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MINISTRY OF LOCAL GOVERNMENT, RURAL DEVELOPMENT & CO-OPERATIVES

LOCAL GOVERNMENT ENGINEERING DEPARTMENT

MANUAL ON PRESTRESSED CONCRETE BRIDGES
PART B - DESIGN CRITERIA AND GUIDELINES

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Manual on PC Bridges
Part B - Design Criteria and Guidelines

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CHAPTER 1

INTRODUCTION

The Manual on Prestressed Concrete Bridge, 1996 will enable LGED to implement larger span prestressed concrete bridges in its jurisdiction throughout the country. A considerable number of the LGED's large span bridge sites are located over channels having perennial flow or over tidal creeks and/or in weak foundation soil where construction of bridge superstructure by cast-in-situ method has a risk of scaffolding and formwork settlement. This Manual includes practicable design and construction methodology of bridges at the above-mentioned difficult sites.

The Manual comprises 2 parts which are as follows:

Part A : Standard Designs
Part B : Design Criteria and Guidelines

The Part A of this Manual contains types and geometry of bridge decks and types of PC girders, abutment-wing-tie wall arrangement, pile cap and piles. It also includes flow charts for the selection of span length, height of abutment and length of piles.

The main and important contents of Part A are all standard drawings consisting of concrete outline and reinforcement details of deck slab and its accessories, prestressed concrete girder, RC cross-girder, abutment-wing- tie wall, pile cap and piles. Different types and sizes of reinforced elastomeric bearings and expansion joints are also included in this part of the Manual.

Standard items of work applicable for construction of PC bridge have been included from the LGED rate schedule. Bill of quantities for all reinforced concrete (RC) and prestressed concrete (PC) works along with the bar bending schedules of each RC component have been furnished in this part of the Manual.

The Part B of this Manual contains standard definition of technical terms used in this Manual, a comprehensive details of the technical considerations for selection of a PC bridge at a particular site, a comprehensive structural analysis and design procedure following the provisions of the AASHTO'92 Standards. Guidelines for implementation and for providing vertical curve profile of the PC bridge across the channel at the selected site are also included.

The primary objective of the Part B is to enhance the capabilities of the engineers of LGED to design and implement larger span PC bridges under their jurisdictions throughout the country. Detailed design criteria and step-by-step procedure for designing different components of bridge superstructure, substructure, foundation and reinforced elastomeric bearing by following the flow charts and using computer software have been furnished in this part of the Manual. Detailed guidelines along with in-country standard construction practice for implementation of different components of a PC bridge with additional and special details concerning PC girder have been incorporated in this part of the Manual. As construction of post-tensioned PC girder involves stressing of cables, grouting of cable ducts and erection /placement (in case of precast girders) which require particular technical expertise, higher control of material qualities and workmanship. A comprehensive details of all these issues have also been included in this part of the Manual.

CHAPTER 2

DEFINITIONS

The following definitions shall be applicable for the purpose of this Manual.

Bridge

Bridge is a structure for carrying the road traffic or other moving loads over a depression or obstruction such as channel, road or railway.

Culvert

Culvert is a bridge having a gross length of six meters or less between the faces of abutments or extreme ventway boundaries and measured at right angles thereto.

Foot Bridge

The foot bridge is a bridge exclusively used for carrying pedestrians, cycles and animals.

Channel

A channel means a natural or artificial water course.

Clearance

Clearance is the shortest distance between boundaries at a specified position of a bridge structure.

Highest Flood Level

Highest flood level is the measured or calculated level for the highest possible flood.

Design High Flood Level

Design high flood level corresponds to a HFL against certain return period used for design purpose. For the purpose of this Manual, design HFL shall correspond to 50 years return period flood level.

Low Water Level (LWL)

The low water level is the level of the water surface generally obtained in the dry season and it shall be specified in case of each bridge.

Length of a Bridge

The length of a bridge structure will be taken as the overall length measured along the centre-line of the bridge from end to end of the bridge deck.

Linear Waterway

The linear waterway of a bridge shall be the length available in the bridge between the extreme edges of water surface at the HFL measured at right angles to the abutment faces.

Effective Linear Waterway

Effective linear waterway is the total width of the waterway of the bridge minus the effective width of obstruction.

Load Bearing Abutment

Load bearing abutment is an abutment which supports span of a bridge.

Width of Footpath/Sidewalk

The width of footpath or sidewalk shall be taken as the minimum clear width anywhere within a height of 0.25 meters above the surface of the deck, such width being measured at right angles to the longitudinal centre line of the bridge.

Carriageway

The carriageway width is the minimum clear width measured at right angles to the longitudinal centre line of the bridge between the inside faces of roadway curbs or wheel guards.

Afflux

The rise in the flood level of the river immediately on the upstream of a bridge as a result of obstruction to natural flow caused by the construction of the bridge and its approaches.

Cofferdam

A structure, usually temporary, built for the purpose of excluding water or soil sufficiently to permit construction or proceed without excessive pumping and to support the surrounding ground.

Foundation

The part of a bridge in direct contract with and transmitting loads to the ground.

Piles

i) Bearing Pile

A pile driven or cast-in-situ for transmitting the weight of a structure to the soil by the resistance developed at the pile base and by friction along its surface. If it supports the load mainly by the resistance developed at its base, it is referred to as an end bearing pile, and if mainly by friction along its surface, it is referred to as a friction pile.

ii) Bored Cast-in-Situ Pile

A pile formed with or without a casing by excavating or boring a hole in the ground and subsequently filling it with plain or reinforced concrete.

iii) Driven Pile

A pile driven into the ground by the blows of a hammer or by a vibrator.

iv) Driven Cast-in-Place

A pile formed in the ground by driving a permanent or temporary casing, and filling it with plain or reinforced concrete.

v) Precast Pile

A reinforced or prestressed concrete pile cast before driving.

vi) Raker or Batter Pile

A pile installed at an inclination to the vertical.

vii) Sheet Pile

One or a row of piles driven or formed in the ground adjacent to one another in a continuous wall, each generally provided with a connecting joint or inter-lock, designed to resist mainly lateral forces and to reduce seepage; it may be vertical or inclined.

viii) Test Pile

A pile to which load is applied to determine the load/settlement characteristics of the pile and the surrounding ground.

ix) Working Pile

Piles forming the foundation of a structure.

Substructure

The portion of the bridge structure such as piers and abutments above the foundation unit and supporting the superstructure. It shall also include return and wing walls but exclude bearings.

Well Foundation (Caisson)

A structure which is generally built in parts and sunk through ground or water to the prescribed depth and which subsequently becomes an integral part of the permanent foundation.

Pneumatic Caisson

A caisson with a working chamber in which air pressure is maintained to a level of human tolerance and to prevent the entry of water and surrounding soil into the excavation.

CHAPTER 3

PLANNING AND INVESTIGATION

3.1 General

Planning and investigation comprise the initial stage activities of the project. The following activities are done at this stage:

- From the preliminary investigation it is ascertained whether a drainage structure is at all required at the proposed site and if required, what type of structure should it be, for example, culvert or bridge ?
- Criteria for selection of the location of the bridge, type of deck e.g., carriageway width, out to out deck width, with or without footpath, RC or PC girders, etc. are decided.
- General data e.g., existing index map and contour survey plans and hydro-meteorological data are collected.
- Topographical survey e.g. preparation of site plan, cross-section surveys are done.
- Alternative and particular bridge sites are selected.
- Geological data for the particular bridge site is collected and assessed.
- Bridge Loading and other traffic data are collected.
- Vertical and horizontal clearances of the bridge, hydraulic studies and sub-soil investigation are conducted at this stage.

The following sub-sections and the subsequent Chapters 4.0 and 5.0 give detailed guidelines and requirements for obtaining and assessing the above data.

3.2 Criteria for Selection of the Type and Location of Bridge

3.2.1 When PC Bridge Shall Be Provided

If no defined drainage channel exists or if the drainage channel is ill-defined the type of structure may be either culvert or small RC bridge of RSM '96. In case of well-defined channel if the selected bridge span is less than 20 m, then also RC bridge of RSM '96 may be used.

If the selected bridge span is 20 m or larger then the PC bridge of this Manual may be provided.

3.2.2 Location of the Bridge

For location of the bridge site the following criteria should be followed:

- Alternative bridge sites should be identified.
- The bridge should preferably be located in a straight reach of channel. For a meandered channel it should be located on the crossing between the bends.
- The approach roads on both sides should be straight for a considerable distance on both banks so that normal bridge can be provided and bank erosion is negligible.
- The abutments of the bridge should be located on a stable bank.
- In case of proximity of the bridge alignment to the tributory which is the offtake of the channel , the bridge alignment should be located away from the disturbing zone of the offtake channel.
- The bridge location should be such that the deep foundation i.e.' piles can be placed in a good bearing strata at a reasonable depth. In case good bearing strata are not available at a reasonable

depth, the bridge alignment should be shifted to a suitable alternative location upstream or downstream where good bearing strata is available.

3.2.3 Type of Deck

Four types of deck are provided in the standard design of this Manual.

Type IA	:	Carriageway width 3.66 m plus 0.4 m wide foot path on each side.
Type IB	:	Carriageway width 3.66 m without foot path.
Type IIA	:	Carriageway width 4.33 m plus 0.4 m wide footpath on each side.
Type IIB	:	Carriageway width 4.33 m without footpath.

Further details are given in Part A, Art. 1.2 of this Manual.

Type IA deck with 3.66 m carriageway plus 0.4 m foot path on each side shall be provided at remote areas away from the thana head quarters, important market places, schools and colleges where vehicular traffic volume is less and pedestrian traffic volume is considerable.

Type IB deck with 3.66 m carriageway but without foot path shall be provided in the rural road in remote areas where vehicular traffic volume is less and pedestrian traffic volume is negligible.

Type IIA deck with 4.33 m carriageway plus 0.4 m footpath on each side shall be provided for rural bridge in remote areas nearer to the thana headquarters, important market places, schools and colleges where both vehicular and pedestrian traffic volume are considerable at present or are likely to generate traffic after construction of the bridge.

Type IIB deck without footpath shall be provided in rural roads away from the thana headquarters, important market places, schools, colleges or other places of congregation where vehicular traffic volume is considerable but pedestrian traffic volume is negligible.

3.3 General and Topographical Survey Data

All detailed information shall be included in the project-documents for a complete and proper appreciation of the bridge project. Generally, the following information shall be furnished:

3.3.1 General Data Including Maps, Plans and Topographical Features

a. Index Map

An index map to a suitable small scale (topo sheets scale one cm to 500 m or 1/50,000 would do in most cases) showing the proposed location of the bridge, the alternative sites investigated and rejected, the existing means of communications, the general topography of the country, and the important towns, etc., in the vicinity.

b. Contour Survey Plan

A contour survey plan of the stream showing all topographical features and extending upstream and downstream of any of the proposed sites, to the distances shown below, (or such other greater distances as the engineer responsible for the design may direct) and a sufficient distance on either side to give a clear indication of the topographical or other features that might influence the location and design of the bridge and its approaches. All sites for crossing worth consideration shall be shown on the plan.

- 100 m for catchment areas less than 3 square km (scale not less than one cm to 10 m or 1/1000).
- 300 m for catchment areas of 3 to 15 square km (scale not less than one cm to 10 m or 1/1000).
- one and a half km for catchment areas of more than 15 square km (scale not less than one cm to 50 m or 1/5000).

c. **Site Plan**

A site plan to a suitable scale showing details of the site selected and extending not less than 100 meters uptream and downstream from the centre line of the crossing and covering the approaches to a sufficient distance which, in the case of a large bridge, shall not be less than 500 m on either side of the channel. The following information shall be indicated on the site plan.

- The name of the channel or bridge and of the road and the identification mark allotted to the crossing, with the location (in kilometers) of the centre of crossing.
- The direction of flow of water and maximum discharge and, if possible, the extent of deviation at lower discharges.
- The alignment of existing approaches and of the proposed crossing and its approaches.
- The angle and direction of skew if the crossing is aligned on a skew.
- The name of the nearest inhabited identifiable locality at either end of the crossing on the roads leading to the site.
- References to the position (with description and reduced level preferably in mPWD) of the bench mark used as datum.
- The lines and identification numbers of the cross section and longitudinal section taken within the scope of the site plan, and the exact location of their extreme points.
- The location of trial pits or borings each being given an identification number and connected to the datum.
- The location of all nullahs, buildings, wells, outcrops of rocks, if any and other possible obstructions, to a road alignment.

d. **Cross-Section Survey**

Cross-section of the channel at the site of the proposed crossing and two other cross-sections at suitable distances, one upstream and the other downstream, all to the horizontal scale of not less than one cm to 10 m or 1/1000 and with an exaggerated vertical scale of not less than 1 cm to 1 m or 1/100 and indicating the following information:

- The bed levels upto the top of banks and the ground levels to a sufficient distance beyond the edges of the channel, with levels at intervals sufficiently close to give a clear outline of markedly uneven features of the bed or ground showing right and left banks and names of villages on each side.
- The nature of the existing surface soil in bed, banks and approaches, and the location and depth of trial pits or borings with their respective identification numbers.
- The highest flood level and the low water level.
- For tidal streams, record of the tidal information, over as long a period as possible, including any local information specific to the site of work. The form given below is recommended for presenting such a records.

- Highest high water (HHW)
- Mean high water springs (MHWS)
- Mean high water (MHW)
- Mean high water neaps (MHWN)
- Mean sea level (MSL)
- Mean low water (MLW)
- Mean low water springs (MLWS) called Chart Datum
- Lowest low water (LLW)

- A few cross-sections, in addition to those required, upstream and downstream of the proposed site of the bridge, with both the horizontal and vertical scales being the same as the horizontal scale adopted for the cross-sections required as given above.
- A longitudinal section of the channel, showing the site of the bridge with the highest flood level, the low water level, (also the highest high tide level and the lowest low tide level for tidal channels), and the bed levels at suitably spaced intervals along the approximate centre-line of the deep water channel between the approximate points to which the survey plan as given above extends. The horizontal scale shall be the same as for the survey plan and the vertical scale not less than one cm to 10 m or 1/1000.

3.4 Alternative and Particular Bridge Sites

A brief description of the reasons for selection of a particular site for the crossing accompanied, if necessary, with typical cross-sections of the channel at alternative sites investigated and rejected.

3.5 Hydraulic Data

- The size, shape and surface characteristics of the catchment including percolation and interception.

- The possibility of subsequent changes in the catchment like afforesation, deforesation, urban development, extension of or deduction in cultivated area, etc.

- Storage in the catchment, artificial or natural.

- The intensity and frequency of rainfall in the catchment.

- The slope of the catchment, both longitudinal and cross directional.

- Hydrographs for one or more years, if possible, and in the absence of such data, fluctuations of the water level observed during different months of the year.

- The highest flood level and the year of its occurrence. If the flood level is affected by backwater, details of the same.

- A chart of the periods of high flood levels for as many years as the relevant data has been recorded.

- The influence of afflux on areas in the vicinity likely to be affected.

- Low water level.

- The design discharge, the linear waterway and corresponding average velocity of flow.
- The observed maximum depth of scour with corresponding level and details of obstruction or any other special causes responsible for the scour.

3.6 Geological Data

- The nature and properties of the existing soil in bed, banks and approaches.
- Susceptibility of the site to earthquake disturbances and its magnitude.

3.7 Climatic Data

Information regarding usual annual temperature range, susceptibility to severe storms, cyclones, etc., and probable wind velocity, rainfall characteristics indicating period of rainy seasons, relative humidity and salinity or presence of harmful chemicals in the atmosphere.

3.8 Loading and Other Data

- The bridge will be designed for the live load as per relevant clauses of AASHTO Standard Specifications for Highway Bridges, 1992. Any specific variation from those clauses, if required, shall be covered by special load conditions.
- Special local conditions, like traffic intensity and pattern to enable the designer to fix the loading to be adopted for the footpath and to fix deck types required.
- Utilities or services, if any, to be provided for and if so nature thereof (e.g. telephone cables, water supply pipes, gas pipes, etc.) and relevant information regarding size, arrangement, etc.
- The minimum vertical and horizontal clearances required for any special requirement like, navigation, raising of the bed, etc., and the basis on which it is suggested.
- An index map showing location of rail and road bridges, if any, crossing the same channel or its tributaries within a reasonable distance of the proposed bridge and a note (with sketches or drawings) giving important details of such bridges.
- A note stating whether large trees and rolling debris, etc. are likely to float down the channel at the proposed bridge site.
- Any other additional information which may be considered essential for complete and proper appreciation of the project.

3.9 Guidelines for Providing Vertical Curves for Multiple Simple Span Bridge

3.9.1 Vertical Curve

The approach roads of a bridge sometimes need to be raised gradually at a certain gradient to provide adequate clearance for the river traffic. A horizontal bridge on a gradually ascending approach road cause an abrupt change in slope. To avoid this, the finished level of the bridge, and if necessary a part the approach roads, are made to lie on a smooth vertical curve. The vertical curve is usually the arc of a parabola.

Bullock carts, bicycles and cycle rickshaws have difficulty in going up a steep grade. It is very dangerous for bullock carts to go down a steep grade since they do not have any mechanical brake. A 3% grade may be considered comfortable for all types of vehicles. The rate of change in grade on vertical curve should also be very smooth to avoid discomfort to the passenger and damage to the vehicles. If the rate of change in grade is taken as g% per chain of 30 m of curve, then the length L of the curve is given by the equation

$$L = \frac{g1 - g2}{g} \times 30 \text{ m}$$

where, g1% is the ascending grade and g2% is descending grade.

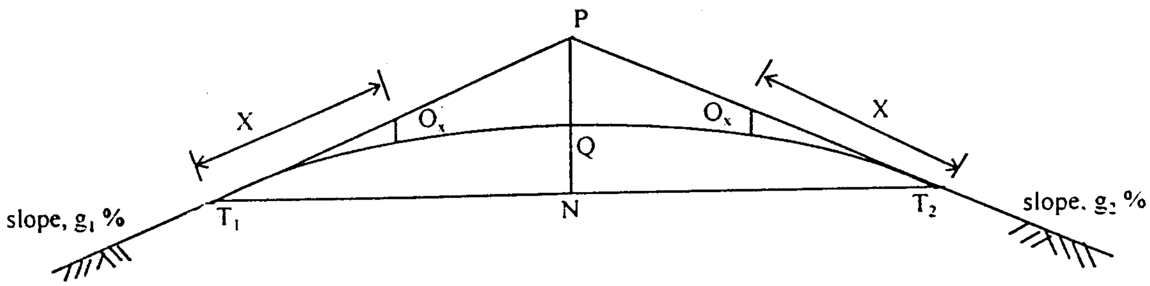


Fig. 3.1 Vertical Curve Joining Two Slopes

The points T_1 and T_2 in Fig. 3.1 are called the first and second tangent points respectively indicating the starting and end of the vertical curve. The ordinates O_x at distance X from the tangent points is given by

$$O_x = PQ \frac{X^2}{T_1Q^2} = 4\ PQ\ (X/L)^2$$

where, PQ = PN/2 = ordinate at the summit point of the curve.

$\therefore$ R.L. of a point at a distance X from T_1 = R.L. of T_1 + (g_1 X/100 - O_x)

where, g_1 is the ascending gradient of the approach road.

The following is an example of three simple span PC girder bridge.

Total length = 3 x 20 m = 60 m

Let the approach road on either side of the 60 m bridge have 3% slope towards the bridge and meet a point P. The rate of change in grade is 1.8% per chain of 30 m of the curve. The R.L. and chainage of the point is 6.5 m PWD and 10+550 respectively.

Total length of vertical curve, $L = \dfrac{3 - (-3)}{1.8} \times 30 = 100$ m

$T_1 P \cong T_1 Q = 100/2 = 50$ m

PN = 3x50/100 = 1.50 m

Chainage of the starting point T_1 or T_2 of the vertical curve = (10+550) - (0+050) = 10+500

R.L. of the starting point T_1 or T_2 = 6.50 - 1.50
= 5.0 m PWD.

PQ = PN/2 = 1.50/2 = 0.75 m

R.L. of any point at a distance X along the tangent at T_1 = R.L. of $T_1 + g_1 X/100$

Ordinate of any point at a distance X along the tangent at T_1 is $O_x = 4\ PQ\ (X/L)^2$

The reduced levels of approach road surface and wearing surface of the bridge at critical locations of the vertical curve are listed in Table 3.1 and the bridge profile is shown in Fig, 3.2

Table 3.1 Computation of R.L. for Vertical Curve Setting

Chainage (Km+m)	Distance (m)	R.L. at X $T_1+O_1X/100$ (m PWD)	Ordinate at X $O_x = 4PQ\ (X/L)^2$ (m)	R.L. of finished surface at X (m PWD)	Remark
10+500	0.000	5.000	0.000	5.000	Vertical curve starts
10+519.175	19.175	5.575	0.110	5.465	Deck elevation at 'a' on abutment
10+529.175	29.175	5.875	0.255	5.620	Deck elevation at mid span 'b'
10+539.175	39.175	6.175	0.460	5.715	Deck elevation at 'c' on pier
10+540	40.000	6.200	0.480	5.720	Deck elevation at 'd' on pier
10+550	50.000	6.500	0.750	5.750	Deck elevation at 'e' of central span
10+560	40.000	6.200	0.480	5.720	
10+560.825	39.175	6.175	0.460	5.715	
10+570.825	29.175	5.875	0.255	5.620	
10+580.825	19.175	5.575	0.110	5.465	
10+600	0.000	5.000	0.000	5.000	

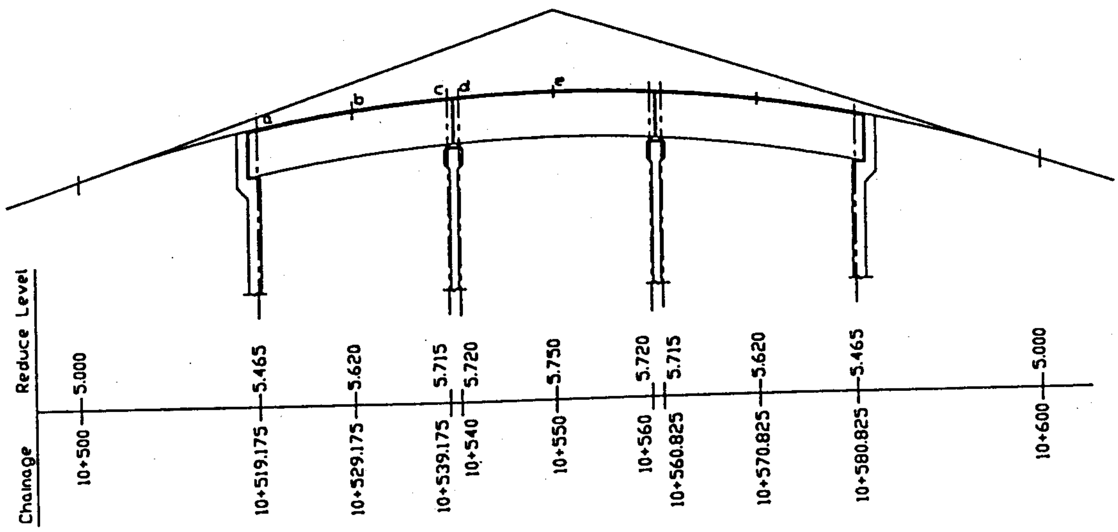


FIG. 3.2 Vertical Curve Setting at Road Structure

3.9.2 Recommendations for Adjustment of the Vertical Curve

- The road levels shown in Fig. 3.2 shall be provided over the top of wearing coarse along centre-line of the bridge.
- The top and bottom level of the deck shall be parallel to the above vertical curve of the profile grade.
- The top and soffit of the PC girder shall also follow the above vertical curve of the profile grade.
- The bottom surface of the PC girder at the locations of the elastomeric bearings shall be made perfectly horizontal either by in-situ girder concrete or by providing precast wedge embedded inside a preformed groove in the PC girder.

CHAPTER 4

HYDRAULIC CONSIDERATIONS AND CLEARANCE

4.1 Design Discharge

The design discharge for which the waterway of the bridge is to be disigned, shall be the maximum flood discharge on record or the estimated maximum discharge for a period of not less than 50 years. In case where the requisite information is not available, the design discharge shall be the maximum estimated discharge determined by consideration of the following or any rational method.

a) From the available records, if any, of observed discharge on the stream at the site of the bridge, or at any other site in its vicinity.

b) From the rainfall and other characteristics of the catchment:

   i) by using an empirical formula applicable to that region, or
   ii) by a recognised method, provided it is possible to evaluate the various factors employed in that method for the region concerned.

c) By the area velocity method with the help of hydraulic characteristics of the channel.

d) By unit hydrograph method.

