G0317001	Girder Bridge; Without Footpath; Load H20; S17.0m;
99	BRDG\G0317501	Girder Bridge; Without Footpath; Load H20; S17.5m;
100	BRDG\G0318001	Girder Bridge; Without Footpath; Load H20; S18.0m;
101	BRDG\G0318501	Girder Bridge; Without Footpath; Load H20; S18.5m;
102	BRDG\G0319001	Girder Bridge; Without Footpath; Load H20; S19.0m;
103	BRDG\G0319501	Girder Bridge; Without Footpath; Load H20; S19.5m;
104	BRDG\G0320001	Girder Bridge; Without Footpath; Load H20; S20.0m;
105	BRDG\G1206001	Girder Bridge; With Footpath; Load H15; S6.0m;
106	BRDG\G1206501	Girder Bridge; With Footpath; Load H15; S6.5m;
107	BRDG\G1207001	Girder Bridge; With Footpath; Load H15; S7.0m;
108	BRDG\G1207501	Girder Bridge; With Footpath; Load H15; S7.5m;
109	BRDG\G1208001	Girder Bridge; With Footpath; Load H15; S8.0m;
110	BRDG\G1208501	Girder Bridge; With Footpath; Load H15; S8.5m;
111	BRDG\G1209001	Girder Bridge; With Footpath; Load H15; S9.0m;
112	BRDG\G1209501	Girder Bridge; With Footpath; Load H15; S9.5m;
113	BRDG\G1210001	Girder Bridge; With Footpath; Load H15; S10.0m;
114	BRDG\G1210501	Girder Bridge; With Footpath; Load H15; S10.5m;
115	BRDG\G1211001	Girder Bridge; With Footpath; Load H15; S11.0m;
116	BRDG\G1211501	Girder Bridge; With Footpath; Load H15; S11.5m;
117	BRDG\G1212001	Girder Bridge; With Footpath; Load H15; S12.0m;
118	BRDG\G1212501	Girder Bridge; With Footpath; Load H15; S12.5m;
119	BRDG\G1213001	Girder Bridge; With Footpath; Load H15; S13.0m;
120	BRDG\G1213501	Girder Bridge; With Footpath; Load H15; S13.5m;
121	BRDG\G1214001	Girder Bridge; With Footpath; Load H15; S14.0m;
122	BRDG\G1214501	Girder Bridge; With Footpath; Load H15; S14.5m;
123	BRDG\G1215001	Girder Bridge; With Footpath; Load H15; S15.0m;
124	BRDG\G1215501	Girder Bridge; With Footpath; Load H15; S15.5m;
125	BRDG\G1216001	Girder Bridge; With Footpath; Load H15; S16.0m;
126	BRDG\G1216501	Girder Bridge; With Footpath; Load H15; S16.5m;