RSM '96, Chapter-V gives guidelines on the methodology of the above calculations.

Where possible, more than one method shall be adopted, results compared, and the maximum discharge fixed by judgment of the engineer responsible for the design. The bridge shall be designed for this maximum discharge.

Peak flood discharges e.g., exeptional discharges due to the failure of a dam constructed upstream to the bridge need not be catered for and the maximum estimated discharge from the catchment area should be considered for design of the bridge.

4.2 Linear and Effective Linear Waterway

For artficial channels (irrigation, navigation and drainage), the effective linear waterway shall generally be such as to pass the full discharge at normal velocity but concurrence shall invariably be obtained from the authority controlling the channel. If it is proposed to flume the channel at the site of the bridge, this fluming shall be subject to the consent of the same authority and in accordance with the essential requirements.

For non-meandering channels in alluvial beds with well-defined banks and for all natural channel beds with rigid nonerodible boundaries, the linear waterway shall be the distance between banks at that water surface elevation, at which the designed maximum discharge determined can be passed without creating harmful afflux.

For natural channels in alluvial beds and having undefined banks, the effective linear waterway shall be determined from the design discharge, using some accepted rational formula at the discretion of the engineer responsible for the design. One such formula for regime conditions is :

$$W = CQ^{1/2}$$

where W = regime width in meters (equal to effective linear waterway under regime conditions);
Q = the design maximum discharge in m^3/sec;
C = a constant usually taken as 4.8 for regime channels but it may vary from 4.5 to 6.3 according to local conditions.

If the river is of a flashy nature and the bed does not subject readily to the scouring effects of the flood, the waterway should be determined by the area-velocity method taking into account the design flood level and the characteristics of the bed material as well as water surface slope (Ref : RSM'96, Part-A, ch. 5.8).

Where it is decided to adopt measures which are likely to affect the volume of the tidal flow and other characteristics of the tide, it shall be ensured that no port or harbour or other installations in the proximity of the bridge are adversely affected.

For calculating the effective linear waterway, the width of obstruction due to pier(s), if any, shall be taken as the mean submerged width of the pier and its foundation upto the mean scour level. The obstruction at the ends due to the abutments or pitched slopes shall be ignored.

4.3 Effect of Presence of Dams, Weirs, etc.

Presence of dams, barrages, weirs, etc. on the rivers affect the hydraulic characteristics of the rivers like obliquity and concentration of flow, scour, silting of bed, change in bed levels, flood levels, etc. These effects shall be considered in the design of bridges depending upon whether the proposed site of the bridge is upstream or downstream of a dam or a barrage, or weir, etc.

Since the above parameters depend on many factors which are varying from site to site, no uniform guidelines can possibly be laid down. Such problems should be jointly taken up with the concerned Departments and suitable provisions must be made in the bridge design.

4.4 Spacing and Location of Piers and Abutments

Piers and abutments shall be so located as to make the best use of the foundation conditions available. Keeping this in view, the number of supports and their locations shall be so fixed as to provide the most economical design of the bridge and at the same time satisfy special requirements, if any, for navigation, drift timber, railway or other crossing and aesthetics, etc.

The alignment of the piers and abutments shall, as far as possible, be parallel to the mean direction of flow in the channel but provision shall be made against harmful effects on the stability of the bridge structure and on the maintenance of adjacent channel banks caused by any temporary variations in the direction and velocity of the current.

4.5 Vertical Clearance

In the case of a channel, vertical clearance is usually the height from the design highest flood level with afflux of the channel to the lowest point of the bridge superstructure at the position along the bridge where clearance is being denoted.

Clearance shall be allowed according to navigational or anti-obstruction requirements; or where these considerations do not arise, shall ordinarily be as follows :

- For openings of high level bridges, which have a flat soffit or soffit with a very flat curve, the minimum clearance shall be in accordance with Table 4.1. The minimum clearance shall be measured from the lowest point of the deck structure inclusive of main girder in the central half of the clear opening unless otherwise specified subject to the minimum freeboard required by Art. 4.6 .

Table 4.1 Minimum Vertical Clearance

Discharge in m^3/sec	Minimum vertical clearance in mm
Upto 0.3	150
Above 0.3 and upto 3.0	450
Above 3.0 and upto 30.0	600
Above 30.0 and upto 300	900
Above 300 and upto 3000	1200
Above 3000	1500

- In structures provided with elastomeric bearings, no part of the bearings shall be below the design high flood level taking into account the afflux.

- In the case of bridges in sub-mountainous region while fixing the vertical clearance, silting of the bed of the river should also be taken into consideration.

4.6 Freeboard

The freeboard for high level bridges shall not be less than 600 mm

4.7 Restricted Waterways

When the waterway is restricted to such an extent that the resultant afflux will cause the channel to discharge at erosive velocities, protection against damage by scour shall be afforded by deep foundations, curtain or cut-off walls, rip-rap, bed pavement, bearing piles, sheet piles, or other suitable means. Likewise, embankment slopes adjacent to all structures subject to erosion shall be adequately protected by pitching, revetment walls or other suitable construction.

4.8 Obstructions and River Training

Obstructions in the channel bed likely to divert the current or cause undue disturbed flow or scour and thereby endanger the safety of the bridge, shall be removed as far as practicable from within a distance, upstream and downstream of the bridge, of not less than the length of the bridge subject to a minimum of 100 meters in each direction. Attention shall be given to river training and protection of banks over such lengths of the river as required.

4.9 Determination of Scour Depth

4.9.1 General

The probable maximum depth of scour below HFL to be taken for the purpose of designing foundations for piers and abutments, and river training works shall be estimated after considering all local conditions. The following may help the judgment in deciding the maximum scour depth.

a) **Observed Scour Depth:** Wherever possible, soundings for the purpose of determining the depth of scour shall be taken in the vicinity of the site proposed for the bridge. Such soundings are best taken during or immediately after a flood before the scour holes have had time to silt up appreciably. Allowance shall be made in the observed depth for increased scour resulting from :

   i) the deisgn discharge being greater than the flood discharge;
   ii) the increase in velocity due to the obstruction in flow caused by construction of the bridge;
   iii) the increase in scour in the proximity of piers and guide bundhs.

b) **Discharge for Design of Foundations and Protection Works :** To provide for an adequate margin of safety the foundation and protection works shall be designed for a larger discharge which should be a per cent over the design discharge given in Art. 4.1 . This percentage may be 30 per cent for small catchments upto 500 sq. kilimetres, 25 to 20 per cent for medium catchments of 500 to 5000 sq. kilometres, 20 to 10 per cent for large catchments of 5000 to 25000 sq. kilometres and less than 10 per cent for larger catchments above 25000 sq. kilometres at the dicretion of the engineer to cover the possibility of floods of longer return period occurring during the life of the structure.

4.9.2 Mainly Noncohesive Soil

The following theoretical method may be adopted when dealing with natural channels flowing in non-coherent alluvium for the estimation of mean depth of scour `d_{sm}' in meters below the highest flood level:

$$d_{sm} = 1.34(D_b^2/K_{sf})^{1/3}$$

where, D_b = the discharge in cumec per meter width. The value of D_b shall be the maximum of the following :

   i) the total design discharge divided by the effective linear waterway between abutments or giude bunds as the case may be.
   ii) The value obtained taking into account any concentration of flow through a portion of the waterway assessed from the study of the cross section of the river. Such modifications of the value may not be deemed applicable to minor bridges with channel width <20 m.
   iii) The effective linear waterway shall be in accordance with Art. 4.2 . K_{sf} = the silt factor for a representative sample of the bed material obtained upto the level of the deepest anticipated scour and given by the expression $K_{sf} = 1.76\sqrt{d_m}$ where `$d_m$' is the weighted mean diameter in millimeters. The value of `K_{sf}' for bed material normally recommended for various grades of materials as given in IRC:5-1985 are given below in Table 4.2 .

Table 4.2 Lacey's Silt Factor `Ksf'

Type of bed material	d_m weighted mean diameter of particle in mm	Value of `K_{sf}' silt factor
Fine silt	0.081	0.500
Fine silt	0.120	0.600
Fine silt	0.158	0.700
Medium silt	0.232	0.850
Standard silt	0.323	1.000
Medium silt	0.505	1.250
Coarse sand	0.725	1.500
Fine bajri and sand	0.988	1.750
Heavy sand	1.290	2.000

4.9.3 Mainly Cohesive Soil

The resistance of cohesive materials, is more complex and depends on the surface physico-chemical characteristics, density and water quality. The only fairly reliable method of estimating scour is to measure the soil properties and to carry out model tests in a laboratory. Table 4.3 is given as a guide to assist in estimating the mean depth of flow in a cohesive bed channel, based on description of the types and grading of the bed material and on the void ratio(e), defined as the ratio of the volume of voids to the volume of solid material in a mass of soil. The bulk densities in Table 4.3 assume that the specific gravity of the particles is 2.64, and is related to the void ratio by :

dry bulk density = Dw.G/(e+1)

saturated bulk density = Dw.G/(e+1)

where, Dw is the mass density of water, G is the specific gravity of the soil particles, and e is the void ratio of the soil mass.

The depth of flow in a channel may be calculated assuming that scour will occur until a depth is reached such that the tractive stress on the bed equals the critical tractive stress.

Thus, $d_{sm} = 51.4n^{0.86}\ q^{0.86}\ T_c^{-0.43}$

where, dsm is the mean depth of flow, m; n is the coefficient of roughness in Manning's equation; q is the discharge per unit width, $m^3/s/m$; T_c is the critical tractive stress for scour to occur, N/m^2.

Table 4.3 Physical Properties of Clay

Void ratio	2.0-1.2	1.2-0.6	0.6-0.3	0.3-0.2
Dry bulk density, kg/m^3	880-1200	1200-1650	1650-2030	2030-2210
Saturated bulk density, Kg/m^3	1500-1740	1740-2030	2030-2270	2270-2370
Type of soil	Critical tractive stress, N/m^2			
Sandy clay	1.9	7.5	15.7	30.2
Silty clay	1.5	6.7	14.6	27.0
Clay	1.2	5.9	13.5	25.4
Soft clay	1.0	4.6	10.2	16.8

4.9.4 Maximum Depth of Scour

a) The maximum depth of scour below the design HFL at obstructions and configurations of the channel shall be estimated from the value of d_{sm} given in Art. 4.9.2 above and the provisions given below :

For the design of piers and abutments located in a straight reach and having individual foundations without any floor protection works.

i) in the vicinity of piers — $2.00\ d_{sm}$

ii) near abutments
- for approach retained — $1.27\ d_{sm}$
- for scour all round — $2.00\ d_{sm}$

b) Special studies should be undertaken for determining the maximum scour depth for the design of foundations in all situations where abnormal conditions such as the following are encountered :

i) in a bridge being located in a bend of the river involving a curvilinear flow, or excessive shoal formation;

ii) a bridge being located at a site where the deep channel in a river hugs to one side;

iii) a bridge having very thick piers inducing heavy local scours;

iv) where the obliquity of flow in the river is considerable;

v) where a bridge is required to be constructed across a canal, or across a river downstream of storage works, with the possibility of the relatively clear water inducing greater scours;

vi) a bridge in the vicinity of a dam, weir, barrage or other irrigation structures where concentration of flow, aggradation/degradation of bed, etc. are likely to affect the behaviour of the structure.

c) If a river is of flashy nature and the bed does not lend itself readily to the scouring effect of floods, the formula for d_{sm} given in Art. 4.9.2 shall not apply. In such cases, the maximum depth of scour shall be assessed from actual observations.

d) For bridges located across streams having bouldery beds, there is yet no rational formula for determining scour depth. However, the formula given in Art. 4.9.2 may be applied, with a judicious choice of values for 'D_b' and 'K_{sf}' and the results compared with the actual observations at site or from experiences on similar structures nearby and their performance. If a pucca floor at bed is provided, in such cases, it is essential to check the hydraulic performance of these structures under various flow conditions to ensure that a standing wave is not formed on the downstream side which may result in very heavy scours. It is also essential to check the usual scour that may take place downstream of a bed flooring and to make adequate provision for the same. If it is not possible to increase the waterway and to avoid the formation of a standing wave, a depressed pucca floor on the downstream may be provided to contain the standing wave within the floor.

CHAPTER 5

SUB-SURFACE INVESTIGATION

5.1 General

The sub-surface investigation is required at the initial stage of the project with the objective to determine the suitability of the sub-soil for the foundation of the bridges. This sub-surface investigation can be carried out in two stages, namely, preliminary and detailed.

5.2 Preliminary Investigation

Preliminary investigation shall include the study of existing geological information, previous site reports, geological maps, air photos, etc. and surface geological examination. In case of important bridge where no previous sub-surface data are available a few bore-holes may be taken. These will help to narrow down the number of sites under consideration and also to locate the most desirable site at which detailed sub-surface investigation like bore or drill holes, etc. are to be conducted.

5.3 Detailed Investigation

5.3.1 Scope

Based on data obtained after preliminary investigation of the bridge site, the type of structure with span arrangement and the location and type of foundation, the program of detailed investigation, etc. shall be decided. Thereafter the scope of detailed investigation including the extent of exploration, number of bore holes, type of tests, number of tests, etc. shall be decided in close liaison with the design engineer and the exploration team, so that adequate data considered necessary for detailed design and execution are obtained. For the bridges of this manual the number of bore-holes should be atleast one below each abutment foundation.

5.3.2 Extent of Boring

Generally the sub-surface investigation should extend to a depth below the anticipated foundation level equal to about one and a half times the width of the foundation; in case of pile foundation the width of pile group shall be considered as the width of foundation. However, where such investigation end in any unsuitable or questionable foundation material the exploration shall be extended to a sufficient depth into firm and stable soils.

a) Location Boring

Where the data made available by detailed exploration indicates appreciable variation or where such variations in a particular foundation, are likely to appreciably affect the construction, it will be necessary to resort to additional bore to establish a complete profile of the underlying strata. The requirement of additional boring will be decided depending upon the extent of variation at a particular foundation location and should cover the entire area of the particular foundation and be decided in consultation with the design engineer.

b) Construction Stage Exploration

Such exploration may become necessary when a change in the sub-soil strata is encountered during construction. In such situations it may be essential to resort to further exploration to establish the correct data, for further decision.

5.3.3 Sub-Surface Data Required

The scope of the detailed sub-surface exploration shall be fixed as mentioned in Art. 5.3.1 and 5.3.2. However, as a general guide it shall be comprehensive enough to enable the designer to estimate or determine the following :

i) the engineering properties of the soil/rock, if any;
ii) the location and extent of soft layers and gas pockets, if any under the foundation strata;
iii) the ground water level;
iv) artesian condition, if any;
v) quality of water in contact with the foundation;
vi) the depth and extent of scour;
vii) suitable depth of foundation i.e. pile length for the purpose of this Manual;
viii) the bearing capacity of the pile foundation for the structures of this Manual;
ix) probable settlement and differential settlement of the pile group;
x) likely sinking or driving effort; and
xi) likely construction difficulties.

5.3.4 Method of Taking Soil Samples

The size of the bores shall be pre-determined so that undisturbed samples as required for the various types of tests are obtained.

5.4 Exploration for Foundations

5.4.1 Type and Extent of Exploration

The type and extent of exploration shall be divided into the following groups keeping in view the different requirements of foundation design and the likely method of data collection:

i) sub-surface investigation requiring large depth of exploration; and
ii) Fills behind abutments and protective works.

For better interpretation, it will be desirable to correlate the laboratory results with the in-situ tests like standard penetration test (SPT) results.

The tests to be conducted at various locations for properties of soil, etc. are different for cohesive and cohesionless soils. These are enumerated below and shall be carried out wherever practicable according to the soil type :

a) Cohesionless Soils

i) Field test: SPT bore-log
ii) Laboratory Tests :

- Classification tests, density, etc.
- Shearing strength tests-triaxial or box shear test; in case of the possibility of rise of water table the tests shall be done on saturated samples.

b) Cohesive Soils

i) Field test : SPT bore-log
ii) Laboratory Tests :

- Classification tests, density, etc.
- Shearing strength tests-triaxial tests
- Consolidation test
- Where dewatering is expected, the samples may be tested for permeability
- Unconfined compression test

5.4.2 Large Depth of Exploration

In this group are covered cases of deep wells, pile foundations, etc. where the use of boring equipment, special techniques of sampling, in-situ testing, etc. become essential. In addition to the problems of soil and foundation interaction an important consideration can be the soil data required from construction considerations. Often in the case of coheisionless soils, undisturbed samples cannot be taken and recourse has to be made to in-situ field tests.

The sub-surface exploration can be divided into three zones :

i) between channel bed level and upto anticipated maximum scour depth (below HFL)
ii) from the maximum scour depth to the pile tip elevation
iii) from pile tip elevation to about 1.50 times the width of pile foundation below it.

Sampling and testing (in-situ and laboratory) requirement will vary in each case and hence are dealt with separately. The sub-soil water shall be tested for chemical properties to evaluate the hazard of deterioration to foundations. Where dewatering is expected to be required, permeability characteristics should be determined.

For the different zones the data required, method of sampling, testing, etc. are given in Table 5.1. Samples of soils in all cases shall be collected at about 1 to 1.50 meter interval or at change of strata.

5.4.3 Approach Fill Materials

Representative disturbed samples shall be collected from borrowpit areas. Laboratory tests shall be conducted for determining the following :

i) classification and particle size
ii) moisture content
iii) density vs. moisture content relationship
iv) shearing strength
v) permeability

Table 5.1 **Program of Sub-Soil Investigation**

Sl. No.	Zone	Data Required	Sampling and Testing	ASTM Test Method	Remarks
1.	Bed levels to anticipated scour level	(i) SPT bore-log (ii) Grain Size Distribution a) Sieve Method b) Hydrometer Method (iii) Shearing Strength Characteristics (iv) Soil Classification (v) Density : Bulk, saturated and dry (vi) void ratio	For (i) - (iv) SPT bore-log and disturbed samples may be collected. For (v) & (vi) undisturbed samples may be collected	(i) D1586 (iia) D421 (iib) D422 (iv) Cassagrande's Plasticity Chart (v) D2216	(iia) for mainly granular soil (iib) for fine-grained and cohesive soil (iii) may be assessed from SPT bore-log (iv) Unified Soil Classification System (UCS) (v) & (vi) for cohesive soil only
2.	Maximum anticipated scour level to pile tip level	(i) SPT bore-log (ii) Soil Classification (iii) Atterberg limits (iv) Grain Size Distribution (v) Natural Moisture Content (vi) Density: Bulk, saturated and dry (vii) Void ratio (viii) Unconfined compression test (ix) Compressibility by one dimensional consolidation test	SPT bore-log and disturbed samples for (i)-(iv) and undisturbed samples for (v) - (ix) may be collected	(i) D1586 (ii) Cassagrande's Plasticity Chart (iii) D4318 (iv) D421/D442 (v) D2216 (viii) D2166 (ix) D2466 Test 17	Tests under (vii) & (viii) will be conducted for cohesive soil only (ix) For highly compressible soil only
3.	Pile tip level to 1.5 times the width of the pile group below	(i) SPT bore-log (ii) Soil Classification (iii) Compressibility by one dimensional consolidation test (iv) Shear strength characteristics	SPT bore-log and disturbed samples for (i) & (ii) and undisturbed samples for (iii) may be collected	(i) D1586 (ii) Cassagrande's Plasticity Chart (iii) D2466 Test 17	(ii) by UCS (iii) for highly compressible soil only (iv) will be assessed from SPT bore-log

CHAPTER 6

DESIGN CODES, STANDARDS AND LOADS

6.1 Design Codes and Standards

Mainly American Association of State Highway and Transportation officials (AASHTO) Standard Specifications for Highway Bridges, 15th edition, 1992 herein after called AASHTO '92 has been followed for design purposes.

IRC : 5-1985, Standard Specifications and Code of Practice for Road Bridges, Section I, General Features of Design (6th revision, 1996) has been used for definitions of terms, hydraulic considerations and guidelines for sub-soil investigation.

American Society of Testing and Materials (ASTM) Standards, AASHTO Standard Specification for Transportation Materials and Methods of Sampling and Testing , Part I-Specifications and Part II-Tests or equivalent have been used for the material testing and specifications.

Bangladesh National Building Code (BNBC), 1993 provisions have been used for the wind and earthquake loading.

Further Codes and Standards used for references are:

BS 5400	:	Code of Practice for Steel, Concrete and Composite Bridges (10 Parts)
BS 8110	:	Structural Use of Concrete (2 Parts)
IRC 78	:	Standard Specifications and Code of Practice for Road Bridges Sec. VIII, Foundations and Substructure
IRC 83	:	Standard Specifications and Code of Practice for Road Bridges Sec. IX, Part II: Elastomeric Bearings

6.2 Loads

6.2.1 Dead Load

In estimating the dead loads, the unit weights of materials have been used as given in Table 6.1. But in some cases, unit weight of soil is taken as the average of unit weight of dry soil and bulk unit weight of submerged soil, i.e. in design calculations unit weight of soil is taken as 19 kN/m^3.

Table 6.1 Unit Weight of Materials

Material	Unit Weight (kN/m^3)
Steel	77.0
Plain concrete	22.0
Reinforced concrete	23.52
Prestressed concrete	24.5
Wearing course	23.0
Compacted sand or earth	18.0
Bulk unit weight of submerged soil	20.0
Saturated unit weight of soil	19.5
Loose sand or earth	16.0

6.2.2 Live Load

AASHTO HS20-44 truck loading and its equivalent lane loading have been considered as the design live load (Ref: Plate 6.1). For superstructure design the live load is increased by an appropriate impact fraction,

$I = 15.24/(S+38)$

where 'S' is the effective loaded span length in meter. Maximum value of 'I' is limited to 0.30 (i.e. =< 0.30).
(Ref: AASHTO '92, Div.-I, Art. 3.8.1)

6.2.3 Earth Pressure

The pressure on abutment-wing wall due to backfill is a function of the relative movement between the structure and the surrounding soil.

Active State

Active earth pressure occurs when the wall moves away from the soil and the soil mass stretches horizontally sufficient to mobilize its shear strength fully, and a condition of plastic equilibrium is reached. According to Terzaghi, this movement may either be translational or rotational. The ratio of the horizontal component of active pressure to the vertical stress caused by the weight of the soil at this stage is the active pressure coefficient (K_a). The active earth pressure coefficient as defined above applies to cohesionless soil only.

Passive State: Passive earth pressure occurs when soil mass is compressed horizontally, mobilizing its shear resistance fully. The ratio of the horizontal component of passive pressure to the vertical stress caused by the weight of the soil is the passive pressure coefficient (K_p). The passive coefficient as defined here, applies to cohesionless soil only.

At Rest State: A soil mass that is neither stretched nor compressed is said to be in at-rest state. The ratio of lateral stress to vertical stress at this state is called the at-rest coefficient (K_o).

The effect of wall movement to wall pressure in the magnitudes of K_a and K_p as given by NAVFAC DM-7.2, Foundations and Earth Structure Design Manual 7.2, Figure 1 is reproduced and given in Plate 6.2 .

The relationship between soil backfill type and wall rotation to mobilize active and passive earth pressures behind rigid retaining walls as given in AASHTO 1992, Table 5.5.2A is as follows:

Soil Type and Condition	Wall Rotation, Δ/H	
	Active	Passive
Dense Cohesionless	0.001	0.020
Loose Cohesionless	0.004	0.060
Stiff Cohesive	0.010	0.020
Soft Cohesive	0.020	0.040

6.2.4 Longitudinal Forces

6.2.4.1 Due to Live Load

The effect of a longitudinal force of 5 percent of the live load in all lanes carrying traffic headed in the same direction has been considered. The center of gravity of the longitudinal force has been assumed to be located 1.83 meter above the floor slab and to be transmitted to the substructure through the bridge bearings (Ref: AASHTO'92, Art. 3.9).

6.2.4.2 Due to Shrinkage and Creep

In-plane deformation occurs on the bridge superstructure due to shrinkage and creep. In the composite precast PC girder shear connected to cast-in-situ deck slab, the effect of creep-modified differential shrinkage is also to be considered in the appropriate load group of AASHTO '92. Longitudinal forces generated due to restraint of the horizontal deformation of the superstructure are transferred to the bridge substructure through the bearings. This is calculated in accordance with the provisions given in the AASHTO'92, Div.-I, Art. 14.4 . The design steps in a flow chart are given in Art. 12.4 , Part B of this Manual.

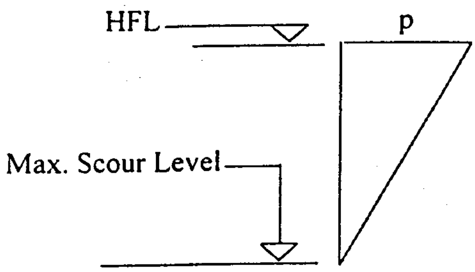
6.2.4.3 Due to Thermal Effect

In-plane deformation in the bridge superstructure will occur due to rise and fall of effective bridge temperature over the erection stage temperature arising out of the seasonal variation of the ambient temperature surrounding the bridge. This deformation is calculated to each support position considering zero movement point (ZMP) at the center of gravity of deck for each simply supported span separately. The co-efficient of thermal variation of concrete $\alpha = 12 \times 10^{-6}/^{0}C$ has been considered. The seasonal variation of temperature over the erection temperature has been considered (+/-) $30^{0}C$ for design purposes. The transfer of thermal load effect from the bridge superstructure to the substructure at the bearing has been considered.

6.2.4.4 Due to Stream Current Force on Bridge Support

The stream current force due to of flowing water on bridge support is calculated using the following formula :

$$p = 1.03\ K.(Vav)^2$$



where p = pressure due to stream current, kN/m^2
Vav = maximum mean velocity of water, m/sec
K = a constant being $1\,{}^3/_8$ for square ends and ${}^2/_3$ for circular ends of support obstructing the flow.

6.2.4.5 Due to Wind Load

The wind speed for a structure shall be read from the Basic Wind Speed Map given in the Bangladesh National Building Code '1993 (BNBC '93) Fig. 6.2.1 (ref : Plate 6.3). A basic wind speed of 240 km/hr has been considered for the design of the standard structures of this Manual.

6.2.4.6 Due to Earthquake Load

The seismic design of the standard structures of this Manual has been made based on the elaborate seismic design procedure given in the AASHTO '92. Vol. I-A, Seismic Design. Earthquake zones are to be considered in accordance with the BNBC '93, Fig. 6.2.8, Seismic Zoning Map of Bangladesh (ref: Plate 6.4). The corresponding seismic zone coefficients are given in Table 6.2 .

Table 6.2 Seismic Zone Coefficient (Ref: BNBC '93)

Seismic Zone	Zone Coefficient
1	0.075
2	0.15
3	0.25

The Zone 2 Coefficient has been used in calculating the seismic loading for designing the structures of this Manual.

6.3 Sidewalk, Curb and Railing Load

6.3.1 Sidewalk Loading

Sidewalk floor is to be designed for a live load of 4.00 kN/m^2 (85 lb/ft^2) of sidewalk area (ref : AASHTO '92, Div.-I, Art. 3.14.1). PC girders and other components of the standard structures of this Manual are to be designed for the following sidewalk live loads :

Spans 7.92 to 30 meters	4.00 kN/m^2 (85 lb/ft^2)
Spans over 30 meters	2.90 kN/m^2 (60 lb/ft^2)

6.3.2 Curb Loading

Curbs shall be designed to resist a lateral force of not less than 7.3 kN/m (500 lb/ft) of curb length, applied at the top of the curb (ref : AASHTO '92, Div.-I, Art. 3.14.2.1).

6.3.3 Railing loading

Rail bar and rail posts shall be designed as traffic railings. For design of rail bar a concentrated load, P = 45 kN (10 kip) will be applied at the center of the panel. This P shall be distributed equally for each rail bar. The pedestal for rail post support shall also be considered as a rail bar for the purposes of distributing P. For designing the rail post, the above loading i.e. P = 45 kN shall be applied longitudinally, divided equally and applied at the centroid of the junction between each rail bar and rail post including curb (ref : AASHTO '92, Div.-I, Fig. 2.7.4 B, Traffic Railing and Art. 2.7).

6.4 Combination of Loads

The combinations of loads and forces for analysing and designing the structure have been used in accordance with AASHTO '92, Div.-I, Table 3.22.1A (ref : Plate 11.1).

The service load method has been used for checking deflection and crack width of RC members, for preparing schedule of stresses for PC girders and in determining service load on piles.

The structural design of the RC members and piles have been made using the AASHTO load factor design (LFD) method. Also the ultimate moment of resistance and shear design of the PC girders have been made following the above LFD method.

PLATE 6.1

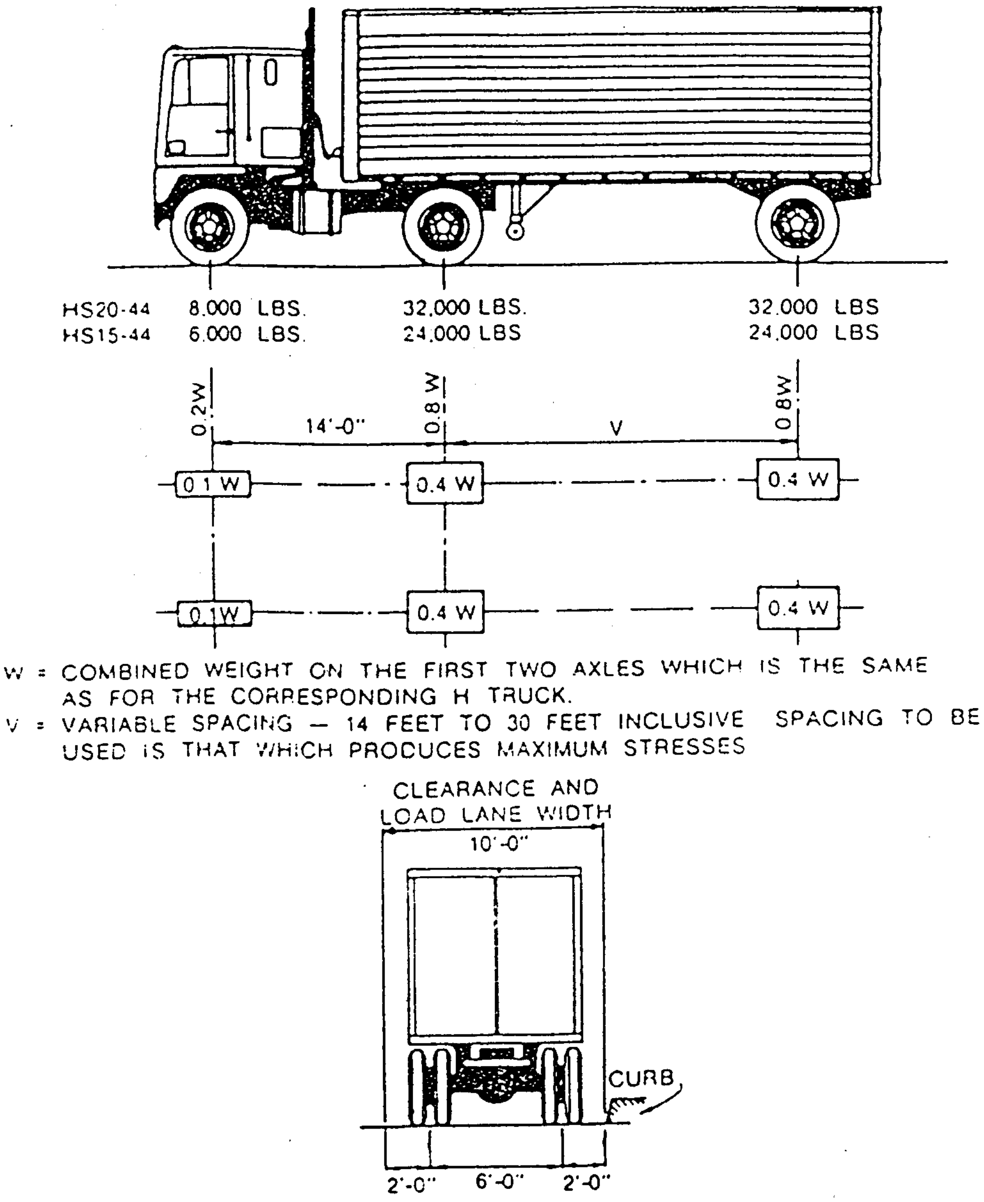
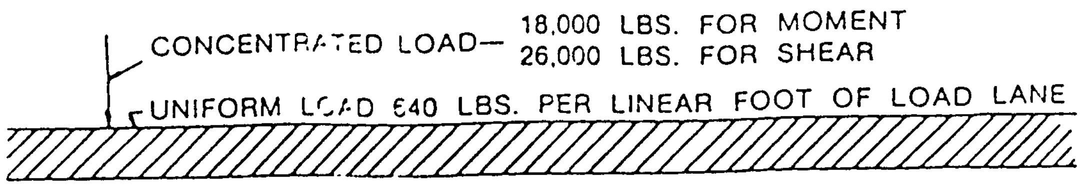
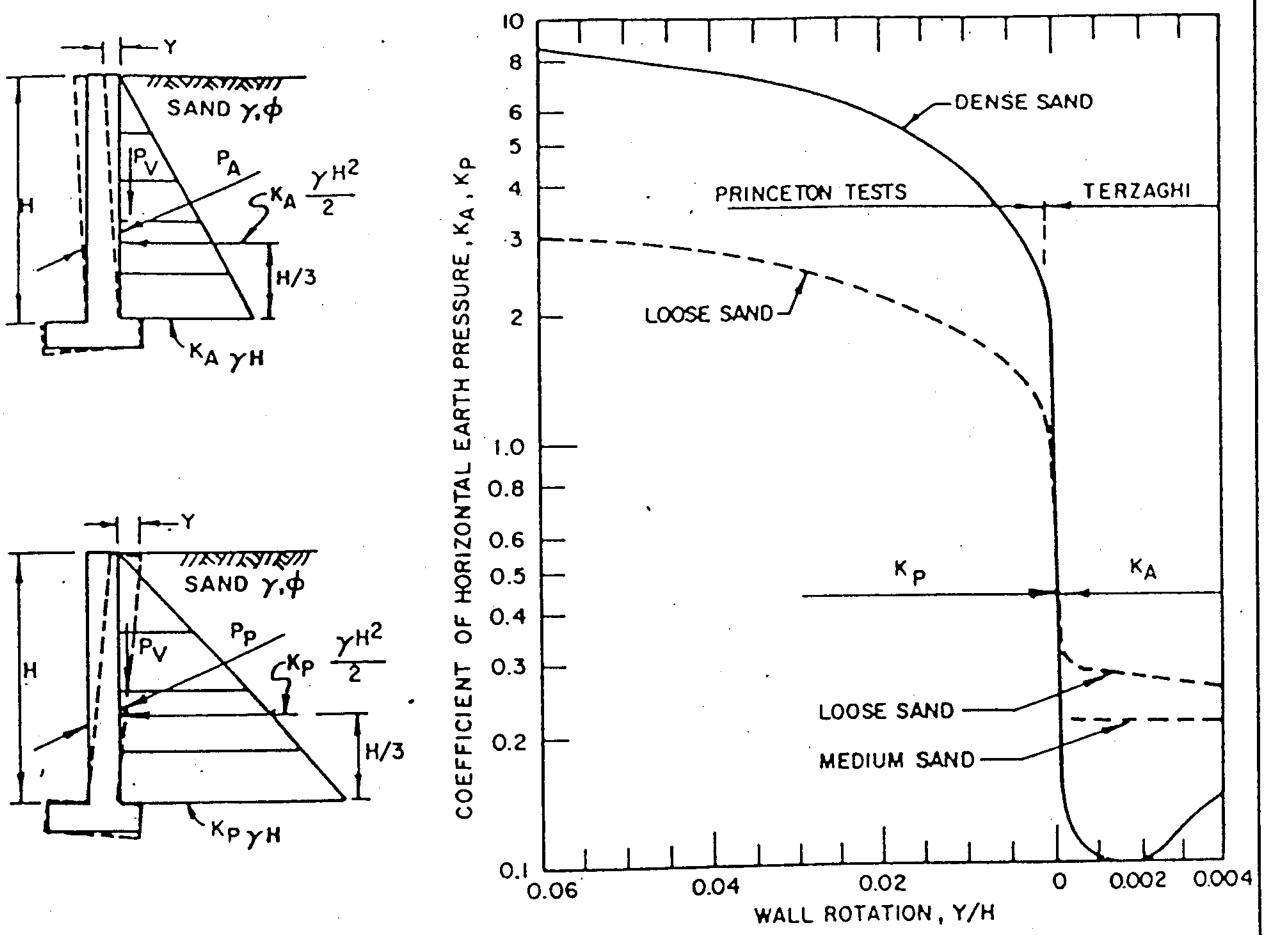


Figure Standard HS Trucks



H20-44 LOADING
HS20-44 LOADING

Figure Lane Loading



MAGNITUDES OF WALL ROTATION TO REACH FAILURE

SOIL TYPE AND CONDITION	ROTATION Y/H*	
	ACTIVE	PASSIVE
DENSE COHESIONLESS	.0005	.002
LOOSE COHESIONLESS	.002	.006
STIFF COHESIVE	.01	.02
SOFT COHESIVE	.02	.04

* Y = HORIZONTAL DISPLACEMENT
H = HEIGHT OF THE WALL

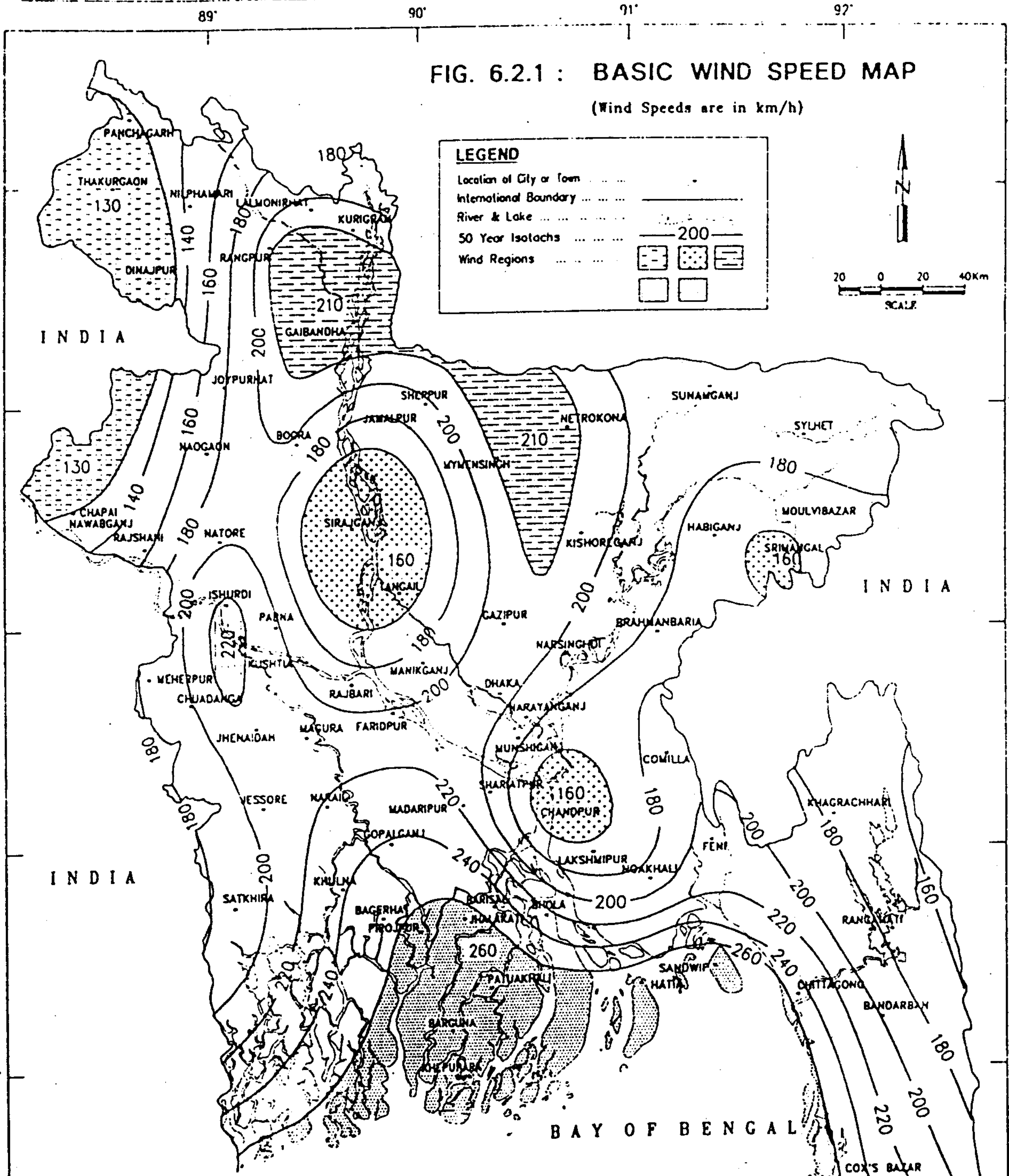
FIGURE 1
Effect of Wall Movement on Wall Pressures

Ref.: NAVFAC DM-7.2. Foundations and Earth Structures. May 1982

PLATE 6.3

FIG. 6.2.1 : BASIC WIND SPEED MAP

(Wind Speeds are in km/h)



NOTE : a) Isotach at a region boundary has the same value as that of the region

b) Basic wind speed for a particular location shall be obtained as follows :

i) When a location is listed in Table 6.2.8, value of the basic wind speed shall be taken from that table.

ii) When the location lies within any region (shown coloured in the map), the value marked for that region shall be taken.

iii) For a location lying on any isotach in this map, the value of that isotach shall be taken

iv) For a location lying outside the positions (i) through (iii) above, linear interpolation shall be made between the adjacent isotachs to obtain the basic wind speed.

PLATE 6.4

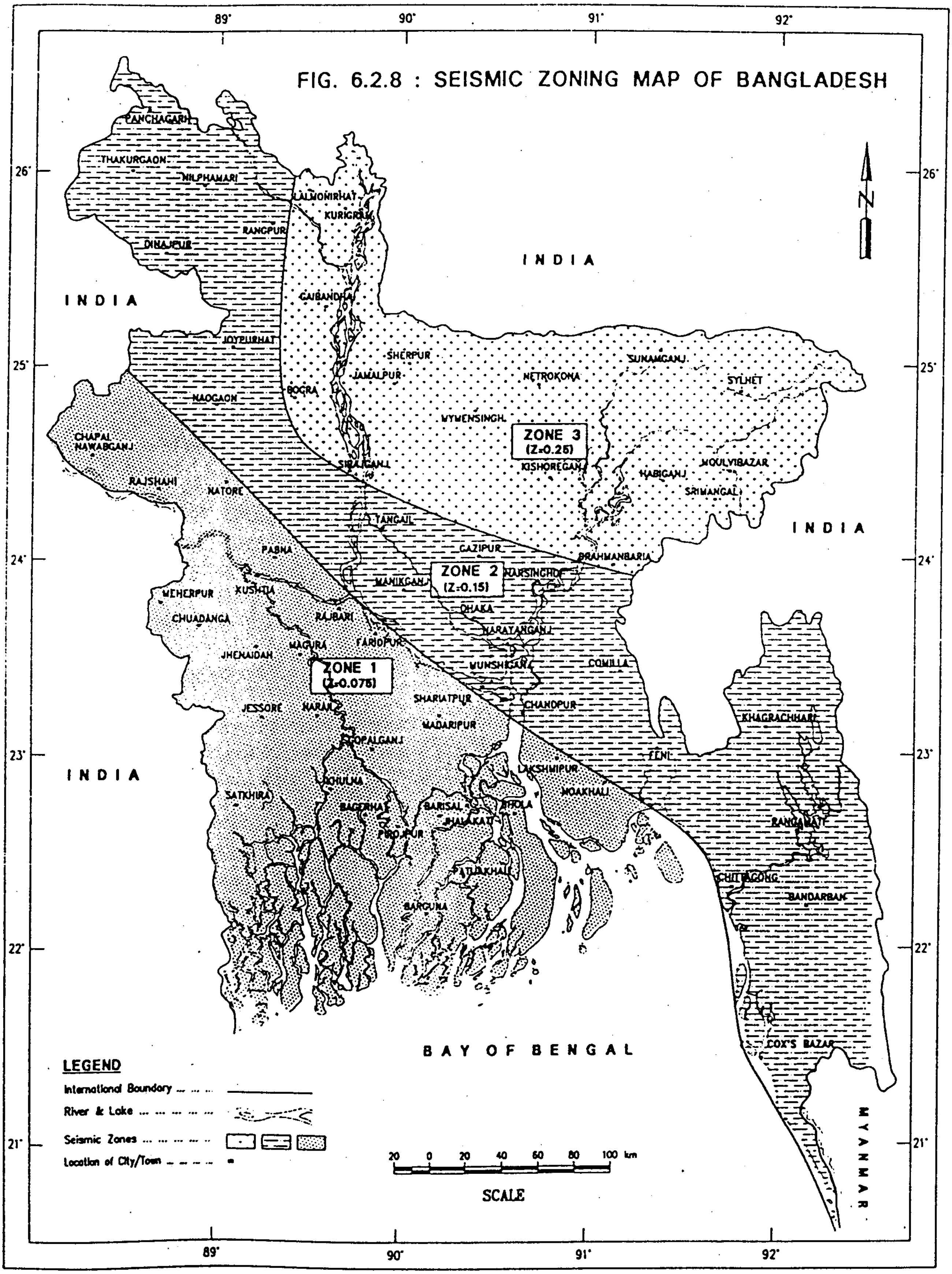


FIG. 6.2.8 : SEISMIC ZONING MAP OF BANGLADESH

CHAPTER 7

MATERIAL STRENGTH, PROPERTIES AND QUALITY CONTROL TESTS

7.1 General

For the material strength, properties and tests the relevant AASHTO, ASTM, BS or equivqlent Bangladesh Standards are mainly to be followed.

7.2 Cement

Portland cement conforming to the AASHTO Designation: M85-89 or ASTM Designation C150-86 or BS 12 with the latest amendments shall be used.
Five types of Portland cement are relevant for this Manual.

- Type I - When the special properties specified for any other type are not required. This Type-I cement is also called Ordinary Portland Cement (OPC)
- Type II - When moderate sulfate resistance or moderate heat of hydration is desired.
- Type III - When early strength is desired.
- Type IV - When low heat of hydration is desired.
- Type V - When high sulfate resistance is desired.

The following minimum tests of cement are recommended:

- Compressive strength of hydraulic cement mortars (using 50 mm cube specimens) — AASHTO T106 or equivalent
- Time of setting of hydraulic cement by Vicat Needle — AASHTO T131 or equivalent

The cement may be rejected if it fails to meet the above tests.

Cement remaining in bulk storage at the mill, prior to shipment for more than 6 months, or cement in bags in local storage for more than 3 months, after completion of tests may be retested before use and may be rejected if it fails to conform to any of the requirements of the test and other specifications.

7.3 Fine Aggregate

The fine aggregate for Portland cement concrete should conform to AASHTO Designation: M6-87 or BS 882: 1983 or equivalent standards.

This shall consist of natural or manufactured sand or combination thereof, having hard, strong durable particles to the requirement of the specification.

The following tests shall be performed:

- Grading by sieve analysis — AASHTO T27 or BS 812 : Sec. 103.1, 1985
- Fineness modulus — AASHTO M6-87

- Deleterious substances e.g. clay lumps and friable particles — AASHTO T112
- Organic impurities — AASHTO T12

Fine aggregate for use in concrete, that will be subjected to wetting, extended to humid atmosphere, or contacted with moist ground, shall not contain any materials that are deleteriously reactive with the alkalies in the cement in an amount sufficient to cause excessive expansion of mortar or concrete. If such materials are present in injurious amounts, the fine aggregate may be used with a cement containing less than 0.6 per cent alkalis or with addition of material that has been shown to prevent harmful expansion due to the alkali-silicate reaction.

7.4 Coarse Aggregate

The coarse aggregate for Portland cement concrete should conform to AASHTO Designation: M80-87 or BS 882: 1983 or equivalent standards.

This shall consist of gravel, crushed gravel, crushed stone or broken picked jhama bricks or a combination thereof to the requirement of the specification.