Sl. No.	Reference Code	Structure Description
127	BRDG\G1217001	Girder Bridge; With Footpath; Load H15; S17.0m;
128	BRDG\G1217501	Girder Bridge; With Footpath; Load H15; S17.5m;
129	BRDG\G1218001	Girder Bridge; With Footpath; Load H15; S18.0m;
130	BRDG\G1218501	Girder Bridge; With Footpath; Load H15; S18.5m;
131	BRDG\G1219001	Girder Bridge; With Footpath; Load H15; S19.0m;
132	BRDG\G1219501	Girder Bridge; With Footpath; Load H15; S19.5m;
133	BRDG\G1220001	Girder Bridge; With Footpath; Load H15; S20.0m;
134	BRDG\G1306001	Girder Bridge; With Footpath; Load H20; S6.0m;
135	BRDG\G1306501	Girder Bridge; With Footpath; Load H20; S6.5m;
136	BRDG\G1307001	Girder Bridge; With Footpath; Load H20; S7.0m;
137	BRDG\G1307501	Girder Bridge; With Footpath; Load H20; S7.5m;
138	BRDG\G1308001	Girder Bridge; With Footpath; Load H20; S8.0m;
139	BRDG\G1308501	Girder Bridge; With Footpath; Load H20; S8.5m;
140	BRDG\G1309001	Girder Bridge; With Footpath; Load H20; S9.0m;
141	BRDG\G1309501	Girder Bridge; With Footpath; Load H20; S9.5m;
142	BRDG\G1310001	Girder Bridge; With Footpath; Load H20; S10.0m;
143	BRDG\G1310501	Girder Bridge; With Footpath; Load H20; S10.5m;
144	BRDG\G1311001	Girder Bridge; With Footpath; Load H20; S11.0m;
145	BRDG\G1311501	Girder Bridge; With Footpath; Load H20; S11.5m;
146	BRDG\G1312001	Girder Bridge; With Footpath; Load H20; S12.0m;
147	BRDG\G1312501	Girder Bridge; With Footpath; Load H20; S12.5m;
148	BRDG\G1313001	Girder Bridge; With Footpath; Load H20; S13.0m;
149	BRDG\G1313501	Girder Bridge; With Footpath; Load H20; S13.5m;
150	BRDG\G1314001	Girder Bridge; With Footpath; Load H20; S14.0m;
151	BRDG\G1314501	Girder Bridge; With Footpath; Load H20; S14.5m;
152	BRDG\G1315001	Girder Bridge; With Footpath; Load H20; S15.0m;
153	BRDG\G1315501	Girder Bridge; With Footpath; Load H20; S15.5m;
154	BRDG\G1316001	Girder Bridge; With Footpath; Load H20; S16.0m;
155	BRDG\G1316501	Girder Bridge; With Footpath; Load H20; S16.5m;
156	BRDG\G1317001	Girder Bridge; With Footpath; Load H20; S17.0m;
157	BRDG\G1317501	Girder Bridge; With Footpath; Load H20; S17.5m;
158	BRDG\G1318001	Girder Bridge; With Footpath; Load H20; S18.0m;
159	BRDG\G1318501	Girder Bridge; With Footpath; Load H20; S18.5m;
160	BRDG\G1319001	Girder Bridge; With Footpath; Load H20; S19.0m;
161	BRDG\G1319501	Girder Bridge; With Footpath; Load H20; S19.5m;
162	BRDG\G1320001	Girder Bridge; With Footpath; Load H20; S20.0m;

Structure List of Brick Abutment

Sl. No.	Reference Code	Structure Description
1	ABUT\B0010301	Abutment Brick; Range 1; H 3.0m;
2	ABUT\B0010331	Abutment Brick; Range 1; H 3.3m;
3	ABUT\B0010361	Abutment Brick; Range 1; H 3.6m;
4	ABUT\B0010401	Abutment Brick; Range 1; H 4.0m;
5	ABUT\B0010431	Abutment Brick; Range 1; H 4.3m;
6	ABUT\B0010461	Abutment Brick; Range 1; H 4.6m;
7	ABUT\B0010501	Abutment Brick; Range 1; H 5.0m;
8	ABUT\B0010531	Abutment Brick; Range 1; H 5.3m;
9	ABUT\B0010561	Abutment Brick; Range 1; H 5.6m;
10	ABUT\B0010601	Abutment Brick; Range 1; H 6.0m;

Structure List of Full Depth Abutment

Sl. No.	Reference Code	Structure Description
1	ABUT\R0110301	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 3.0m;
2	ABUT\R0110331	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 3.3m;
3	ABUT\R0110361	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 3.6m;
4	ABUT\R0110401	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 4.0m;
5	ABUT\R0110431	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 4.3m;
6	ABUT\R0110461	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 4.6m;
7	ABUT\R0110501	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 5.0m;
8	ABUT\R0110531	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 5.3m;
9	ABUT\R0110561	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 5.6m;
10	ABUT\R0110601	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 6.0m;
11	ABUT\R0110631	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 6.3m;
12	ABUT\R0110661	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 6.6m;
13	ABUT\R0110701	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 7.0m;
14	ABUT\R0110731	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 7.3m;
15	ABUT\R0110761	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 7.6m;
16	ABUT\R0110801	Abutment Full Depth; Without Footpath; CW 2.44m; Range 1; H 8.0m;
17	ABUT\R0210301	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 3.0m;
18	ABUT\R0210331	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 3.3m;
19	ABUT\R0210361	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 3.6m;
20	ABUT\R0210401	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 4.0m;
21	ABUT\R0210431	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 4.3m;
22	ABUT\R0210461	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 4.6m;