The following tests shall be performed:

- Grading (Sieve analysis) — AASHTO T27 & M43 or BS 812 : Sec. 103.1, 1985
- Deleterious substances e.g. clay lumps and friable particles in aggregate — AASHTO T112
- Resistance to abrasions by use of the Los Angeles machine — AASHTO T96
- Soundness of aggregate — AASHTO T104
- Water Absorption Tests for Bricks

The coarse aggregate for use in concrete that will be subjected to wetting, extended exposure to humid atmosphere or contacted with moist ground shall not contain any material that are deleteriously reactive with the alkalies in the cement in an amount sufficient to cause excessive expansion of mortar or concrete. If such materials are present in injurious amounts, the coarse aggregate may be used with a cement containing less than 0.6 per cent alkalies or with the addition of a material that has been shown to prevent harmful expansion due to the alkali-aggregate reaction.

7.5 Reinforcing Steel

This shall be deformed or plain billet-steel bar as shown on the drawings and conforming to AASHTO Designation: M31-89 or ASTM Designation: A615 M-87 with the latest amendments.

The grade 40 steel with minimum tensile strength 482 N/mm^2 and minimum yield strength 276 N/mm^2 has been used in the design.

The following tests shall be performed:

- Tensile and yield strength and elongation — AASHTO T244 or equivalent
- Bending requirement — AASHTO T285 or equivalent

7.6 Prestressing Steel

The prestressing steel shall be used uncoated stress-relieved wire for prestressed concrete conforming to AASHTO Designation: M204-89 (ASTM Designation: A421-80 (1985)). Type WA wire shall be used for application in which the ends are anchored by wedges, and no cold-end deformation of the wire is involved.

The wire shall be cold-drawn to size and suitably stress relieved after cold drawing by a continuous heat treatment to produce the prescribed mechanical properties.

The chemical composition of the steel shall be at the discretion of the manufacturer but phosphorus and sulphur values shall not exceed the following:

- Phosphorus 0.040%
- Sulphur 0.050%

12ϕ7 cables have been used in which nominal diameter of each wire is 0.276 inch (7.01 mm), the minimum tensile strength for type WA wire is 235,000 psi (1620 N/mm^2) and yield strength i.e. minimum stress at 1% extension is 199,750 psi (1377 N/mm^2).

The tests for tensile and yield strength and elongation shall be made in accordance with AASHTO Designation T244. The total elongation under load of all wire shall not be less than 4.0% when measured in a gage length of 250 mm.

7.7 Concrete

Concrete is basically a mixture of two parts: aggregates and paste. The paste, consisting of Portland cement and water, binds the aggregates (sand and gravel or crushed stone) into a rock like mass as the paste hardens due to chemical reaction of the cement and water.

The quality of the concrete depends to a greater extent upon the quality of the paste. For given materials and condition of curing, the quality of the hardened concrete is determined by the amount of the water used in relation to the amount of cement. Following are some advantages of reducing water content:

- Increased compressive and flexural strength
- Increased watertightness
- Lower absorption
- Increased resistance to weathering
- Better bond between successive layer
- Better bond between concrete and reinforcement
- Less volume change from wetting and drying

The less water is used, the better will be the quality of the concrete, provided it can be compacted properly. Vibration or consolidation permits improvement in the quality of concrete and in economy.

Freshly mixed concrete should be plastic or semifluid, capable of being moulded by hand. A plastic mix keeps all grains of sand and pieces of gravel or stone encased and held in place without segregation. A plastic and cohesive paste is attained with a low water - cement ratio and the correct proportions of all ingredients.

The following 28 days cylinder crushing strengths of concrete, f'_c in accordance with ACI-83 (revised '92), have been used in the design:

- PC girder = 30 N/mm^2
- Deck slab and others = 20 N/mm^2

7.8 Mixing Water for Concrete

Almost any natural water that is drinkable and has no pronounced taste or odour can be used as mixing water for making concrete.

Water of questionable suitability can be used for making concrete if mortar cubes made with it have 7 days and 28 days strengths equal to atleast 90% of that specimens made with drinkable water. Mortar cubes should be made and tested according to ASTM Designation: C109-Standard Method of test for compressive strength of Hydraulic cement mortars. In addition, ASTM Designation: C191-Vicat Needle tests should be made to ensure that impurities in the mixing water do not adversely shorten or extend the setting time of the cement.

Water containing less than 2000 parts per million (ppm) of total dissolved solids can generally be used satisfactorily for making concrete.
The chemical limit of mixing water is given in Table 7.1.

Table 7.1* Chemical Limits for Mixing Water

Chemicals	Maximum Concentration (ppm)	Test Method
Chloride as 'Cl'		ASTM D512
• Prestressed Concrete or Concrete in bridge decks	500	
• Other reinforced concrete in moist environments	1000	
Sulfate as 'SO_4'	3000	ASTM D516
Alkalies as (Na_2O+0.658K_2O)	600	
Total solids	50000	AASHTO T26

* Adapted from PCA, Design and Control of Concrete Mixtures, 12th ed., 1979.

7.9 Admixtures for Concrete

Admixture are those ingredients in concrete other than Portland Cement, water and aggregates that are added to the mixture immediately before or during mixing.

Prior approval will be needed for the use of admixtures in concrete.

Admixtures, if used in concrete for water reducing, retarding, accelerating, water reducing but retarding and water reducing but accelerating, shall conform to ASTM C494 : Specification for chemical admixtures of concrete or ASTM C1017 : Specification for chemical admixtures for use in producing Flowing Concrete.

The classification of admixtures used in the work is given in Table 7.2.

Table 7.2* Admixtures by Classification

Desired Effect	Type of Admixture	Material
Reduce water required for given consistency	Water reducer (ASTM C494. Type A)	Lignosulfonates Hydroxylated carboxylic acids. (Also tend to retard set so accelerator is added)
Retard setting time	Retarder (ASTM C494, Type B)	Lignin Borax Sugars Tartaric acid and solts
Accelerate setting and early-strength development	Accelerator (ASTM C494, Type C)	Calcium chloride (ASTM D98) Triethanolamine
Reduce water and retard setting	Water reducer and retarder (ASTM C494, Type D)	(See water reducer, Type A,above)
Reduce water and accelerate setting	Water reducer and accelerator (ASTM C494, Type E)	(See water reducer, Type A,above. More accelerator is added)
Improve workability and plasticity	Pozzolan (ASTM C618)	Natural pozzolans(Class N) Fly ash (Class F and C) Other materials (Class S)

* Adapted from PCA, Design and Control of Concrete Mixture, 12th ed., 1979

CHAPTER 8

DESIGN OF PRESTRESSED CONCRETE (PC) GIRDER

8.1 General

8.1.1 Design Philosophy

AASHTO'92 requires that the PC girders are to be designed to satisfy the following two conditions:

i) Members shall be designed as reinforced concrete based on the strength (Load Factor Design, LFD) method. That means, the PC members shall be proportioned for adequate strength using the provisions of the AASHTO'92 LFD method of design.

ii) Behaviour of the PC members in service condition shall be determined by elastic analysis, taking into account the reactions, moments, shear and axial forces produced by prestressing, the effect of temperature, creep, shrinkage, axial deformation, restraint of attached structural elements and foundation settlement.

Accordingly the ultimate moment of resistance and shear are to be checked under LFD method and schedule of stresses, deflection and crack width are to be checked under service load criteria of AASHTO.

The PC members may be assumed to act as uncracked members subjected to combined axial, shear and bending stresses under service loads. In calculations of section properties, the transformed area of bonded reinforcement may be included for post-tensioned members after grouting. For the PC girders of this Manual, gross area of the section instead of transformed area has been used after grouting of the cable ducts, and is on the conservative side. For unbonded cables areas of the open ducts shall be deducted.

8.1.2 Span Length and Overall Length

The effective span length of simply supported girders shall be center to center bearings but shall not exceed the clear span plus the depth of the girder. Extra length is to be provided beyond center line bearing at each end. Thus overall length of the girder is center to center bearing span plus twice the end projection.

8.1.3 Effective Compression Flange Width for Composite T-Girder

For composite prestressed concrete construction where slabs or flanges are assumed to act integrally with the girder, the effective flange width shall conform to the following:

a) 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.

b) For girder having a slab on one side only, the effective overhanging flange width shall not exceed 1/12 of the span length of the girder, six times the thickness of the slab, or one-half of the clear distance to the next web.

c) If the concrete strength of the PC girder and the deck slab are different, then the above flange width will be multiplied by a correction factor = $Ec_{(deck)} / Ec_{(girder)}$ to get the effective flange width.

8.1.4 Deflection

Deflection calculations shall consider dead load, live load, prestressing, erection loads, concrete creep, shrinkage and steel relaxation.

8.1.5 Composite Flexural Member

Composite flexural members consist of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to superimposed loads as a unit.

The design shall provide for full transfer of horizontal shear forces at contact surface of interconnected elements.

In structures with a cast-in-place slab on precast girders, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of the girder. Because the tensile shrinkage develops over an extended time period, the effect on the girder is reduced by creep. Differential shrinkage may influence the cracking load and the girder deflection profile. When these factors are particularly significant, the effect of differential shrinkage should be added to the effect of loads.

8.2 Allowable Stresses

The characteristic strengths of concrete, reinforcing and prestressing steel used in the design of the PC girder, deck slab and other structural components are given in Chapter 6.0 .
The allowable stresses for the different components of the PC girder in accordance with AASHTO '92, Div.-I, Art. 9.15 are given below:

a) Prestressing Steel Considering Post-Tensioned Members

All units are in N/mm^2

Stress at anchorage after seating	= 0.70 f's
Maximum limit of overstressing for short period to offset seating and friction losses	= 0.9 fy*
The maximum stress at the end of the seating loss zone	= 0.83 fy*
Stress at service load (in accordance with AASHTO '92, Table 3.22.1A without overload provisions) after losses	= 0.80 fy*

b) Concrete

i) Temporary stresses before losses due to creep and shrinkage for post-tensioned members.

- Compression = $0.55\ f'_{ci}$

  Tensile stress in precompressed tensile zone: No temporary stresses are specified by AASHTO '92

- Tensile stress in other areas

  - In tension areas with no bonded reinforcement = $1.38\ N/mm^2$ or $0.249\sqrt{f'_{ci}}$ where, f'_{ci} is the concrete strength at the time of transfer of prestressing.

  - In tension areas where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The max. tensile stress with bonded reinforcement = $0.623\sqrt{f'_{ci}}$

ii) Stresses at service load after all losses have occurred

- Compression = $0.40\ f'_c$
- Tension in the precompressed tensile zone
  - For members with bonded reinforcement
    - For mild to moderate exposure condition = $0.498\sqrt{f'_c}$
    - For severe corrosive exposure conditions, such as coastal areas = $0.249\sqrt{f'_c}$
  - For members without bonded reinforcement = 0

iii) Modulus of rupture for normal weight concrete = $0.623\sqrt{f'_c}$

iv) Anchorage Bearing Stress at service condition = $20\ N/mm^2$ (but not exceeding $0.9f'_{ci}$)

8.3 Loss of Prestressing Force

8.3.1 Types of Losses

There are several factors which cause the force in the prestressing cables to fall from the initial force imparted by the jacking system. Some of these losses are immediate, affecting the prestressing force as soon as it is transferred to the concrete member. Other losses occur gradually with time. These immediate and long term losses for post-tensioned members are given in Table 8.1 .

Table 8.1 Prestress Losses

Immediate	Long-term
Elastic Shortening Friction Anchorage/Wedge Pull-in	Concrete Shrinkage Concrete Creep Steel Relaxation

8.3.2 Immediate Losses

(a) Elastic Shortening, ES

For Post-tensioned members

$$ES = 0.5 \frac{E_S}{E_{ci}} f_{cir}$$ [Ref. Art. 9.16.2.1.2, AASHTO'92]

where,

E_S = Modulus of elasticity of prestressing steel, which can be assumed to be 205×10^6 kN/m^2

E_{ci} = Modulus of elasticity of concrete in kN/m^2 at transfer of stress, which can be calculated from:

$$E_{ci} = 13.53\, w^{3/2} \sqrt{f'_{ci}}$$

in which, w is the unit weight of concrete in kN/m^3 and f'_{ci} is in kN/m^2

f_{cir} = Concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer. f_{cir} shall be computed at the section / sections of maximum moment. (At this stage, the initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon friction for post-tensioned members.

(b) Friction Losses

Friction losses in post-tensioned steel is calculated as follows:

$$T_1 = T_x\, e^{(KL+\mu\alpha)}$$

where, T_1 = Steel stress at jacking end.
T_x = Steel stress at any point x.
K = Friction wobble co-efficient per meter.
L = Length of prestressing steel from jacking end to the point x.
μ = Friction curvature coefficient.
α = Total angular change of prestressing steel profile in radians from jacking end to the point x .

(c) Wedge Pull-in Losses

At load transfer when the tensioning circuit of the jack is depressurised, two actions take place simultaneously:

- a slight swelling of the female cone
- a slight reduction in the diameter of the male cone

As a result there is a partial loss of elongation at the anchorage and a drop of prestress force in the cable end which is generally advantageous to the structure as follows :

- local stresses are reduced
- there is reduced risk of cracking near anchorages
- reduction of initial steel stress temporarily used to overcome cable friction but generally of no use at the end of the beam in its final condition.

Loss of prestress force due to friction per unit length

$p = T_1 (1\text{-}\exp(-(\mu/r+K))$

where, r = Radius of curvature of cable
$r = L^2/8dr$ in which 'L' is the total length of girder and 'dr' is cable sag at mid span

Loss of prestress due to wedge pull-in, $\Delta f_{wp} = (\Delta_{wp} / l_{wp})\, E_{ps}\, A_{ps}$

where,

Δ_{wp} = Length of wedge pull-in, usually 6mm for 12ϕ7 HT wire anchorage system.
E_{ps} = Modulus of elasticity of HT wires
A_{ps} = Area of HT wires/cable

Length of cable subjected to prestress loss due to wedge pull-in and consequent reverse friction, $l_{wp} = (\Delta_{wp}\, E_{ps}\, A_{ps} / p)^{1/2}$

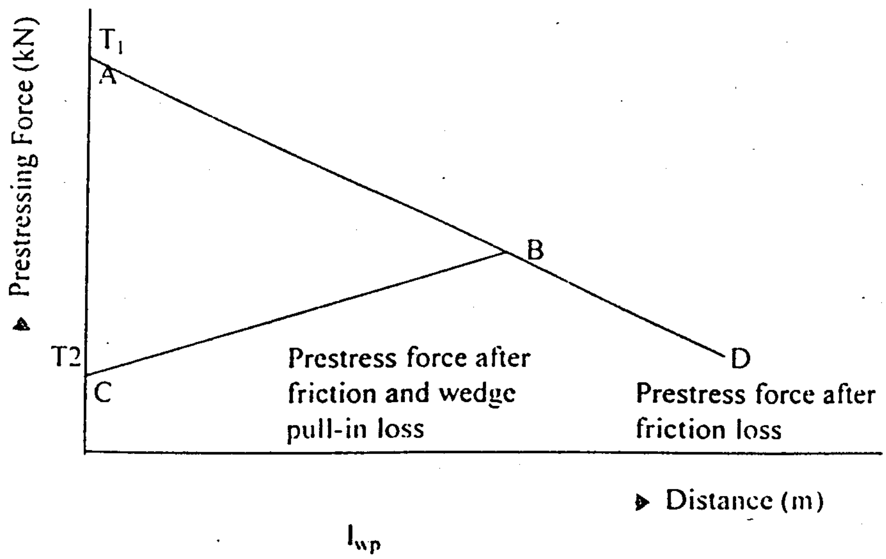


Fig. 8.1 Loss of Prestress Due to Friction and Wedge Pull-in

8.3.3 Long Term Losses

(a) Shrinkage Loss, SH

Loss of prestress due to shrinkage in post-tensioned members is calculated by using the following equation (Ref. Art. 9.16.2.1.1, AASHTO'92):

$$SH = 0.8(1700 - 150RH)\ \text{psi}$$
$$\text{or, } SH = 0.8(1700 - 150RH)/145\ \text{N/mm}^2$$

where,
SH = Shrinkage loss
RH = Mean annual ambient relative humidity in per cent.

(b) Creep Loss, CR_c

Creep Loss in post-tensioned members are calculated as follows (Ref. AASHTO'92, Div. I, Art. 9.16.2.1.3) :

$CR_c = 12f_{cir} - 7f_{cds}$ where,

CR_c = Creep loss in N/mm^2
f_{cds} = Concrete stress at the c.g. of the prestressing steel due to all dead load except the dead load present at the time of prestressing force is applied in N/mm^2
f_{cir} = As in Art. 8.3.2 (a) above in N/mm^2

(c) Relaxation Loss

Relaxation loss is a time dependent reduction of stress in prestressing steel at constant strain. This loss depends on the type of prestressing steel. For stress-relieved wires, normally used in this country, relaxation loss is 3.5% at 100hrs and a total of 5% at 1000hrs.

8.4 Cable Elongation

In post-tensioned members cable elongation is calculated by using the following formula:

$$\Delta = \frac{P_{av}\ L}{A_{ps}\ E_{ps}}(1 + 0.01ES)$$

where,
Δ = Cable elongation in mm.
P_{av} = Average prestressing force(N) between jacking end and passive end of a cable for stressing from one end and between jacking end and mid span for a cable stressing from both ends.
L = Length of cable(mm) for which average prestressing force is calculated. An additional length known as jacking length is to be added.
A_{ps} = Area of prestressing cable in mm^2.
E_{ps} = Modulus of elasticity of prestressing steel in N/mm^2.
ES = Elastic shortening loss in per cent.

8.5 Flow Chart for Design of PC Girder

a) Flow Chart 1: Preliminary works for design of PC Girder

Start

↓

Select - Span
- Deck geometry e.g., carriageway, footpath, curb, railing, rail post, cross slope, wearing course (WC), etc.

↓

Select material strength and properties for deck and cross-girder
- Concrete f_c'
- Reinforcing steel f_y

↓

Select
- Type of PC Girder I-section
  T-section
- No. of main and cross-girder, spacing and x-sec.

↓

Assume deck slab thickness
Design deck slab by using Pucher's chart/Yield line theory/ AASHTO simplified method
(Ref: Art 9.4)

↓

Flow Chart 2
Steps for design of PC Girder

b) Flow Chart 2: Steps for Design of PC Girder

PC Girder

↓

Select girder X-sec. for particular span from
- AASHTO Standard girder X-sec.
- PCI Standard girder X-sec.
- ROIP, FRIP Standard girder X-sec.

↓

Select material properties
- Concrete strength f_c' for girder concrete
- Prestressing steel conforming to ASTM A421-80 (85) HT wire
  - Diameter, As*
  - Tensile strength, f_s'
  - Yield strength, f_y^*
  - Type of cable 12φ7

Select method of prestressing
- Post-tensioned

↓

Select Design Code
AASHTO '92

↓

Select
- no. of cables
- cable layout at mid-span and end span
- cable profile and ordinates at mid-span and girder end. (At girder-end, cable eccentricity should be nearer to zero).
- sheath diameter for post-tensioning
- cable eccentricity
- clear cover

↓

Calculate section properties
Noncomposite girder X-sec.
- Gross area, $A_{noncomp}$
- Net area deducting ungrouted duct holes for post-tensioned concrete
- Depth from N.A. to girder top and bottom Y_t, Y_b
- Second moment of area, I_{NA}
- Section modulus Z_b, Z_t

↓

1

1

Calculate section properties
Composite section
- Effective flange width , b_f (Art. 8.1.3)
- Gross area, A_{comp}
- Net area deducting ungrouted duct holes for post-tensioned concrete, if any
- Depth from N.A. to girder, deck top & girder bottom Y_{tc}, Y_{ts}, Y_{bc}
- Second moment of area, I_{NAcomp}
- Section modulus Z_{tc}, Z_{ts}, Z_{bc}

↓

At mid-span calculate B.M. due to self weight of girder
Calculate stresses f_{b1} . f_{t1} $f_{b1} = M/Z_b$; $f_{t1} = M/Z_t$

↓

Assume stage 1 prestressing force,NP and eccentricity.e
Calculate $fb_2 = NP/A_{noncomp} + NPe/Z_b$
$ft_2 = NP/A_{noncomp} - NPe/Z_t$

↓

Calculate loss of prestress fb_3 & ft_3 due to friction, wedge pull-in, elastic shortening, relaxation loss (100 hrs.) at mid-span

↓

Calculate sub-total $f_{b4} = f_{b1} + f_{b2} + f_{b3}$ and $ft_4 = f_{t1} - f_{t2} + f_{t3}$
Check adequacy $fb \leq 0.55\ f_{ci}'$ and $f_t \leq 0.249\ \sqrt{f_{ci}'}$

↓

Shift the girder and place in position.
Stage 2 prestressing may then be done before concreting of cross-girder and deck slab depending upon the concrete strength of PC girder gained.

↓

Calculate f_{b5} & f_{t5} due to 2nd stage prestressing for both axial comp. and eccentricity, e

↓

Calculate loss of prestress f_{b6} & f_{t6} due to friction, wedge pull-in, elastic shortening and relaxation loss (100/1000 hrs.)

↓

Calculate sub-total 2 of stresses and check adequacy up to this stage $f_{b7} = fb4 + f_{b5} + f_{b6}$ and $f_{t7} = f_{t4} + f_{t5} + f_{t6}$

↓

Calculate B.M. due to X-girder at mid-span
Calculate f_{b8}, f_{t8}

↓

2

2

Calculate B.M. due to deck concrete at mid-span
Calculate f_{b9}, f_{t9} & f_{ts1}

Calculate B.M. due to WC, curb & railing at mid-span
Calculate f_{bc1} , f_{tc1} & f_{ts2}

For AASHTO HS20-44 loading,
calculate load intensity on the interior and exterior girders by simplified AASHTO method (AASHTO'92,Div.I,Art. 3.23.2) or load distribution analysis at mid-span

Calculate B.M. due to live load at mid-span
Calculate f_{bc2} , f_{tc2} & f_{ts3}

Calculate axial force and B.M. due to creep-modified differential shrinkage due to cast-in-situ deck concrete at mid-span
Calculate stresses f_{bc3} ,f_{tc3} & f_{ts4} (AASHTO'92,Div.I,Art. 9.13.3)

Calculate remaining losses f_{bc4} , f_{tc4} , f_{ts5} due to prestressing after stage 2 at mid-span
- Shrinkage
- Creep
- Final relaxation

Calculate sum of the above stresses at mid-span and check adequacy

$$f_{b10} = f_{b7}+f_{b8}+f_{b9}+f_{bc1}+f_{bc3}+f_{bc4}$$
$$f_{t10} = f_{t7}+f_{t8}+f_{t9}+f_{tc1}+f_{bc2}+f_{tc3}+f_{tc4}$$
$$f_{ts} = f_{ts1}+f_{ts2}+f_{ts3}+f_{ts4}+f_{ts5}$$

Determine cable ordinates at 3/8th, 1/4th and 1/8th span.
Check Schedule of Stresses at the above locations following the steps given above for mid span.

Provide end block size (AASHTO '92, Div. I, Art. 9.21)

3

3

Calculate external web shear force and check adequacy of web shear strength at the junction between normal section and transition to end block (AASHTO'92, Div.I, Art. 9.20)

↓

Check adequacy of flexural shear strength due to combined effect of shear and moment (AASHTO '92, Div.I, Art.9.17)

↓

Calculate ultimate moment of resistance at mid-span and other locations and check adequacy of ultimate moment resistance (AASHTO '92, Div.I. Art. 9.20)

↓

Design end block for
- Shear
- Bursting tensile force and edge spalling tensile force and provide secondary reinf.

↓

Design shear connectors (AASHTO '92, Div. I, Art. 9.21)

↓

Calculate girder deflection under service condition and provide precamber, if needed considering 2-3 yrs. situation after construction (AASHTO '92, Div. I, Art. 9.11)

↓

Check ductility (AASHTO '92, Div. I, Art. 9.18)

↓

Check minimum reinforcement (AASHTO '92, Div. I, Art. 9.18.2)

↓

Design temporary work for girder
- formwork
- scaffolding
- erection mechanism

CHAPTER 9

DESIGN OF RC DECK, CROSS-GIRDER, RAILING, SIDEWALK AND CURB

9.1 RC Deck

9.1.1 Design Philosophy

The structural design of bridge deck depends upon the spacing of the main and cross-girders and the cantilever overhangs on either end of the deck.

The standard method of design of deck slab is either of the following:

- AASHTO simplified method
- Yield line analysis
- Hillerborg strip method
- Pucher's chart

Since the main and cross-girder have been placed according to the specifications of the AASHTO'92 Standards, the analysis and design of the deck slab have also been made based on the same AASHTO Standards.

9.1.2 Allowable Stresses of Materials

Concrete : 28 days cylinder strength in accordance with ACI-83 (Revised '92) $f_c^{'} = 20\ N/mm^2$

Steel : ASTM A615M-88 Grade 40 deformed and plain bars $f_y = 276\ N/mm^2$

9.2 RC Cross-Girder

9.2.1 End Cross-Girder

It provides lateral stiffness to the deck at the girder end. In addition to the stresses produced by the typical design loads for X-girders, the end X-girders are to be checked against stresses resulting from reaction of flat jack placed on either end of the main girder to lift the latter in order to replace the elastomeric bearings if required.

9.2.2 Intermediate Cross-Girder

It is provided to restrain the longitudinal main girder against buckling. It is designed to carry dead loads and the critical load reaction arising out of the truck or lane loading with concentrated load for shear or moment.

9.3 RC Railing, Sidewalk and Curb

9.3.1 Railing

Traffic railings have been provided at the edges of structure for protection against vehicular traffic. These railings have been designed for load specified by AASHTO. 1992. Div. I, Art. 2.7.1 and Fig. 2.7.4.

Total height of the railing has been fixed as follows:

- For Type IA and IIA deck 0.80m over 0.20m plinth from sidewalk.
- For Type IB and IIB deck, 0.8m over 0.25m kerb over deck concrete.

- **Rail Post**

For design of rail post a transverse load of P = 45.00 kN has been considered acting on each rail post distributed equally at centre of gravity of each rail bar and top of parapet. The load P has been increased by the factor C = 1+(h-825)/450 when the height of top of the traffic rail exceeds 0.85 m.

Ultimate moment capacity of the rail post has been checked at the base level of post.

- **Rail Bar**

A design transverse load of P/3 or C.P/3 has been considered acting at mid point of rail bar spanning between two rail posts. Ultimate capacity of the rail bar has been checked against maximum factored moments. The design has also been checked for the uniformly distributed pedestrian live load @ 0.72 kN/m length of rail.

9.3.2 Sidewalk

Sidewalk has been designed for a live load of 5.00 kN/m^2 of the sidewalk area.

9.3.3 Curb

The curb has been designed to resist a lateral force of not less than 7.00 kN/m length of the curb and applied at the top of it.

9.4 Flow Chart for Design of Deck Slab

Start

▼

Select Deck Type i.e. IA, IB, IIA or IIB
(Deck comprises 2-main girder and 3-cross girder system)

▼

Assume thickness of Deck Slab 'h' from consideration of deflection.
For T-Girder
- Simply-supported span $h = 0.07S$, mm
- Continuous span $h = 0.065S$, mm

S = effective span, m

▼

Select
- for concrete, $f'c = 20\ N/mm^2$
- for reinforcing steel, $fy = 276\ N/mm^2$
- clear cover to reinforcement as shown in general requirements for RC construction (Dwg. no. PC-GN-00).

▼

Calculate under service dead load
- Max. -ve B.M. over girder due to cantilever span MDLES
- Max. +ve B.M. at span, $MDLIS = W.S^2/8$

▼

Calculate for AASHTO H20-44 Live Load under service condition
- Max. +ve B.M. at span due to internal span loading considering simple supports MLLIS
  $MLLIS = \{(S+0.61)/9.74\}\ P18$ kN-m
- Max. -ve B.M. over girder due to cantilever span loading MLLES

▼

Calculate impact fraction, $I = 15.24/(L+38)$ subject to a max. value of 0.30

▼

Calculate LL B.M. with Impact under service condition
- Max. +ve $ML+I = (1+I)\ MLLIS$
- Max. -ve $ML+I = (1+I)\ MLLES$

▼

1

1

Calculate factored +ve Design B.M. (+ MDES = MPOS)
MPOS = 1.3 [1.0 x MDLIS + 1.0 x MLLIS(1 + I)]
MDES = MPOS

Calculate factored -ve Design B.M. (- MDES = MNEG)
MNEG = 1.3 [1.0 x MDLES + 1.0 x MLLES(1 + I)]
MDES = MNEG

Assume reinforcement dia., spacing and area.

Calculate lever arm factor, a = As fy/0.85 fc'b

Calculate moment capacity
MULT = Ø Mn = 0.9 As.fy (d-a/2)

No — Is MULT > ± MDES? — Yes

Reinf. for flexure is ok.
Check slab thickness

Calaulate dist. reinf., Asd = 121/(S**0.5)
and it should not be greater than 67%
S = Effective span of deck slab

Calculate cracking moment
M cr = fr .Ig / yt as per AASHTO '92,
Div. I, Art. 8.13.3 & 8.17.1

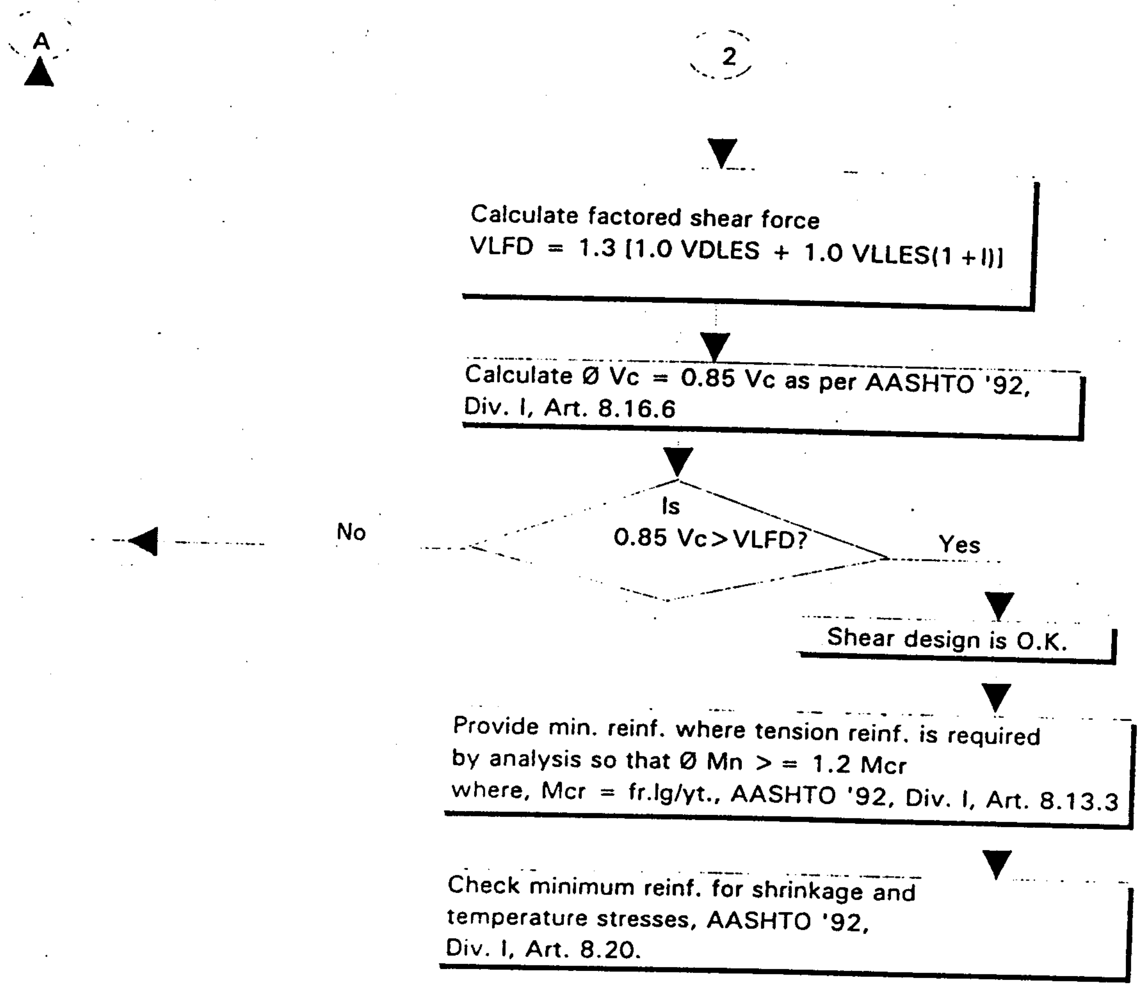
No — Is ØMn > = 1.2 Mcr.? — Yes

Design is O.K.

Check and provide min. reinf.
complying shrinkage requirement as
per AASHTO '92, Div. I, Art. 8.20 .

A

2


A
2
Calculate factored shear force
VLFD = 1.3 [1.0 VDLES + 1.0 VLLES(1 + I)]
Calculate Ø Vc = 0.85 Vc as per AASHTO '92, Div. I, Art. 8.16.6
Is 0.85 Vc > VLFD?
No
Yes
Shear design is O.K.
Provide min. reinf. where tension reinf. is required by analysis so that Ø Mn > = 1.2 Mcr where, Mcr = fr.Ig/yt., AASHTO '92, Div. I, Art. 8.13.3
Check minimum reinf. for shrinkage and temperature stresses, AASHTO '92, Div. I, Art. 8.20.

CHAPTER 10

DESIGN OF SUBSTRUCTURE

10.1 General

Substructure of a bridge is the portion between the elastomeric bearing and the pile cap which transmits loads from the superstructure to the foundation. Generally abutment-wing wall, pier, tie wall or counterfort. etc. are considered as substructure of a bridge.

10.1.1 Allowable Stresses of Materials

Concrete : 28 days cylinder strength in accordance with ACI-83 (Revised '92) $f_c' = 20$ N/mm^2

Steel : ASTM A615 M-88 Grade 40 deformed and plain bars $f_y = 276$ N/mm^2

10.2 Longitudinal Load

Longitudinal loads on the abutment-wing wall are due to the following lateral load items:

- Active pressure due to saturated backfill
- Active pressure due to live load surcharge equivalent to 0.61 meter earth load
- Hydrostatic pressure due to 0.61meter head difference below high flood level (HFL)
- Due to wind
- Due to seismic loading
- Due to stream current force arisen after the passive earth is removed from 3 sides of the abutment-wing wall
- From superstructure due to thermal expansion and contraction, shrinkage, creep and braking of vehicles transferred through the bridge bearings.

Most likely all these loads will not occur simultaneously. The load combinations have been used in accordance with the AASHTO load groups [Table 11.1].

The procedure for calculating the above pressures are given in chapter 6.

10.3 Design Philosophy

The design is made in accordance with the AASHTO LFD method [AASHTO'92, Chapter 9].

The design steps are shown in the Design Flow Chart [Art. 10.4].

10.4 Flow Chart for Design of Abutment - Wing Wall

Start

▼

Select the height of abutment-wing wall, from GAD and Fig. 2.1 of Part A of this Manual.

▼

Select for the wall type abutment height of stem and the backwall depending upon the depth of PC girder and thickness of elastomeric bearing and concrete plinth.

▼

Select flag length and type of wing wall
- Minimum 1.0 m depth of embedment below design slope of embankment (1V:1.5H)
- Length of wing wall = 1.5 x (height of wing wall above pile cap top elevation) - length of toe - 0.725 m

▼

Select tie wall(s) for wing walls, if needed, depending upon the height of wing wall, type of deck and foundation types, Dwg. nos. AF-6A-01 to AF-10I-01.

▼

Select material properties :
- Concrete, fc' = 20 N/mm^2
- MS reinforcement, fy = 276 N/mm^2
- Modulus of elasticity of concrete, Ec = 19.50 kN/mm^2
- Bulk unit weight of submerged soil = 20.00 kN/m3
- Bulk unit weight of submerged backfill material = 20.00 kN/m3
- Saturated unit weight of backfill material = 19.00 kN/m3
- unit weight of water = 10.00 kN/m3
- Coeff. of active earth pressure ka = (1-SinØ')/(1 + Sin Ø')

▼

Select HWL,
Fig. 2.1 of Part A of this Manual.

▼

For structural analysis of abutment, wing wall and tie walls consider the lateral pressures for the following load items:
- saturated back fill pressure
- 0.61 m surcharge pressure
- hydrostatic pressure due to 0.61 m trapped water below HWL.

▼

1

1

Calculate vertical and horizontal moments in abutment wall considering 3-edge fixed support condition for the above lateral pressures

Calculate the B.M. and shear force for the wing wall using the factored load as per AASHTO'92, Table 3.22.1A. Divide the wing wall into various regions for convenience of calculations.

Assume member thickness, dia. and spacing of reinf. steel for the components in each region

Calculate min. flexural strength = 1.2 Mcr
where Mcr = fr.Ig/yt, AASHTO'92, Div.I, Art.8.13 & 8.17.1
-ve Design Moment, -MLFD = Max. of (MLFD & 1.2 Mcr)
+ve Design Moment, +MLFD = Min. of (1.33MLFD & 1.2 Mcr)

Calculate lever arm factor a = As.fy/(0.85 fc'.b).
Calculate flexural strength of each component
Ø Mn = 0.9 As.fy (d-a/2)

No — Is Ø Mn > MLFD? — Yes

Calculate
- ratio of reinf. p
- ratio of reinf. pb
  which produces balanced strain condition, AASHTO,'92, Div. I, Art. 8.16.3

No — Is p < 0.75 pb? — Yes

Flexural Design is O.K.

Calculate shear strength provided by concrete
Vc = 0.166 (sqrt fc') bw.d in SI unit

No — Is 0.85 Vc > VLFD? — Yes

Shear design is O.K.

Calculate deflection for flag portion of wing wall under service load and check adequacy

Calculate reaction/tension forces T on the tie wall under factored load. Assume member thickness, and provide reinforcement, As required = T/0.7fy

Check minimum reinf. for shrinkage and temperature stresses, AASHTO '92, Div. I. Part D, Art. 8.20

CHAPTER 11

DESIGN OF CAST-IN-SITU RC PILE FOUNDATION AND PILE CAP

11.1 General

Deep pile foundations have been provided below 6 to 10 meter high abutment-wing walls. Cast-in-situ bored piles have been used in place of precast piles for the following reasons :

- Vertical piles require relatively larger size to cater for flexural stresses due to lateral load on the pile head. For the standard structures of this Manual 600 mm diameter vertical piles have been provided. The alternative was to provide raker piles in which case pile diameter could be reduced. But the piling rigs for approximately 600mmx600 mm size precast vertical piles; or for raker piles may not be available in the region.
- Cast-in-situ bored piles can be installed upto any reasonable length required by design with the rig available in the region.
- Cast-in-situ bored piles can be installed in almost any type of soil by using bentonite slurry plus temporary casing at the top only, provided the soil contains more than 10 per cent fines.
- In the limited cases of purely granular soil, permanent casing might be required for which special design will be needed.
- The pile diameter has been restricted to a single size of 600 mm to facilitate easy installation with the available rigs.

11.1.1 Cast-in-Situ Bored Pile

Design Philosophy

The piles have been designed as laterally loaded piles. For 6 to 10 meter high abutments, the lateral load due to backfill is considerable. The group of vertical piles have been provided to cater for the axial and lateral loads. Raker bored piles have been avoided as construction of the same is difficult with the technology available in the country.

Scour below RC pile cap is likely to occur in case of extreme flood unless the bottom level of pile cap is placed at maximum scour level. In that case deep excavation below ground water table(GWT) might require lowering of the GWT either by using turbine pump or well point dewatering system for construction of the pile cap in dry bed. This is an expensive method of construction - as an alternative the bottom elevation of pile cap has been provided at a higher elevation by providing a minimum of about 1.0 meter earth cushion above top of the pile cap. Further, it is to be ensured that the maximum scour depth below pile cap bottom does not exceed 1.0 meter for the standard structures of this Manual.

Service Load on Pile

This has been calculated in accordance with the AASHTO load groups given in AASHTO'92, Div.I, Table 3.22.1A, considering the entire abutment-wing wall and the pile cap as one unit The flow chart given in Art. 11.2 gives the design steps for the stability analysis and for determining the service and factored vertical and horizontal loads on each pile row.

Structural Design of Pile

The structural analysis and design have been made based on the AASHTO LFD method. The factored vertical and horizontal loads have been calculated for all applicable AASHTO load groups. The critical load is normally the combination which gives the minimum vertical and maximum horizontal load per pile. While calculating the horizontal load per pile the RC pile cap has been considered as a rigid plate distributing total longitudinal load at foundation level equally amongst all piles.

The lateral load capacity of each pile has been calculated based on the lateral subgrade modulus of soil. The pile head has been embedded about 75 mm in the pile cap, and in addition all the pile reinforcement have been projected by the anchorage bond length of 40xbar diameter into the pile cap. Accordingly, the pile head has been considered as fixed-headed.
The design bending moment for the fixed-headed laterally loaded pile has been calculated based on the method of Reese and Matlock (Ref: M.J. Tomlinson, Pile Foundation Design and Construction Practice, 3rd. edition).
The pile reinforcement has been calculated based on the Design Charts for Circular Piles given in CP 110 : 1972. For the design steps, refer to flow chart in Art.11.2.

Reinforcement

In the pile, the primary reinforcement (upper cage) has been provided as designed by the above method. A minimum of 1.0% reinforcement has been provided in the lower cage of pile reinforcement in compliance with the AASHTO'92, Div.I, Art. 8.18.1.2 . The spacing of spiral reinforcement has been provided as 75 mm c/c (max.) as per AASHTO'92, Div.I, Art. 8.18.2.2.3.

11.1.2 RC Pile Cap

Maximum pile loads found by LFD method are to be used for pile cap design. While designing pile cap reinforcement along traffic direction, maximum positive moment at toe side shall be calculated by taking moment about the river side face of the abutment wall with a suitable design strip as described in flow chart for design of pile cap (Art. 11.3) . The design loads have been considered as the maximum reaction of piles and minimum vertical downward factored load due to self weight of pile cap and soil fill as per AASHTO'92. While designing pile cap reinforcement along transverse direction, the strip has been considered as the entire area of the pile cap behind the earth face of the abutment. The maximum vertical upward reaction of piles on the pile cap and balanced uniformly distributed downward loads/reactions have been considered for equilibrium of the strip and design purposes. At the base of wing wall a concentrated moment due to backfill and surcharge pressure has been calculated in accordance with either Reynold's Hand book (12th ed.), Table 53 or W. T. Moody, Moments and Reactions for Rectangular Plates, Fig. 1 has been used. Punching shear is checked for a corner pile as by examination it is found that corner piles are critical for punching shear. For conformance to the AASHTO requirement, PR_{max} should be greater than Vc, where

PR_{max} = Maximum pile reaction in LFD
V_c = $\varphi \times 0.332 \sqrt{f_c'}\, b_o\, d$
φ = reduction factor = 0.85
b_o = effective perimeter for punching shear
d = effective depth of pile cap

Reinforcement:

At any section of pile cap where tension reinforcement is required by analysis, the reinforcement provided should be adequate to develop a moment equal to at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture as per AASHTO'92, Div.I, Art. 8.17.1.

Accordingly, the Design Moment φM_n should be greater or equal to $1.2 M_{cr}$

where

$M_{cr} = \sigma_{cr} \cdot I_g / C$

σ_{cr} = modulus of rupture
= $7.5 \sqrt{f_c'}$, f_c' is in psi [AASHTO'92, Div. I, Art. 8.15.2.1.1]
= $19.70 \sqrt{f_c'}$, f_c' is in kN/m^2

I_g = $bh^3/12$

C = pile cap thickness/2

11.2 Flow Chart for Design of Cast-in-situ Bored Pile

Start

Calculate the factored vertical and horizontal loads for the applicable AASHTO load groups

Select design load group which gives minimum vertical load Nu and maximum longitudinal load at pile-head Hu

Calculate maximum scour depth Dmax, Chapter 4.9 of this Manual

Select the sub-soil type below pile cap and around the maximum scour depth

Yes — Is the soil stiff clay? — No — Is the sub-soil granular? — Yes

No

Select
- for normally consolidated clay nh = 700 kN/m3
- for organic soil nh = 150 kN/m3

Select relative density Dr from SPT Ncorr. and then read coefficient of modular variation nh from Fig. 11.1

Select lareral subgrade modulus K from Table 11.2

Calculate stiffness factor R = (EI/K)^ ¼

Calculate Zf = 1.4R

Select pile dia. and calculate stiffness factor
T = (EI/nh)^1/5 where E = Modulus of elasticity of pile concrete kN/m2
I = Second moment of area of pile concrete m4

Select e = depth to max. scour level below pile cap bottom
Zf = assumed depth of fixity below max. scour level = 1:8T

Calculate pile column length L = e + Zf

A

1

A

1

Provide pile length $Lp >= 4T$
or $Lp >= 3.5R$

Develop fixed connection between pile head and pile cap and calculate design moment for the fixed headed pile $Mu = Hu/2(e+Zf)$

Calculate $Nu/h2$ and $Mu/h3$

From BS CP 110: P3:1972 design chart for circular column against $fcu = 25$ N/mm2, $fy = 276$ N/mm2 and ratio of core dia. to pile dia. Hs/h, read steel ratio 100 As/Ac

Is $As < 4\%$?

No

Yes

Reinf. design is O.K.

Provide spirals, AASHTO'92, Div. I, Art. 8.18

11.3 Flow Chart for Design of Pile Cap

Start

▼

Select design method
- AASHTO Standard Specifications for Highway Bridges, 1992, Division I, Part C, Chapter 8, Strength Design Method Load Factor Design

▼

Assume pile cap thickness 'h' and reinf. dia. & spacing

▼

Calculate the factored pile loads for the applicable load groups as per AASHTO '92 Bridge Standards, Div. I , Table 3.22.1A . Select critical load group and maximum vertical load on piles (PMAX = VLFD)

▼

Calculate MLFD&VLFD for toe side along traffic direction

- select strip width
  for corner pile = (½xpile spacing + edge distance of pile in transverse direction).
  for intermediate pile = transverse pile spacing
  consider critical strip width = smaller of the above two
- calculate maximum upward factored load on toe side piles and minimum downword factored load on the strip
- calculate for the critical strip design B.M. Mxx1 about water face edge of abutment wall
- calculate design shear force VLFD1 for the effective strip along waterface of abutment wall.

▼

A

1

1

Calculate MLFD & VLFD for heel side along traffic direction

(i) strip length = distance between earthface of abutment wall at its junction with the pile cap and the heel side edge of pile cap. strip width = out to out distance of the pile cap in the transverse direction.
(ii) calculate maximum factored upward pile load and minimum factored downward UDL on the strip.
(iii) To satisfy equilibrium conditions unbalanced shear will be taken by the pile cap depth along the cut edge.
(iv) Calculate max. B.M. Mxx2 about earth face edge of abutment wall and max. shear force VLFD2.
(v) Calculate - maximum factored downward load on the strip and pile reaction accordingly
- maximum B.M.(±)Mxx3 for the strip and similarly calculate max. VLFD3
- Select the maximum value of B.M. and shear force from the sl. (iv) & (v) above as the design B.M. Mxx4 and shear force VLFD4.