Sl. No.	Reference Code	Structure Description
23	ABUT\R0210501	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 5.0m;
24	ABUT\R0210531	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 5.3m;
25	ABUT\R0210561	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 5.6m;
26	ABUT\R0210601	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 6.0m;
27	ABUT\R0210631	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 6.3m;
28	ABUT\R0210661	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 6.6m;
29	ABUT\R0210701	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 7.0m;
30	ABUT\R0210731	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 7.3m;
31	ABUT\R0210761	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 7.6m;
32	ABUT\R0210801	Abutment Full Depth; Without Footpath; CW 3.66m; Range 1; H 8.0m;
33	ABUT\R1210301	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 3.0m;
34	ABUT\R1210331	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 3.3m;
35	ABUT\R1210361	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 3.6m;
36	ABUT\R1210401	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 4.0m;
37	ABUT\R1210431	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 4.3m;
38	ABUT\R1210461	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 4.6m;
39	ABUT\R1210501	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 5.0m;
40	ABUT\R1210531	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 5.3m;
41	ABUT\R1210561	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 5.6m;
42	ABUT\R1210601	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 6.0m;
43	ABUT\R1210631	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 6.3m;
44	ABUT\R1210661	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 6.6m;
45	ABUT\R1210701	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 7.0m;

Sl. No.	Reference Code	Structure Description
46	ABUT\R1210731	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 7.3m;
47	ABUT\R1210761	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 7.6m;
48	ABUT\R1210801	Abutment Full Depth; With Footpath; CW 3.66m ; Range 1; H 8.0m;
49	ABUT\R0120301	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 3.0m;
50	ABUT\R0120331	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 3.3m;
51	ABUT\R0120361	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 3.6m;
52	ABUT\R0120401	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 4.0m;
53	ABUT\R0120431	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 4.3m;
54	ABUT\R0120461	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 4.6m;
55	ABUT\R0120501	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 5.0m;
56	ABUT\R0120531	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 5.3m;
57	ABUT\R0120561	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 5.6m;
58	ABUT\R0120601	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 6.0m;
59	ABUT\R0120631	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 6.3m;
60	ABUT\R0120661	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 6.6m;
61	ABUT\R0120701	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 7.0m;
62	ABUT\R0120731	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 7.3m;
63	ABUT\R0120761	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 7.6m;
64	ABUT\R0120801	Abutment Full Depth; Without Footpath; CW 2.44m; Range 2; H 8.0m;
65	ABUT\R0220301	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 3.0m;
66	ABUT\R0220331	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 3.3m;
67	ABUT\R0220361	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 3.6m;
68	ABUT\R0220401	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 4.0m;

Sl. No.	Reference Code	Structure Description
69	ABUT\R0220431	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 4.3m;
70	ABUT\R0220461	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 4.6m;
71	ABUT\R0220501	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 5.0m;
72	ABUT\R0220531	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 5.3m;
73	ABUT\R0220561	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 5.6m;
74	ABUT\R0220601	Abutment Full Depth; Without Foo Range 2; H 6.0m;
75	ABUT\R0220631	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 6.3m;
76	ABUT\R0220661	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 6.6m;
77	ABUT\R0220701	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 7.0m;
78	ABUT\R0220731	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 7.3m;
79	ABUT\R0220761	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 7.6m;
80	ABUT\R0220801	Abutment Full Depth; Without Footpath; CW 3.66m; Range 2; H 8.0m;
81	ABUT\R1220301	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 3.0m;
82	ABUT\R1220331	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 3.3m;
83	ABUT\R1220361	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 3.6m;
84	ABUT\R1220401	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 4.0m;
85	ABUT\R1220431	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 4.3m;
86	ABUT\R1220461	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 4.6m;
87	ABUT\R1220501	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 5.0m;
88	ABUT\R1220531	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 5.3m;
89	ABUT\R1220561	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 5.6m;
90	ABUT\R1220601	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 6.0m;
91	ABUT\R1220631	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 6.3m;