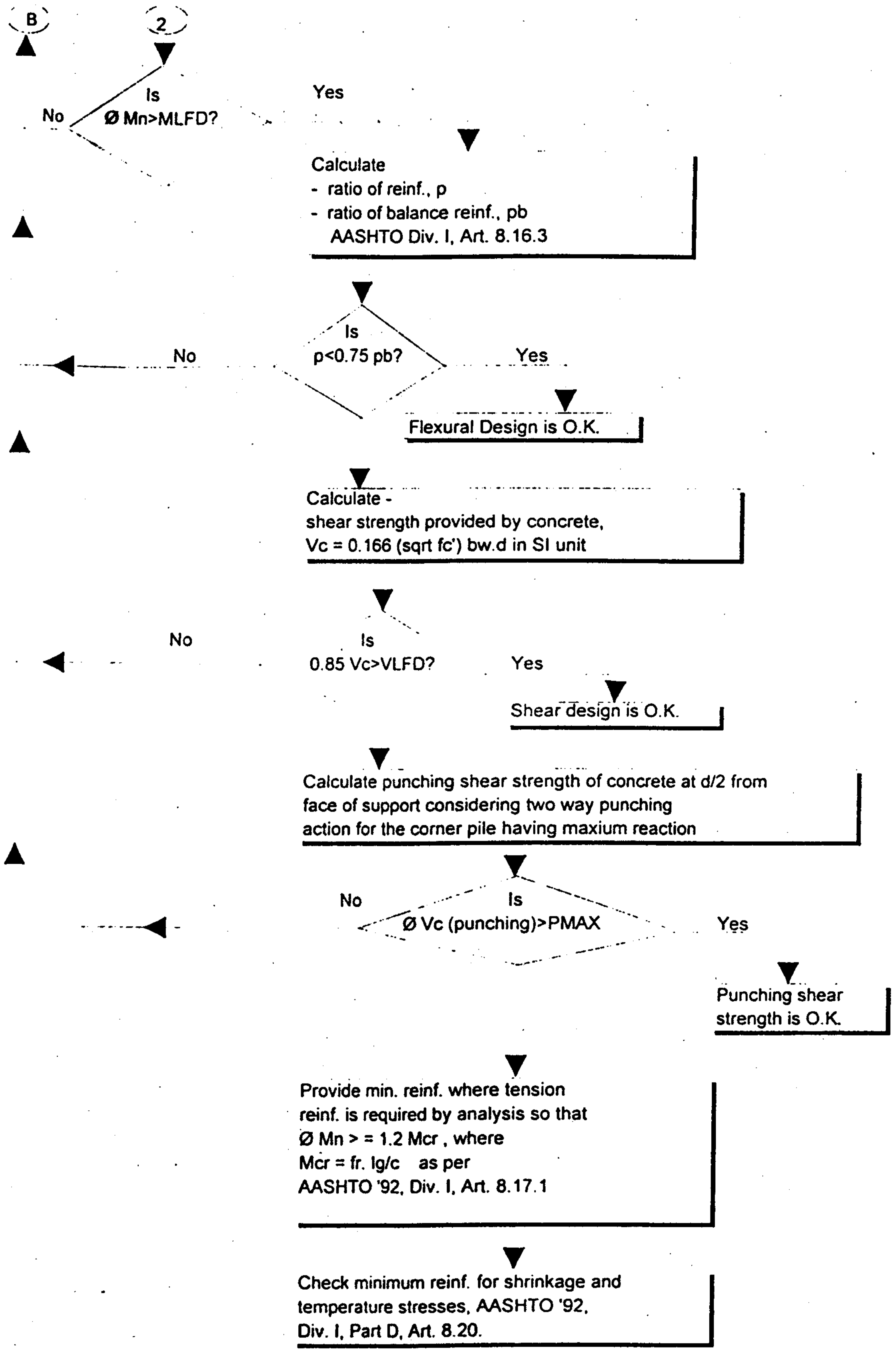
Calculate MLFD & VLFD along transverse direction

- consider the same strip as in sl. (i) above
- calculate max. B.M. and shear force in the transverse direction for the load cases under sl. (ii) & (iii) above
- select design B.M. = Myy1 and design shear force = VLFD5.

Select
- concrete strength, fc' = 20 N/mm²
- reinf. steel strength, fy = 276 N/mm²
- embedment depth of pile inside pile cap = 75 mm
- clear cover to pile cap reinf. above pile head = 75 mm

Calculate ult. moment capacity of pile cap
Ø Mn = 0.9 As.fy (d-a/2) where a = As.fy/(0.85 fc'.b)

2


B
2
Is
Ø Mn>MLFD?
No
Yes
Calculate
- ratio of reinf., p
- ratio of balance reinf., pb
AASHTO Div. I, Art. 8.16.3
Is
p<0.75 pb?
No
Yes
Flexural Design is O.K.
Calculate -
shear strength provided by concrete,
Vc = 0.166 (sqrt fc') bw.d in SI unit
No
Is
0.85 Vc>VLFD?
Yes
Shear design is O.K.
Calculate punching shear strength of concrete at d/2 from
face of support considering two way punching
action for the corner pile having maxium reaction
No
Is
Ø Vc (punching)>PMAX
Yes
Punching shear
strength is O.K.
Provide min. reinf. where tension
reinf. is required by analysis so that
Ø Mn > = 1.2 Mcr , where
Mcr = fr. Ig/c as per
AASHTO '92, Div. I, Art. 8.17.1
Check minimum reinf. for shrinkage and
temperature stresses, AASHTO '92,
Div. I, Part D, Art. 8.20.

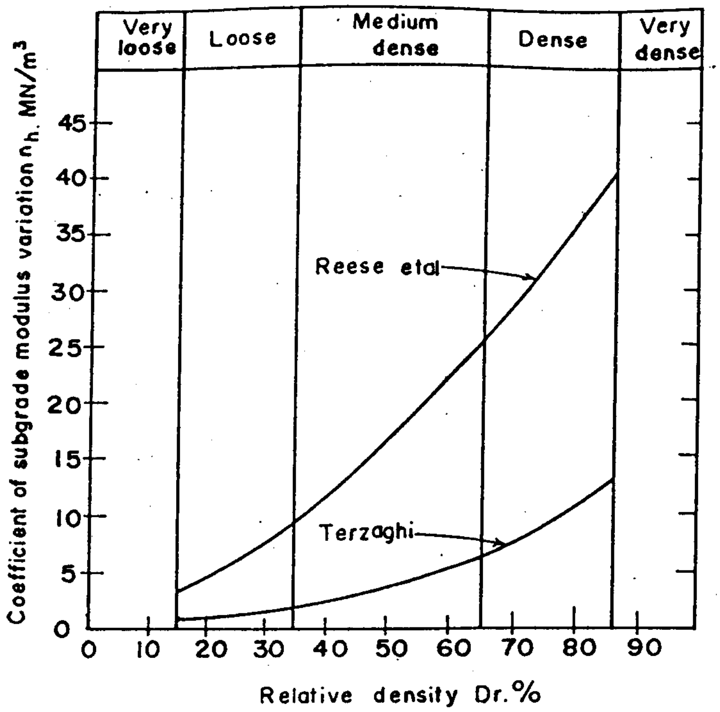


FIG. 11.1 Relationship between coefficient of modulus variation and relative density of sands (after Garassino et al).

TABLE 11.1

a) Relationship between SPT Value (N) and Relative Density

SPT Value (N)	0	4	10	30	50
Relative Density, Dr	0	15%	35%	65%	85%

b) Relationship between SPT Value (N) and Undrained Shear Strength

SPT Value (N)	0	2	4	8	16	32
Undrained Shear Strength, Cu (kN/m2)	0	12	24	48	96	191

TABLE 11.2 Relationship of modulus of subgrade reaction (k_1) to undrained shearing strength of stiff overconsolidated clay (after Elson[6-15])

Consistency	Stiff	Very stiff	Hard
Undrained shear strength (c_u) kN/m²	50-100	100-200	> 200
Range of k_1 MN/m³	15-30	30-60	> 60
Soil modulus (k) MN/m²	3-6	6-12	> 12

Table 3.22.1A Table of Coefficients γ and β

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
GROUP		γ	β FACTORS													%	
			D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE		
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		
	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0		Culvert

$(L + I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L + I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

$$\text{Group } (N) = \gamma[\beta_D \cdot D + \beta_L (L + I) + \beta_C CF + \beta_E E + \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL + \beta_L \cdot LF + \beta_R (R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE] \quad (3\text{-}10)$$

where

- N = group number;
- γ = load factor, see Table 3.22.1A;
- β = coefficient, see Table 3.22.1A;
- D = dead load;
- L = live load;
- I = live load impact;
- E = earth pressure;
- B = buoyancy;
- W = wind load on structure;
- WL = wind load on live load—100 pounds per linear foot;
- LF = longitudinal force from live load;
- CF = centrifugal force;
- R = rib shortening;
- S = shrinkage;
- T = temperature;
- EQ = earthquake;
- SF = stream flow pressure;
- ICE = ice pressure.

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs).

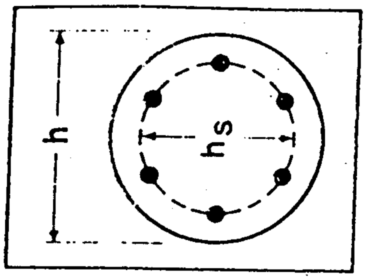
For Load Factor Design

- β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts.
- β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames.
- β_E = 1.0 for vertical earth pressure
- β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity .....For
- β_D = 1.0 when checking member for maximum axial load and minimum moment ....Column Design
- β_D = 1.0 for flexural and tension members
- β_E = 1.0 for Rigid Culverts
- β_E = 1.5 for Flexible Culverts

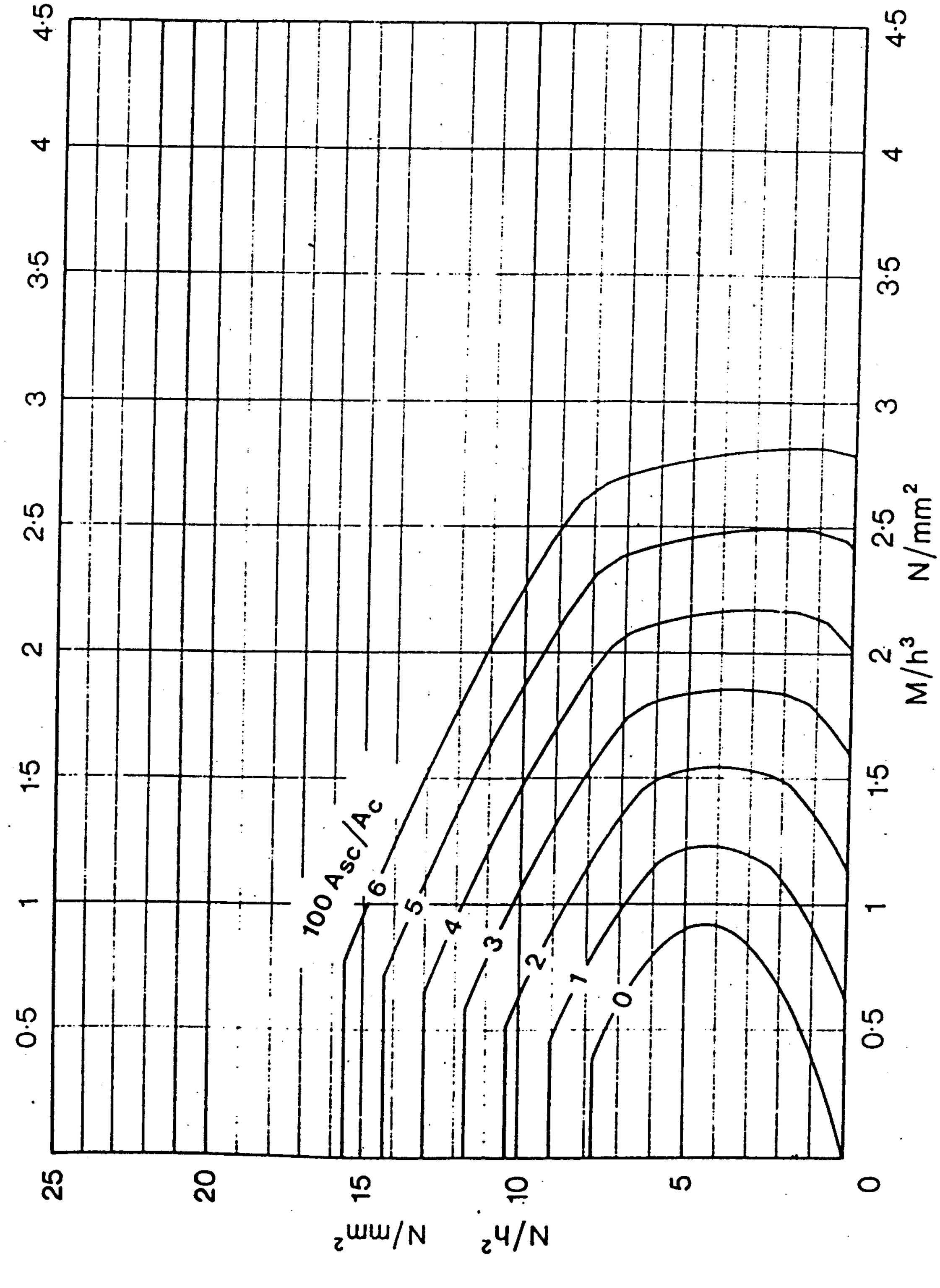
For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

$A_c = \frac{\pi h^2}{4}$

A_{sc} = total area of reinforcement



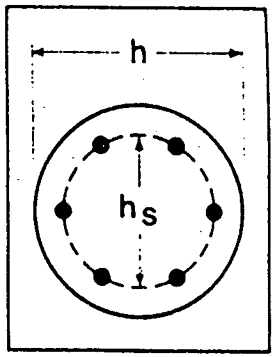
f_{cu}	25
f_y	250
h_s/h	0·70



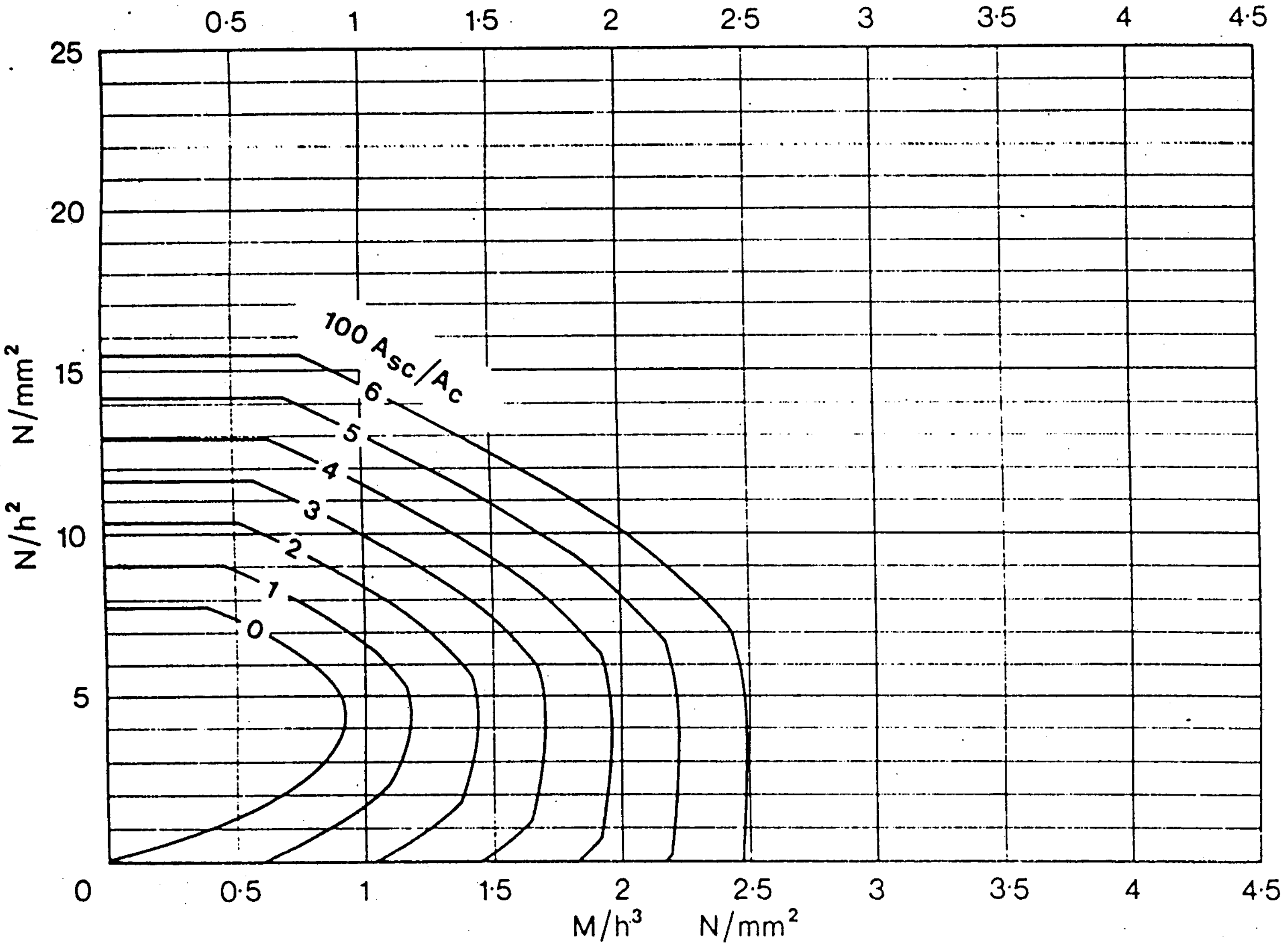
Circular columns

$A_c = \frac{\pi h^2}{4}$

A_{sc} = total area of reinforcement



f_{cu}	25
f_y	250
h_s/h	0·60



Circular columns

CHAPTER 12

DESIGN OF REINFORCED ELASTOMERIC BEARING

12.1 General

An elastomeric bridge bearing is a device constructed partially or wholly from elastomer, th purpose of which is to transmit loads and accommodate movements between a bridge and it supporting structure. The elastomeric bearing included in this manual consists of multiple layer o elastomer bonded to internal steel laminates by the process of vulcanization. The bearing allow translation and/or rotation of the superstructure through elastic deformations.

12.2 Material Properties

The shear modulus of elastomer at 73° F shall be used as the basis of design. If the material is specified by its shear modulus, that value shall be used in design and other properties shall obtained from the Table given below. If the material is specified by its hardness, the shear modulus shall be taken as the value from the range for that hardness given in the Table below. Materials with a shear modulus greater than 1.43 N/mm^2 or nominal hardness greater than 60 shall not be used for reinforced bearings. However, properties of elastomer considered in designing the elastomeric bearings of this Manual are presented in the following Table 12.1.

Table 12.1 Properties of Elastomer

Hardness (Shore 'A')	50
Shear modulus at 73°F (N/mm^2)	0.68 - 0.93
Creep deflection at 25 yrs (Instantaneous deflection)	25%
$\bar{k}$	0.75

12.3 Design Requirements

The design procedure for reinforced elastomeric bearing as mentioned in AASHTO'92 has been presented in the following subsections. It is based on service loads and requires that no impact fraction be added to the live load.

12.3.1 Compressive Stress

Unless shear deformation is prevented, the average compressive stress σ_c in any layer shall satisfy.

$\sigma_c \leq GS/\beta$
≤ 6.896 N/mm^2 (1000 psi) for rectangular bearings

These stress limits may be increased by 10 per cent where shear deformation is prevented.

where,

G = shear modulus of elastomer (N/mm^2) at 73°F
S = shape factor of one layer of bearing
= Plan area / Area of perimeter free to bulge

β = Modifying factor
= 1.0 for internal layers of reinforced bearings
= 1.40 for cover layers. If slip is prevented under all circumstances, 1.0 can be used at the discretion of Engineer.

12.3.2 Compressive Deflection

The compressive deflection, Δ_c of the bearing shall be so limited as to ensure the serviceability of the bridge. Instantaneous deflection shall be calculated as $\Delta_c = \sum \varepsilon_{ci} h_{ri}$
where,

ε_{ci} = Instantaneous compressive strain in elastomer layer number i. Value of ε_{ci} shall be obtained from design aid presented in Figure 14.4.1.2A, AASHTO'92.
h_{ri} = Thickness of elastomer layer number i.

12.3.3 Shear

The horizontal bridge movement shall be taken as the maximum possible deformation caused by creep, shrinkage, post tensioning combined with thermal affects. The maximum shear deformation of the bearing. Δ_s, shall be taken as the horizontal bridge movement, modified to account for the pier flexibility and construction procedures.

The bearing shall be designed so that
$h_{rt} \geq 2\Delta_s$

where,

h_{rt} = Total elastomer thickness of the bearing. ($\sum h_{ri}$)
Δ_s = Shear deformation of the bearing in one direction from the undeformed state.

12.3.4 Rotation

The rotational deformation about each axis shall be taken as the maximum possible rotation between the top and bottom of the bearing caused by initial lack of parallelism and girder end rotation.

Rotation about each axis shall be limited by

$Q_{TL,x} \leq 2\Delta_c/L$
and $Q_{TL,z} \leq 2\Delta_c/W$

where,

$Q_{TL,x}$ = Relative rotation of top and bottom surfaces of bearing due to total load about an axis x perpendicular to the traffic direction.
$Q_{TL,z}$ = Relative rotation of top and bottom surfaces of bearing due to total load about an axis z parallel to the traffic direction.
L = Length of bearing parallel to the traffic direction.
W = width of bearing perpendicular to the traffic direction.

12.3.5 Stability

To ensure stability, the total thickness of the bearing shall not exceed the smallest of L/3 and W/3. Where L and W are length and width of bearing respectively as defined above.

12.3.6 Reinforcement

The reinforcement shall be steel and its resistance in 'N' per linear 'mm' at working stress levels in each direction shall not be less than 11.72 h_{ri}. For these purposes, h_{ri} shall be taken as the mean thickness of the two layers of the elastomer bonded to the reinforcement if they are of different thickness.

CHAPTER 13

STRESSING AND GROUTING OF PC GIRDER

13.1 General

The post-tensioning method of prestressing has been used for the PC girder of this Manual. The girders are of 20.80 to 40.80 meter total lengths and will be constructed in the remote areas where skilled supervisors for prestressing works may not be available in all cases. Therefore, the detailed procedures of the storing, handling of the sheath and cables, sheathed cable laying, stressing and grouting are given in the subsequent subsections. Since the anchorages shown on the drawings are based on Fressinet system, Fressinet Int'l Organization's guide for Fressinet Method has been used for detailing of the prestressing and grouting works. For further details. the original literature may be consulted.

13.2 Handling and Placing of Cables

13.2.1 Cable Storing

The bright metal sheaths and cables should be stored under cover.

13.2.2 Cable Placing

The cables must be placed in the form according to the plan and profile shown on the drawings and attached sufficiently and strongly to the reinforcing cage in order to avoid any displacement during concreting.

'U' shaped saddles of 8 mm mild steel welded (preferably) or attached to one branch of the stirrup to fix the sheathed cables. The first support should be 30 cm from the anchorage so as to ensure the cable is co-axial with the anchorage and firmly held during concreting. The spacing of the other supports for 12Φ7 cables should be about 1.15 meters.
The sheathed cables may be pulled into a mild steel cage. Initially the cables are suspended temporarily either from the stirrups or from supports on the side form. Their adjustment and final fixing are made after placing the last cable.

For the required cable profile, points may be painted on side form. The cable need only be adjusted to this profile. For final adjustment it is the good eye of the site engineer which best avoids the cable wobble and sharp changes of radius which increase the friction losses.

After completion of placing but before closing the form (for girders). the engineer must check the following:

a) Cable Profile

The cables must follow a regular profile without wobble. The cable ordinates at the most important sections should be checked.

b) Water Tightness of the Sheath

The sheath should be inspected when placed. Damage caused by handling and placing must be repaired with adhesive tape. A second inspection must be made after final adjustment before closing the form. The grout vents must be checked at this time.

c) Strength of the Supports

The cables must be firmly held vertically and horizontally and must remain in position during concrete placement and vibration. Welded cables are the most reliable cable supports. The anchorage must be strongly attached to the form.

d) Anchorage Zone

The sheath connection with an anchorage sleeve must be water tight. The joint between anchorage and form must also be water tight. The extra length of wire for jack attachment must be checked particularly when the cables cannot be moved in their sheaths after concreting or if replacement is impossible. The cable must be straight and coaxial with the anchorage for a distance of atleast 40 cm. All bursting reinforcement. particularly for anchorages near an edge, must be carefully checked.

Never forget that time spent in doing the job well and carefully checking the cables will avoid a much greater loss of time in the following operations.

13.3 Stressing Equipment and Handling of HT Wires

a) Stressing Equipment

For 12ϕ7 mm wire cables either U-3 or U-5 type Fressinet Hydraulic jacks will be used. Its specifications are as follows:

Item	U-3	U-5
Effective Stroke	200 mm	300 mm
Weight Empty	70 kg	80 kg
Tensioning Piston Area	157.8 cm^2	157.8 cm^2
PRESSURE		
• Max. Tension	490 kg/cm^2	490 kg/cm^2
• Blocking	300 kg/cm^2	300 kg/cm^2
• Max. Return	300 kg/cm^2	300 kg/cm^2

b) Stressing Schedules

The elongation and jacking force for each cable, the minimum concrete strength before stressing and stressing phases and their sequences are given in the drawings of the PC girders.

c) Pre-Start Checks

Stressing can begin when the following conditions are met:

- The concrete strength shown on the drawing has been reached.