Sl. No.	Reference Code	Structure Description
92	ABUT\R1220661	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 6.6m;
93	ABUT\R1220701	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 7.0m;
94	ABUT\R1220731	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 7.3m;
95	ABUT\R1220761	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 7.6m;
96	ABUT\R1220801	Abutment Full Depth; With Footpath; CW 3.66m; Range 2; H 8.0m;
97	ABUT\P0000001	Detailed of Cast-in-situ and precast pile

Structure List of R.C.C Open Abutment

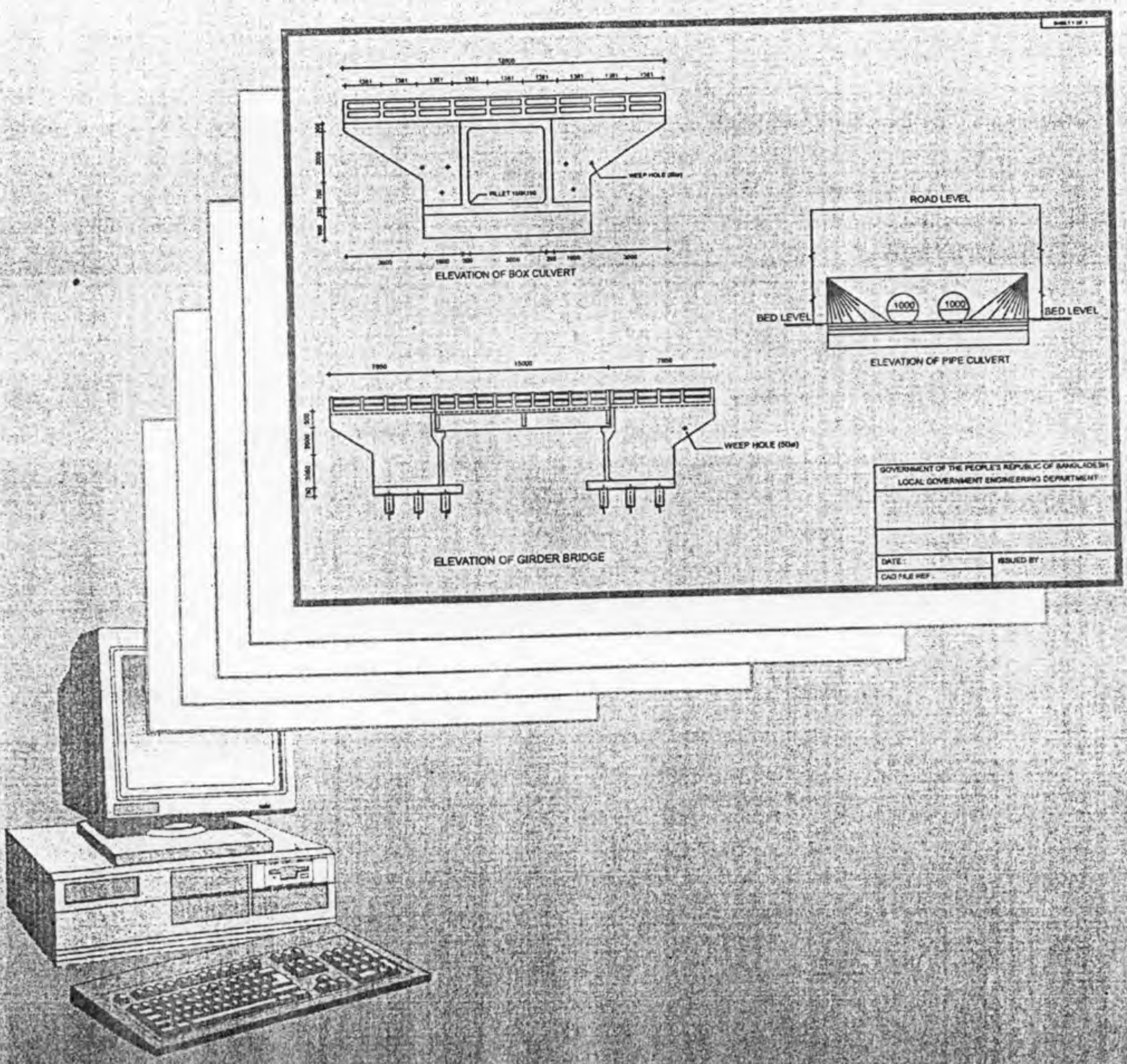
Sl. No.	Reference Code	Structure Description
1	ABUT\O1210301	Abutment Open; With Footpath; CW 3.66m; Range 1; H3.0m;
2	ABUT\O1210331	Abutment Open; With Footpath; CW 3.66m; Range 1; H3.3m;
3	ABUT\O1210361	Abutment Open; With Footpath; CW 3.66m; Range 1; H3.6m;
4	ABUT\O1210401	Abutment Open; With Footpath; CW 3.66m; Range 1; H4.0m;
5	ABUT\O1210431	Abutment Open; With Footpath; CW 3.66m; Range 1; H4.3m;
6	ABUT\O1210461	Abutment Open; With Footpath; CW 3.66m; Range 1; H4.6m;
7	ABUT\O1210501	Abutment Open; With Footpath; CW 3.66m; Range 1; H5.0m;
8	ABUT\O1210531	Abutment Open; With Footpath; CW 3.66m; Range 1; H5.3m;
9	ABUT\O1210561	Abutment Open; With Footpath; CW 3.66m; Range 1; H5.6m;
10	ABUT\O1210601	Abutment Open; With Footpath; CW 3.66m; Range 1; H6.0m;
11	ABUT\O1220301	Abutment Open; With Footpath; CW 3.66m; Range 2; H3.0m;
12	ABUT\O1220331	Abutment Open; With Footpath; CW 3.66m; Range 2; H3.3m;
13	ABUT\O1220361	Abutment Open; With Footpath; CW 3.66m; Range 2; H3.6m;
14	ABUT\O1220401	Abutment Open; With Footpath; CW 3.66m; Range 2; H4.0m;
15	ABUT\O1220431	Abutment Open; With Footpath; CW 3.66m; Range 2; H4.3m;
16	ABUT\O1220461	Abutment Open; With Footpath; CW 3.66m; Range 2; H4.6m;
17	ABUT\O1220501	Abutment Open; With Footpath; CW 3.66m; Range 2; H5.0m;
18	ABUT\O1220531	Abutment Open; With Footpath; CW 3.66m; Range 2; H5.3m;
19	ABUT\O1220561	Abutment Open; With Footpath; CW 3.66m; Range 2; H5.6m;
20	ABUT\O1120601	Abutment Open; With Footpath; CW 3.66m; Range 2; H6.0m;
21	ABUT\O0110301	Abutment Open; Without Footpath; CW 2.44m; Range 1; H3.0m;
22	ABUT\O0110331	Abutment Open; Without Footpath; CW 2.44m; Range 1; H3.3m;
23	ABUT\O0110361	Abutment Open; Without Footpath; CW 2.44m; Range 1; H3.6m;
24	ABUT\O0110401	Abutment Open; Without Footpath; CW 2.44m; Range 1; H4.0m;
25	ABUT\O0110431	Abutment Open; Without Footpath; CW 2.44m; Range 1; H4.3m;
26	ABUT\O0110461	Abutment Open; Without Footpath; CW 2.44m; Range 1; H4.6m;
27	ABUT\O0110501	Abutment Open; Without Footpath; CW 2.44m; Range 1; H5.0m;
28	ABUT\O0110531	Abutment Open; Without Footpath; CW 2.44m; Range 1; H5.3m;
29	ABUT\O0110561	Abutment Open; Without Footpath; CW 2.44m; Range 1; H5.6m;
30	ABUT\O0110601	Abutment Open; Without Footpath; CW 2.44m; Range 1; H6.0m;
31	ABUT\O0210301	Abutment Open; Without Footpath; CW 3.66m; Range 1; H3.0m;
32	ABUT\O0210331	Abutment Open; Without Footpath; CW 3.66m; Range 1; H3.3m;
33	ABUT\O0210361	Abutment Open; Without Footpath; CW 3.66m; Range 1; H3.6m;
34	ABUT\O0210401	Abutment Open; Without Footpath; CW 3.66m; Range 1; H4.0m;
35	ABUT\O0210431	Abutment Open; Without Footpath; CW 3.66m; Range 1; H4.3m;
36	ABUT\O0210461	Abutment Open; Without Footpath; CW 3.66m; Range 1; H4.6m;
37	ABUT\O0210501	Abutment Open; Without Footpath; CW 3.66m; Range 1; H5.0m;
38	ABUT\O0210531	Abutment Open; Without Footpath; CW 3.66m; Range 1; H5.3m;
39	ABUT\O0210561	Abutment Open; Without Footpath; CW 3.66m; Range 1; H5.6m;
40	ABUT\O0210601	Abutment Open; Without Footpath; CW 3.66m; Range 1; H6.0m;
41	ABUT\O0120301	Abutment Open; Without Footpath; CW 2.44m; Range 2; H3.0m;
42	ABUT\O0120331	Abutment Open; Without Footpath; CW 2.44m; Range 2; H3.3m;
43	ABUT\O0120361	Abutment Open; Without Footpath; CW 2.44m; Range 2; H3.6m;
44	ABUT\O0120401	Abutment Open; Without Footpath; CW 2.44m; Range 2; H4.0m;