- The cable ends are accessible.
- All end forms and recess boxes are removed.
- All obstructions must be removed from the jacking area. Space must be left for the jack opening due to the elongation of the cable: about 6 mm per meter of wire cables.
- The female cone of embeded anchorages must be checked the day after concreting and cleaned if necessary.

d) The Stressing Equipment is in Working Order

The following must be checked:

- Jacks, pumps, leads and all accessories delivered to site correspond to the materials ordered.
- The pump is wired for the power supply available on site (220V or 380V-3 Phase). Provide a generator (13 kVA) if necessary.
- The electrical leads are of sufficient lengths and amperage for the power supply.
- The hydraulic oil in the pumps is upto level.
- The pressure gauges have been calibrated. The recalibration with a master gauge (sent with the pump) must be carried out regularly , every 200 stressing operations or atleast once a week, as well as at any sign of malfunctioning.

e) Handling and Hanging for the Jacks on Site

The jack must be hung from a height sufficient to allow easy vertical and horizontal movement for threading onto the cables to be stressed. A suspension ring is provided on the jack. Because of the ring the body of the jack remains accessible for fixing the wires. Various commercial hoists exists. mounted on wheels, which facilitates moving the jack from one cable to another. If a crane is available on site, it is often used for handling the jack.

f) Site Record with the Supervising Engineer

The schedule with the stressing sequence. the gauge pressure and the elongations should be in the hands of the site supervising engineer. Check that the internal jack friction of 4% is included in the gauge pressure.

13.4 Stressing Procedure

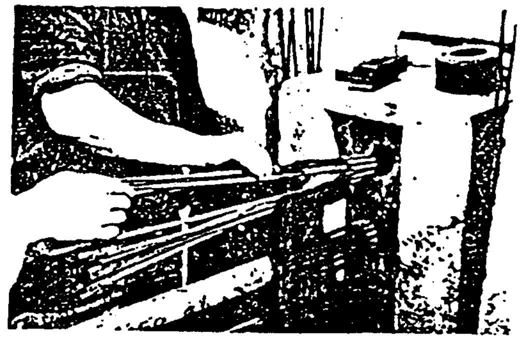
13.4.1 Preparation of the Cable and the Anchorage

The wire surface must be perfectly clean of all substances which could prevent a good anchorage. Traces of mortar must be removed with a steel brush. The internal surface of the female cone must be clean.

13.4.2 Placing the Male Cone

Wire crossing in the sheath just behind the anchorage must be avoided when placing the male cone:

a) Insert the male cone between the wires/or strands so that it is firmly held with each wire or strand in its groove.



When a central spiral is used, it must be removed from the anchorage zone before driving in the male cone. Pull it to one side and cut as close to the anchorage as possible. The end of the spiral springs back into the sheath and will not jam between the male cone and female cone when blocking.

The placing of the male cone is simplified by using a special tool called a star template.

b) Drive the male cone inside the female cone using:

- either a wooden shaft and a small hammer;
- or a steel tube and a sledge hammer for strand cables.

The male cone must project slightly of the outer face of the female cone.

When stressing does not take place immediately after placing the male cones, tie the prestressing wire with a wire to avoid cuts or damage to clothes.

13.4.3 Placing the Jack

- Grease very lightly the grooves of the jack for easy automatic dewedging of jacks using a graphite grease (Belleville Grease with 60% graphite) or, if not available, use candle grease or paraphine. For the jacks which use individual grips to anchor the wires or strands, the inside of the barrels must be greased with the same graphite grease.

- Place the jack on the anchorage with each wire or strand in a groove of the jack head.. Push the jack home.

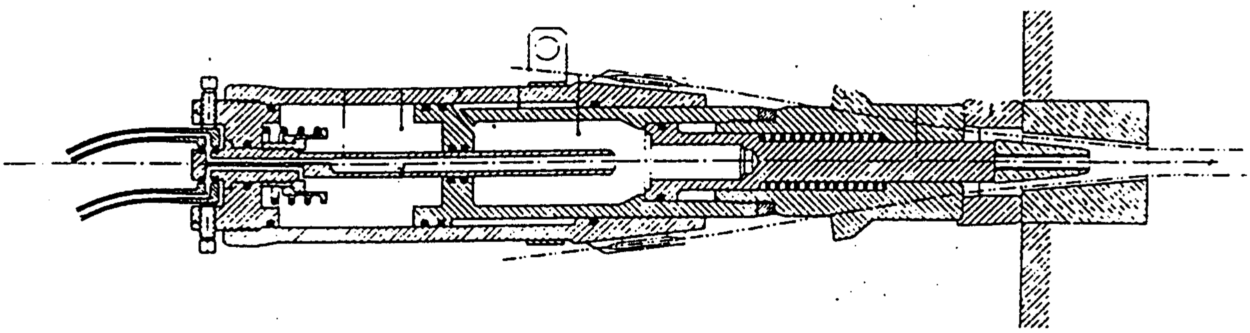
- Place the wedges (for wire jacks) using sharp hammer blows making sure that the wires lie correctly in the grooves of the jack. When the serrations become blocked, soak the wedges in petrol for a few minutes and clean them with a steel brush.

  If the serrations become worn, change the wedges.

- For jack with individual grips, first attach the wires or strands by threading on the barrels and then place the wedges and drive them home using a piece of tube supplied with the jack. Check the wedge serrations and clean them as above or change when necessary.

13.4.4 Stressing

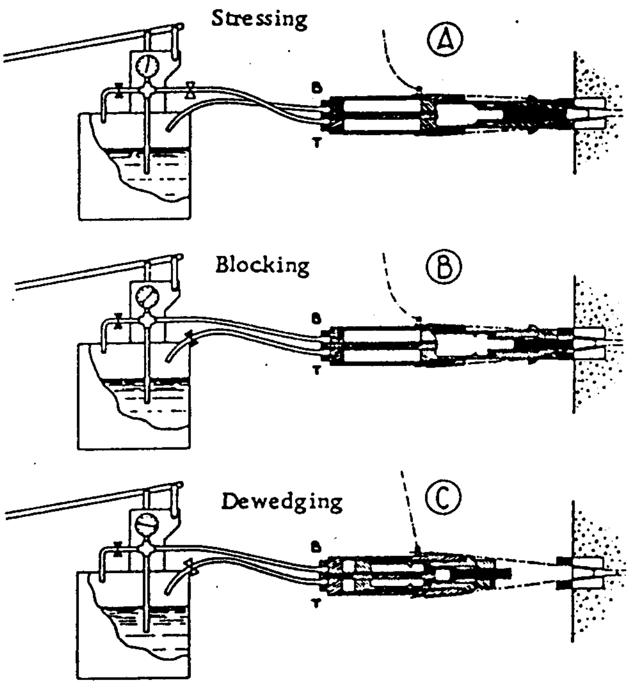
Most construction sites today use motor powered pumps. For the electric pumps used the stressing and return circuits are activated through a distributor. For hand pumps, the leads are alternately connected to the pump for stressing or dipped into reservoir for the return.



The operations must be carried out as follows respecting the order given.

1. Cable Stressing

a) Hand Pump without Distributor

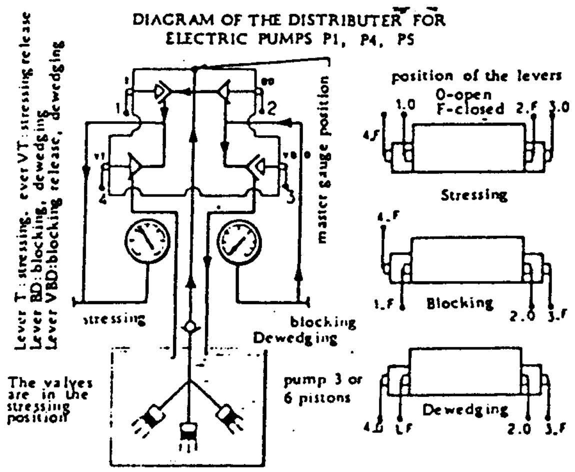


- Hand pump may be allowed upto 25.80 m long PC girders only
- Purge all the leads.
- Attach the lead with valve to the tensioning inlet T of the jack (Fig. A) and to the delivery outlet of the pump.
- Attach one end of the second lead to the blocking chamber B of the jack so as to discharge into the reservoir of the pump (return to the reservoir).
- After stressing to 50 Kg/cm^2 approx. release the jack suspension. Check the wedges by a tap with the hammer.
- Pump until the cable has reached the required stress and elongation and fill out the stressing book.

  Never allow the pressure drop when taking readings. Make elongation readings when the pump is stopped.

b) Electric Pump with Distributor

The same operations are carried out using the levers of the distributor, the leads remaining permanently attached. Pumps P2, P4 and P5 carry schematic diagrams of the lever positions for the various operations.



Important : To pass from "stressing" to "blocking" and from "blocking" to "dewedging" operate levers in numerical order.

2. Blocking the Male Cone and Dewedging

a) Hand Pump without Distributor

- Depressurise the pump by gently opening the return valve; then close the valve (stressing release valve).
- disconnect the tension lead at the pump and connect the blocking lead (fig. B).

  Pump to the pressure given on the technical data sheets (approximately the tensioning pressure).
- Reattach the jack suspension.
- Depressurise the jack by gently opening the valve in the lead. Avoid shocks to the pressure gauge by sudden opening (Fig. C).
- When the pressure is zero continue pumping into the blocking chamber for jack return.

For jacks with wedges, the dewedging occurs at the end of the return stroke when the shoulders of the piston bear on the wedges. With a small rise in pressure dewedging will occur.

If dewedging is difficult (ungreased grooves of the wedges - paragraph 13.5.3 "Jack Placing") shims may be placed between the shoulders and two opposite wedges to release them. The dewedging shock will loosen the other wedges.
For jacks with grips, the grips may be removed as soon as they become loose in their slots. If necessary, tap the barrels with a steel tube.

Depressurise the blocking circuit by opening the emptying valve of the pump.

b) Electric Pump with Distributor

Blocking the male cone is carried out by putting the levers at "blocking". Jack return (and dewedging) is carried out by putting lever 4 at "emptying".

3. Removal of the Jack

After removal of the jack, the bearing ring is best withdrawn by sliding a long length of tube over the wires or strands and then passing the ring over the tube.

4. Safety Measures

During stressing it is absolutely forbidden to stand behind the jack or in its near vicinity as well as behind a preblocked anchorage being stressed from the other end.

Jack side ejection can be caused by poor concrete under the anchorage or insufficient corner or edge reinforcement.

Sudden dewedging (incorrect) or a broken wire can cause ejection of the jack and wedges.

A handrail must be provided if personal are likely to fall when jumping clear of the jack, particularly for cantilever constructed bridges.

5. **Various Recommendations**

1) For two end stressing, the pressure is raised step by step in the two jacks simultaneously. After plugging, dewedging must not be carried out simultaneously but consecutively.

2) The jacks are to be used at pressures of 350 - 600 Kg/cm^2. Refer to the jack brochures or to the technical data sheets for the allowable pressure.

3) When stressing over water or at a height, put a bag over the jack when dewedging so as to catch the wedges or grips.

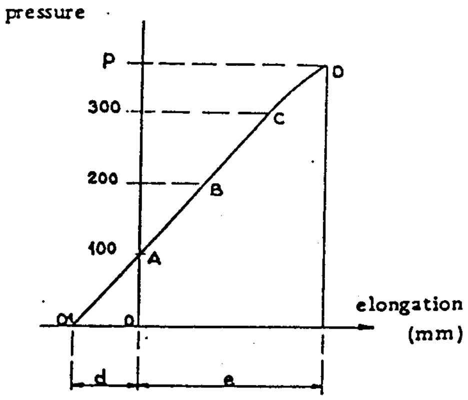
13.4.5 Control of the Cable Tension

The control of the cable tension is one of the most important operation of the whole of prestressing. The result of this control, made under the responsibility of the site engineer, must be recorded in the special booklets called :Stressing Books". Information concerning the steel supplier must be recorded in the booklet.

a) Method of Elongation Measurement of the Cable

At the beginning of stressing, the cable is lying freely in its sheath and the jack is not tightly attached. To remove slack, a small jacking force must be applied to the cable which makes it difficult to define exactly the beginning, i.e. the moment where the tension in the cable is equal to zero and where, in theory, the marks must be placed to measure the elongation.

The fixed zero point is determined in the following manner: the pressure is raised to an arbitrary value, 100 Kg/cm^2 for example, and the marks are placed on the wires or strands. This is point A of the diagram. The stressing is then carried out in steps, 200 then 300 Kg/cm^2, until the final pressure P, and at each pressure the corresponding elongation is measured, thus giving BCD points of the diagram.



For a series of cables of the same type, the number of points recorded to may be reduced.

Because the elongation are proportional to the load in the elastic zone of the diagram the line CBA can be prolonged until it meets the base line at the origin 0_1. The total elongation is thus d+e, e being the elongation from the arbitrary starting point O.

b) Wedge/Male Cone Pull-in

After completion of stressing the pull-in is recorded, so as to check with the values given in the drawing.

The check is essential if the pull-in could have an influence on a critical section of the structure.

The movement of the marks after transfer of the prestressing to the anchorage is measured. From this value must be subtracted the elastic shortening of the section of wire between the anchorage and the marks.

		$12\ \phi\ 7$
Example:	Distance of marks from the anchorage after stressing	430 mm
	Distance of marks from the anchorage after transfer	422 mm
	Distance (movement of the marks)	8 mm
	Subtract the elastic shortening between the anchorage and the marks .	3 mm
	Remainder - pull-in of the male cone	5 mm

In this way the stressing circuit can also be checked for leaks, which produce a cable shortening noticeable as an abnormally high "pull-in". This high apparent pull-in could be from a leak between the end of stressing and blocking: the stressing circuit must be checked.

If the measurements are taken from the bearing ring, always check that the ring is tight against the anchorage when measuring.

c) Irregularities in the Elongation Results

If after reaching the calculated final pressure P in the jack a discrepancy of more than 5% exists between the calculated and measured elongations, a special investigation must be made.

Firstly the pressure-elongation diagram of Art. 13.4.5 must be drawn. Misreadings of marks or gauge are possible and these errors can be detected by misalignment of the points on the diagram. The diagram will also permit the correct location of the point of origin. A bend in the diagram at high pressures is due to plastic fow of the steel as it passes the yield point, resulting in extra elongation.

If calculated elongation A_0 is reached before the calculated gauge pressure P_0, continue tensioning until P_0 provided the elongation does not exceed 1.05 A_0. In case this elongation 1.05 A_0 is reached before P_0, stop stressing and inform the engineer.

If at the pressure P_0 the elongation A_0 has not been reached, continue tensioning by intervals of 5 Kg/cm^2 until the elongation A_0 has been reached or a pressure no greater than 1.05 P_0.

If the elongation at 1.05 P_0 is less than 0.95 A_0 the following measures must be taken, in succession, to define the cause of this lack of elongation.

- Recalibrate the pressure gage;
- Check the correct functioning of the jack, pump and leads;
- Detension the cable. Slide it in its duct to check that it is not blocked by mortar which has entered through holes in the sheath. Retension the cable if free.

If the required elongation is not obtained, no following finishing operation such as cutting or sealing, may take place without approval of the engineer.

13.4.6 Stage Stressing, Restressing and Shimming

a) Stage Stressing

Stage stressing is the operation of partially stressing a cable, and then later restressing the same cable. This operation is necessary when the stroke of the jack is insufficient or when the prestressing must be applied in stages to the concrete.

The male cone must be blocked normally after partial stressing, at a pressure of about 3/4 of the tensioning pressure. The jack is then released normally and the next stage carried out.

When resetting the jack, check very carefully that it has been placed coaxial with the anchorage and that the barrels of the temporary grips have been well seated in their sockets when placing the wedges.

If cables 12ϕ7 have several stressing stages, or the first stage was at a stress greater than 120 Kg/mm^2, cast iron or forged steel male cones are used in preference to concrete male cones.

When removing the jack, care must be taken to not efface the marks for measurement of elongation.

When the first stage jacking pressure is less than 200 Kg/cm^2, cable marked at the beginning of the second stage after checking carefully that the male cone has dislodged.

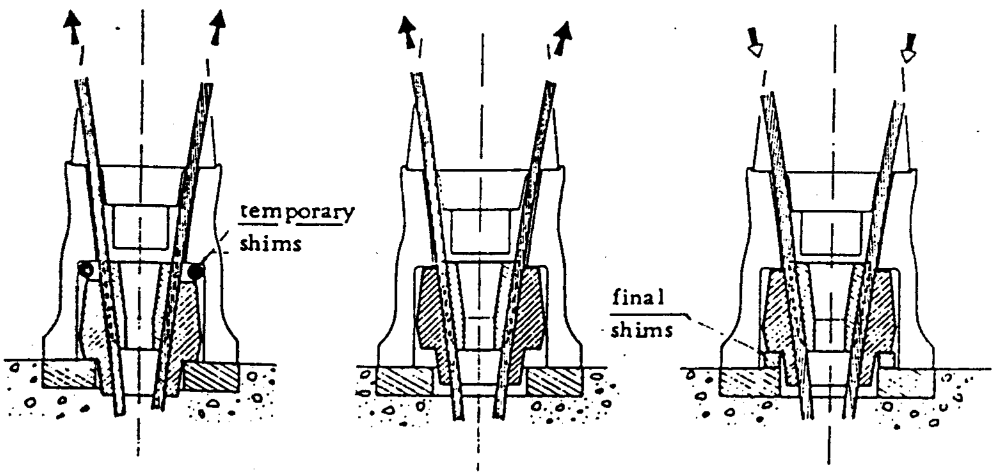
b) **Restressing and Shimming**

It is mentioned earlier that loss of elongation resulting from the anchorage pull-in can be compensated for by restressing to the required value using a shimming foot. An external female cone must be used.

Stressing is carried out in the normal manner. The male cone is blocked in its final position. The jack is reactivated and the force led directly to the concrete outside the female cone by a steel shimming foot.

The female cone is pulled back the required distance (equal to the loss of elongation which must be recovered) and steel shims slid under the anchorage.

The inside of the female cone must be degreased with trichlorethylene or any other solvent.



1 STRESSING THE CABLE   2 RETENSIONING   3 SHIMMING

c) One End Stressing

For short (less than 30 m length) or straight cables, stressing carried out from one end only so as to reduce the cost of prestressing operations. At the nonjacking end a preblocked or dead end anchorage is used.

Before stressing, the male cone at the nonjacking end must be driven to the refusal with a sledge hammer. Use a hard-wood dolly for the concrete male cones and a tube for the steel cones.

The male cone is placed as for the tensioning end. The inside of the female cone must be carefully cleaned. The wires must project at least 20 cm for the anchorages 12ϕ7, so as to allow correct placing of the male cone.

When the cable is marked at the jacking end, the wires at the nonjacking end are marked similarly so as to measure the cone pull-in (to be deducted from the movement of the marks at the jacking end).

The preblocked anchorage must not be sealed until after stressing so as to allow free pull-in of the male cone necessary for a perfect blocking.

d) Detensioning

It is sometimes necessary to detension the cable: for example in the case of temporary prestressing or faulty concrete requiring repair.

For Concrete Male Cones:

- Sharpen two or three high strength steel (ϕ5 - ϕ7 or ϕ8) off-cuts 30 cm long
- Unscrew and remove the end of the blocking ram. If the blocking piston turns during this operation, open it to the end of its stroke and pressurise. Unscrew the end with a spanner.
- Thread on the bearing ring, open the jack to a length equal to the elongation of the cable plus 3 cm, place the jack and attach the wires.
- Stress in order to dislodge the male cone (the pressure rise very quickly). It is possible to see the male cone dislodge through a hole in the bearing ring. At deblocking of the male cone the pressure drops. Continue to stress but never exceed a tension of 95% breaking strength for wires.
- Release the pressure slowly jamming the male cone with the pointed wires to stop it blocking home due to the movement of the wires. Remove the jack after full release of elongation.

For Steel Male Cones:

Detensioning rings are used in which the male cone is held by screws or secial extractors.

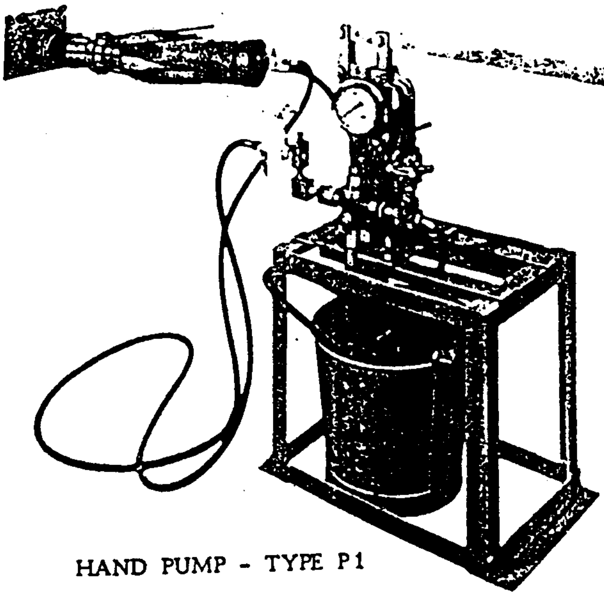
e) High Pressure Pumps

High pressure pumps are available e.g. P1 - P2 - P3 - P4 - P5 of STUP adapted to various site conditions and to the tendons to be stressed

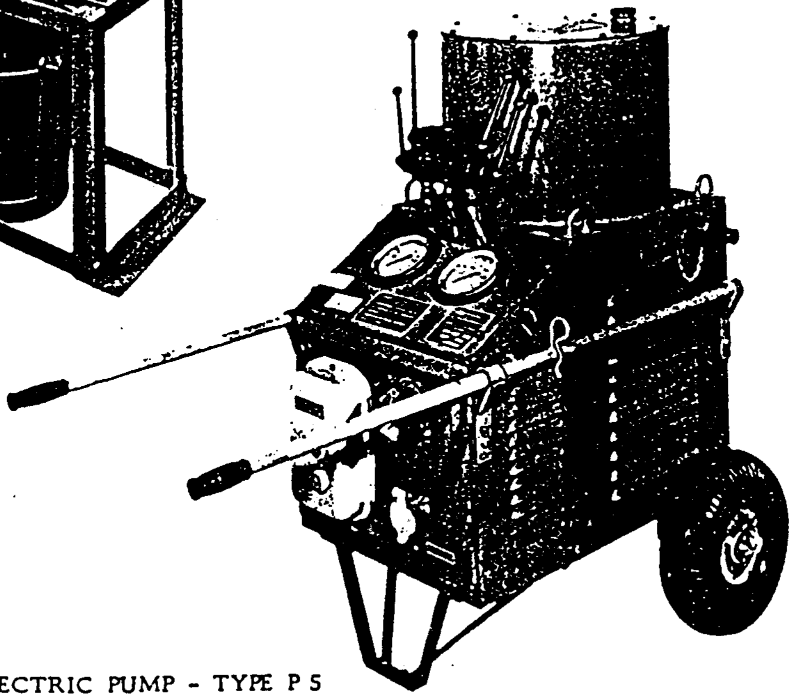
The jacking pressure is very high: depending on the tendons, it varies between 360 and 640 Kg/cm^2.

Generally speaking the pumps are oil operated and activated by electric motors.

The jacks for the smaller system (12ϕ7) require operating pressure between 350 and 400 Kg/cm^2. Water or oil operated hand pumps may be used.



HAND PUMP - TYPE P1



ELECTRIC PUMP - TYPE P 5

The choice of stressing by hand pumps depends on wage rates and the availability of workmen accustomed to heavy work.

The choice of pump depends essentially on the size of the site, the frequency of prestressing operations, and the availability of three phase power of wire cables.

Hand pumps are only used for stressing a small number of wire cables.

13.5 Grouting

13.5.1 General

Grouting with cement mortar of the space between the prestressing steel and the sheath which serves as the duct has two major objectives:

a) To protect the steel against corrosion

The life of the structure depends essentially on the quality of this operation.

b) To effect the bond between the prestressing steel and the concrete.

The following conditions must be present in order to ensure a good grouting operation:

i. The mortar must completely fill the duct, without air pockets or bleeding pockets.
ii. The mortar must not contain any component which could attack the steel.
iii. The mortar must have a strength of about 250 kg/cm^2 after hardening.

In addition good grouting requires the following:

- The duct must in no way have obstacles to the flow of mortar, and must be uniform as much as possible, without sudden changes of cross section;
- The area of the free space inside the sheath, must be at least equal to the prestressing steel area;
- The grouting equipment must be sufficiently powerful to ensure the passage of the mortar from one end to the other of the duct in spite of the head loss;
- The cables must have cut-off at both ends so that the grout can be kept under pressure until it sets.

It must be noted that before beginning grouting, the sealing of the anchorages must have reached a sufficient strength.