Sl. No.	Reference Code	Structure Description
45	ABUT\O0120431	Abutment Open; Without Footpath; CW 2.44m; Range 2; H4.3m;
46	ABUT\O0120461	Abutment Open; Without Footpath; CW 2.44m; Range 2; H4.6m;
47	ABUT\O0120501	Abutment Open; Without Footpath; CW 2.44m; Range 2; H5.0m;
48	ABUT\O0120531	Abutment Open; Without Footpath; CW 2.44m; Range 2; H5.3m;
49	ABUT\O0120561	Abutment Open; Without Footpath; CW 2.44m; Range 2; H5.6m;
50	ABUT\O0120601	Abutment Open; Without Footpath; CW 2.44m; Range 2; H6.0m;
51	ABUT\O0220301	Abutment Open; Without Footpath; CW 3.66m; Range 2; H3.0m;
52	ABUT\O0220331	Abutment Open; Without Footpath; CW 3.66m; Range 2; H3.3m;
53	ABUT\O0220361	Abutment Open; Without Footpath; CW 3.66m; Range 2; H3.6m;
54	ABUT\O0220401	Abutment Open; Without Footpath; CW 3.66m; Range 2; H4.0m;
55	ABUT\O0220431	Abutment Open; Without Footpath; CW 3.66m; Range 2; H4.3m;
56	ABUT\O0220461	Abutment Open; Without Footpath; CW 3.66m; Range 2; H4.6m;
57	ABUT\O0220501	Abutment Open; Without Footpath; CW 3.66m; Range 2; H5.0m;
58	ABUT\O0220531	Abutment Open; Without Footpath; CW 3.66m; Range 2; H5.3m;
59	ABUT\O0220561	Abutment Open; Without Footpath; CW 3.66m; Range 2; H5.6m;
60	ABUT\O0220601	Abutment Open; Without Footpath; CW 3.66m; Range 2; H6.0m;

Structure List of Stub Abutment

Sl. No.	Reference Code	Structure Description
1	ABUT\S0100001	Abutment Stub; Without Footpath; CW 2.44m; Precast;
2	ABUT\S0200001	Abutment Stub; Without Footpath; CW 3.66m; Precast;
3	ABUT\S1200001	Abutment Stub; With Footpath; CW 3.66m; Precast;
4	ABUT\S0101001	Abutment Stub; Without Footpath; CW 2.44m; Cast-in-situ;
5	ABUT\S0201001	Abutment Stub; Without Footpath; CW 3.66m; Cast-in-situ;
6	ABUT\S1201001	Abutment Stub; With Footpath; CW 3.66m; Cast-in-situ;

ATTACHMENTS



Attachment 'A'

Approximate Site Plan

Attachment 'B'

1) Cross Section Along C.L. of Proposed Structure

2) Cross Section at 25m Upstream

3) Cross Section at 25m Downstream