13.5.2 Composition of the Mortar

a) Properties of the Mortar

In order to correctly fulfil all of the varied requirements of a good grout the mortar must:

- be homogeneous and therefore mixed mechanically;
- have less than 3% bleeding.

It must be sufficiently fluid to allow grouting with the available equipment.
The consistency of a good grout must be thickish or about that of a thick paint and not a liquid. The table of Art. 13.6.3 gives the water content generally required.

From the preceding the advantages are apparent in using plastifiers. These allow a reduction of the water cement ratio in the mixture and facilitate the maintenance in suspension of the cement particles, improving the workability. The freeze resistance of the mortar is likewise improved by certain additives.

INTRAPLAST Z which is a combined plastifier, reatarder and expansive agent or equivalent may be used. Expansion is due to an aluminium powder in the mixture. For cases where the use of this aluminium powder is not recommended, Intraplast Y can be used, but the benefit of the expansive agent is lost.

Other plastifiers may be used. The supplier must give a guarantee that his product does not contain any component likely to cause corrosion of the steel.

The cement to be used is Portland cement and it must contain absolutely no calcium chloride.

Salt water must not be used for making grout.

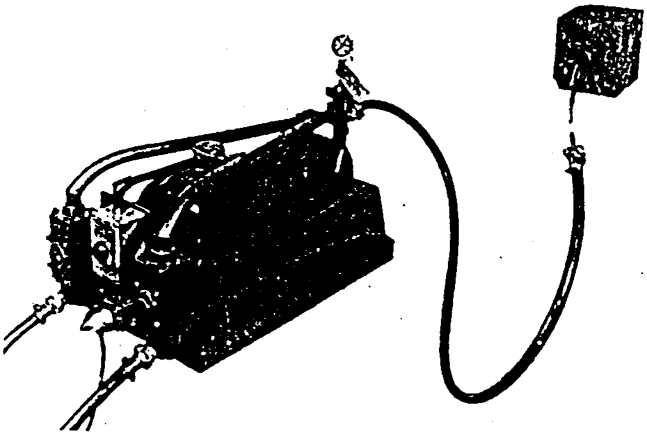
13.5.3 Grout Mixes

Recommended composition:

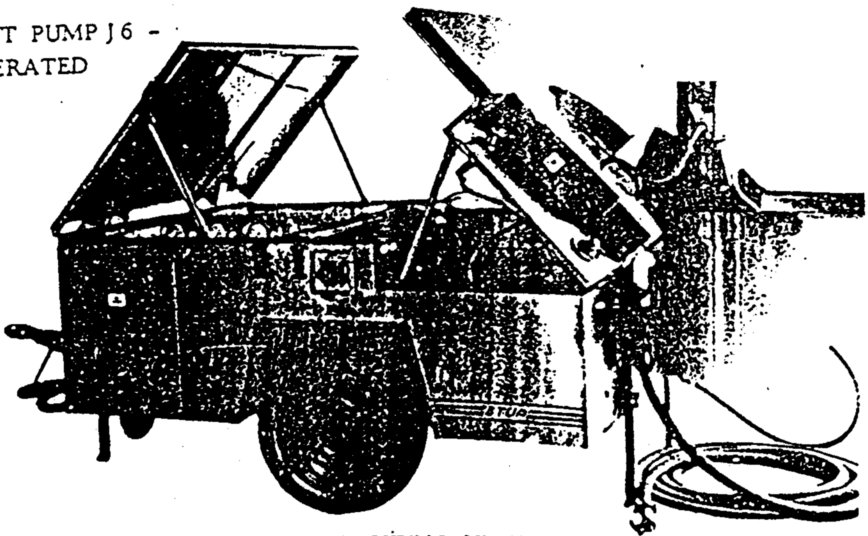
Sieved Portland cement .... 50 Kg
Water .... 17 liters
Plastifier - Intraplast Z (normal case)............. 1 Kg

Hot cements must not be used since the grout obtained is difficult to pump.

The proportion of water given above may be increased (10% maximum) if the grouting pressure becomes too high (greater than 15 Kg/cm^2).



PISTON GROUT PUMP J6 - ELECTRIC OPERATED



CENTRAL GROUTING UNIT

Table 13.1 Quantity of grout and cement per linear meter of cable for sheath dimensions used in France

Type of cable	Internal diameter of sheath (mm)	Calculated volume of mortar per 1 m (liters)	Calculated weight of cement in kg/1m (1.5 kg per liter)	Practical weight of cement (1) (kg per 1m)
12 ϕ 7	36.7 39.8	0.60 0.80	0.900 1.200	1.030 1.370

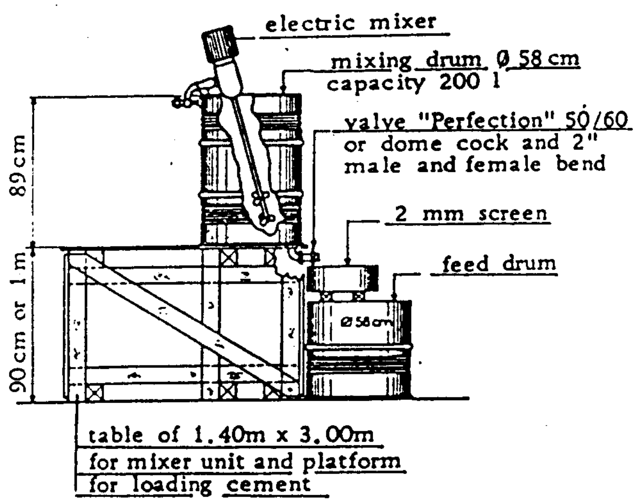
The percentage loss of cement increases as the number, cross section and length of cable decreases. It is therefore inversely proportional to the tonnage of prestressing steel.
In this table the losses vary from 15 to 10 %

13.5.4 Mixing of the Mortar

The mortar is prepared in a mixing drum first by introducing the water, then the cement, the plastifier (when using intraplast Z). The mixing must be started up just before adding the cement. Generally electric paddle mixers are used. Mixing must not be interrupted while adding the cement and must be continued for 5 minutes after adding all components.

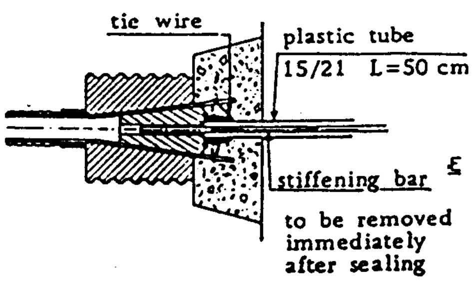
The mixing drum must be placed at a height such that the mortar can flow directly into a second tank, the feed tank, placed beneath the mixing drum. Before flowing into the feed tank the mortar must pass through a 2 mm mesh screen so as to eliminate impurities and lumps.

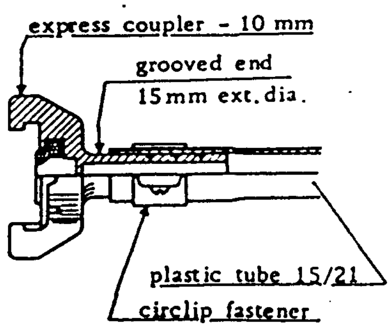
If it is not to be used immediately, the mortar in the feed tank must be stirred from time to time to avoid segregation.



13.5.5 Grouting Details

To ensure a good connection between pump and duct, a plastic inlet must be provided from the anchorage at the time of sealing. Before grouting the sealing must be checked for sufficient strength, in general reached after one or two days. When grouting, the plastic tube is connected to the pump leads using an "express" coupler.





Before grouting a test with compressed air must be made to ensure that the duct is clean and free.

For an oiled cable, or in hot weather condition, or when a bare concrete duct has been used (without sheath), the cable should be washed by pumping water under pressure through the duct and then blowing out with compressed air before grouting.

Grouting must be performed, avoiding any sudden pressure fluctuations. Pressure must be held below a limiting value depending upon the sheath strength and strength of the surrounding concrete (for ducts near outer surfaces). In practice the pressure should not exceed 20 kg/cm^2.

The site personnel who connect the leads and activate the pump must wear protective goggles to avoid accidents such as high pressure mortar getting into the eyes.

When the grout consistency at outlet end is comparable with that at the inlet, the outlet plastic tube is bent back and tied in the same way as a balloon valve. If the pressure falls, it is raised again to about 5 kg/cm^2 and maintained at this level.

The plastic tube at the grouting end is then bent back and tied in the same way.

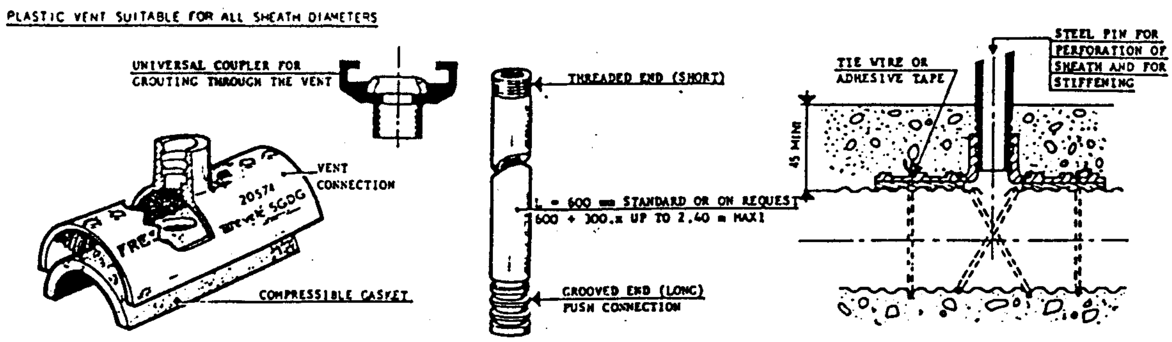
The pressure in the pump is then released through the bypass valve and the leads taken off.

The plastic tubes must not be cut off for 24 hours in summer and 48 hours in winter, when a retarder (Intraplast or equivalent) is used, which is generally the case.

13.5.6 Vents

Under certain conditions drain tubes should be provided at low points of long cables and for cables with a very high drape.

Steel tube vents brazed to a piece of sheet metal bent to a half cylinder can be attached to the sheath by adhesive tape or tiewire after sealing with a mastic. Plastic vents of the same type are also available.



Certain sheath manufactures supply short lengths of sheath with vents which may be screwed onto the normal sheath. It is advisable to extend these vents at the high points by 50 cm plastic tubes thus providing voids into which air and bleed water can accumulate. The vents are closed by bending back and tying the plastic tube.

When grouting draped cables with intermediate vent pipe at the lowest point, this intermediate vent is closed, when the grout consistency is comparable to the inlet grout, and pumping is continued until a good mortar flows from the vent pipe at the opposite end of the PC girder.

13.5.7 Grouting Program

When the sheaths are touching or close together, the grout under pressure could escape through the sheath joints and enter neighboring ducts by passing through regions of porous concrete. Plugs in adjacent ducts can form and prevent their grouting.

A grouting program must be used in which groups of closely spaced ducts are grouted simultaneously or shortly after each other, before any escaped grout from one sheath into another has time to set.

If grouting is occasionally interrupted, neighboring ducts must be checked for blockages. Water is pumped through the duct for this reason to wash out any grout which could have entered.

When using couplers, grouting can be carried out in two phases. A vent must be provided at the couples.

CHAPTER 14

CONSTRUCTION / ERECTION OF PC GIRDER, DECK AND RAILING

14.1 General

The bridge structures of this Manual will be constructed mainly in the remote rural areas. The country is reverine where early monsoon flood is likely to occur. Thus construction period is short. The construction methodology adopted for the bridge superstructure should keep this into consideration.

The composite superstructure of the bridge involves 20-40 meters c/c bearing span PC girders. Cast-in-situ RC deck slab is shear-connected with these PC girders. Erection of scaffolding and formwork for these PC girders and RC deck in the alluvial bed of the moving streams has risk of formwork settlement. Thus in-situ construction of the superstructure by erecting scaffolding from the river bed should preferably be avoided.

The following sections give details of the appropriate construction methodology of the components of the superstructure.

14.2 Construction / Erection Methodology of PC girder

14.2.1 Construction Methodology

The PC girders of this Manual are of post-tensioned type for which basically two methods of construction are available :

- Cast-in-situ Method : and
- Precast Method

Cast-in-situ Method

The PC girders may be constructed cast-in-situ in the following situations :

- For overpasses where props can be supported over the road/ground level below by diverting traffic.
- For construction of bridge across a river or channel whose bed remains dry during the winter season and the construction is done by erecting scaffolding on the dry bed.
- For construction of bridge across a perennially flowing river or channel or for cases where very large height of scaffolding is required, cast-in-situ method of construction by erecting truss support for the girder forms over the adjacent abutment/pier might be used. In that case the construction of the PC girder can be continued during monsoon season also.

The sketch showing typical details of a truss support for PC girder form is shown in Fig. 14.1. The trusses shown in Fig. 14.1(c) shall be used in pair which will be adequately braced together against lateral buckling. This method of construction is suitable upto 30.0 m span length.

Precast Method

In precast method, the girders may be constructed at places adjacent to the exact locations of the girders, immediately below the spans, behind the abutment or in precast yards. After concreting and prestressing of the girders either partially or fully and grouting the stressed cable ducts, these are transported, lifted, shifted and placed in position as applicable [Ref. Fig. 14.2(a)].

14.2.2 Erection Methodology

Lifting and Shifting of Girders

When the PC girders are concreted adjacent to their actual locations, these may be lifted by hydraulic jacks, shifted by sliding and placed in positions by lowering as required.

Fig. 14.2(b) shows the typical details of lifting and shifting arrangement of precast girders.

Using Jin Pole System

After completing the girder concreting adjacent to the spans immediately below, either at dry bed level or by erecting low height scaffolding, this may be lifted and shifted by using Jin pole system.

Jin Pole is an improvised indigenous system for lifting of the precast girders. Jin Pole is actually a steel trestle like arrangement with a central pipe pole. Similar two poles are used to lift the girders by holding them at the two lifting points adjacent to their ends. The stability of the poles are maintained through several guy ropes tied to the anchors at ground level. The pole can be lifted by playing with the guys so that the girders can be shifted or rotated by certain amount. This helps to place the girders in position at the bearing level. Fig. 14.3 shows one typical detail of the Jin-Pole.

Using Launching Girder

The launching girder is a space truss system to move the girders longitudinally to their actual positions from their casting yard behind the abutment. Fig. 14.4 shows the typical plan and longitudinal sections of a launching girder. Fig. 14.5 shows the details of the lifting arrangement using the launching girder.

Using Crane

The girders are concreted in the precast yard either away from or nearer to the location of the bridge. These are then transported by trailers and then lifted and placed in position by using cranes.

14.3 Deck Slab

Deck slab is generally cast on deck forms supported on PC girders. The form supports for the deck including its cantilever projections may be provided by putting inserts in the PC girders. The inserts are double nuts threaded inside the top flange of the PC girders. Deck forms are supported by the girders through bolts threaded inside these nuts. All temporary holes for inserts shall be grouted by pressure grout using non-shrink admixture. A typical details of such inserts are given in Fig. 14.6 .

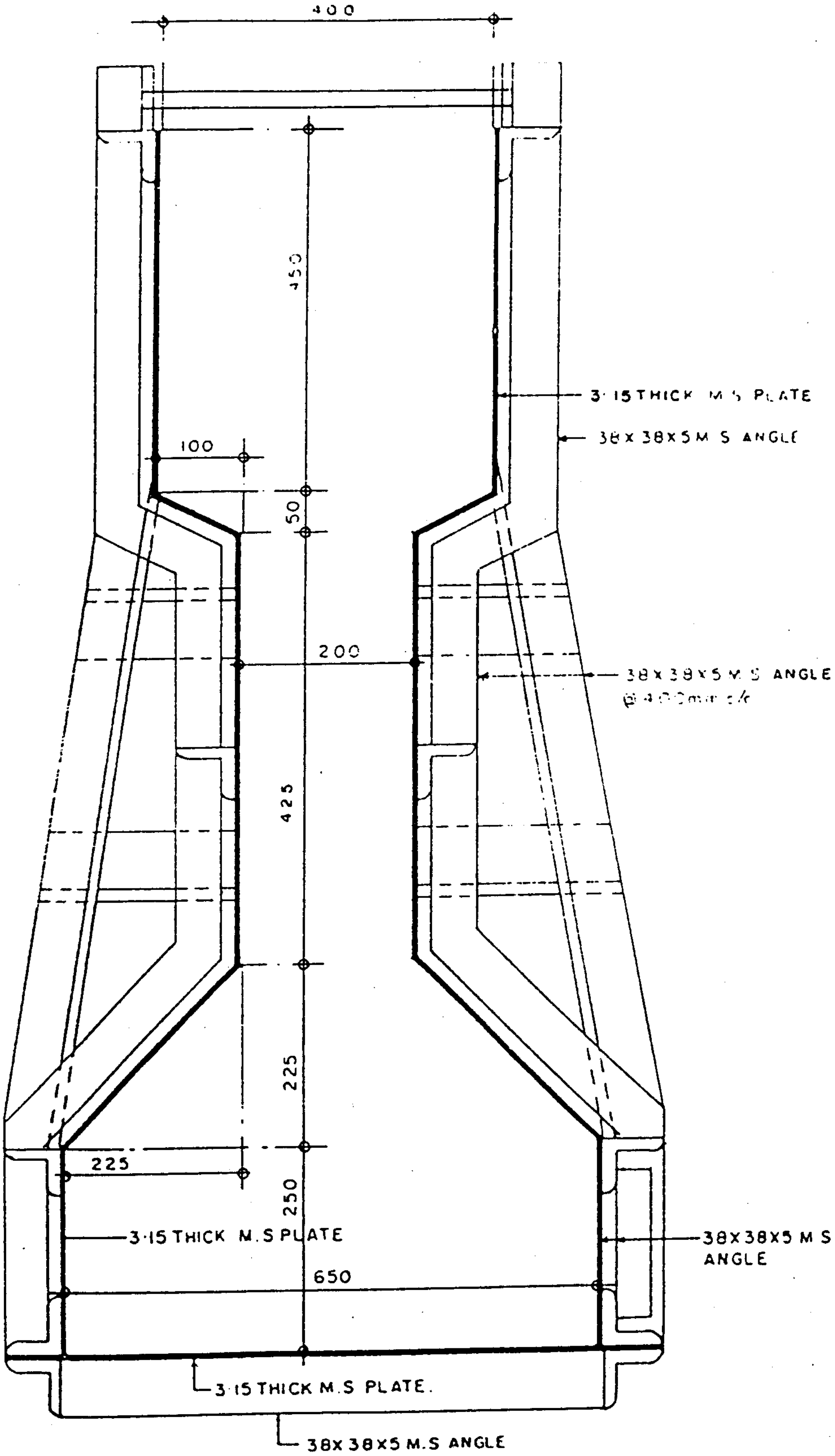
14.4 Rail and Rail Post

The railing system consists of precast rail bars and rail posts as shown on the drawings. The objectives of selecting precast railing system are:

- The standard formworks can be used repeatedly for making the rails in the precast yards and repair and replacement of these during service period will be easy.

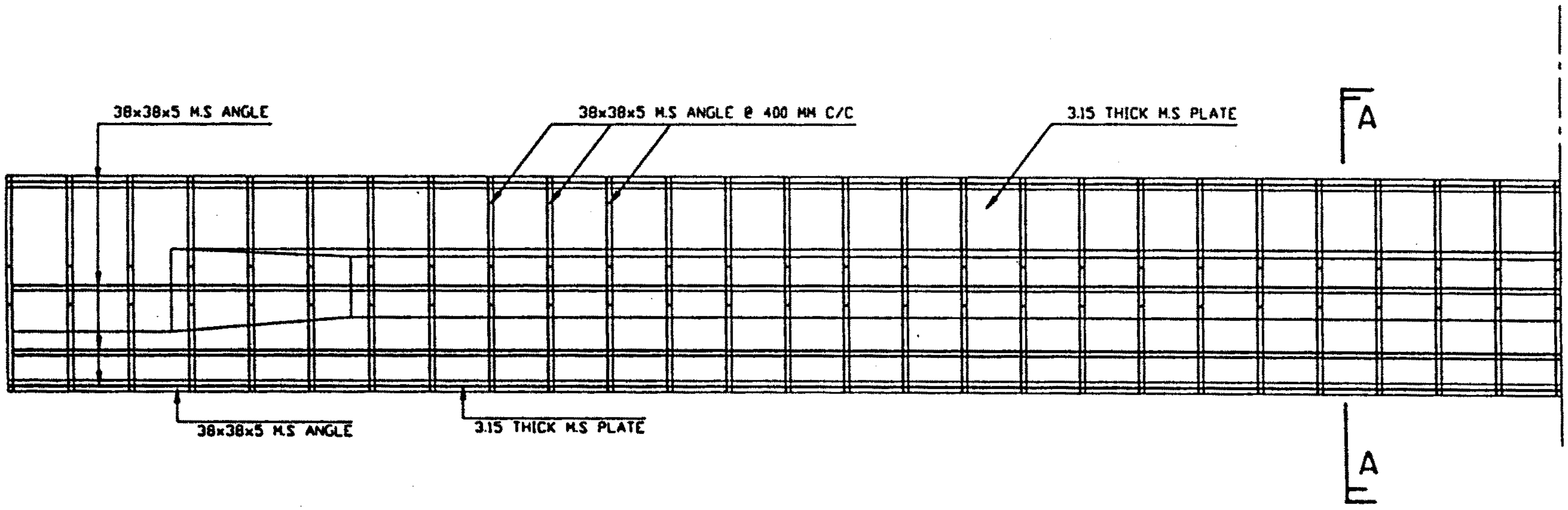
- The repeated use of the standard formwork will make the construction economical.
- Repair and replacement of the precast rail and rail posts will be easier.
- Being precast, the members will have better finish and therefore these will be aesthetically pleasant.
- Fabrication, erection and placement of precast rails and rail posts will be easier and faster.

The precast rail posts will be placed inside preformed grooves. The grooves will be cleaned, wetted and dried to saturated surface dry condition. The empty space between the groove and rail post base will be grouted. Precast rail bars will then be fixed with rail posts by bolts as shown on the drawings and thereafter the recess will be grouted for corrosion protection of the nuts and bolts.



SEC. A-A
TYPICAL SECTION OF STEEL FORM
FOR PC GIRDER

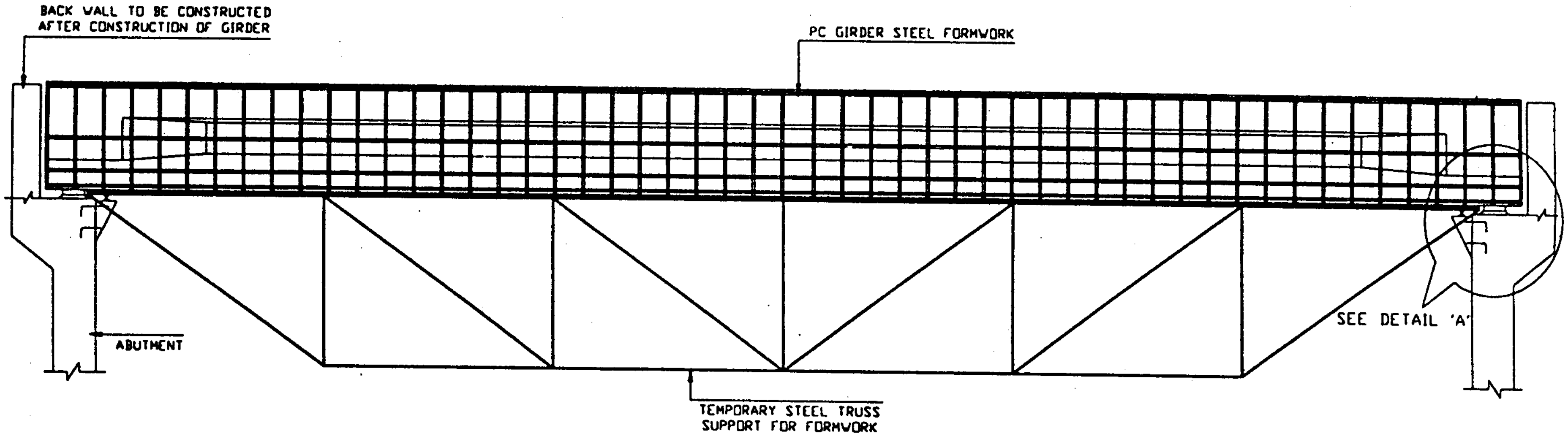
FIG. 14.1(a)



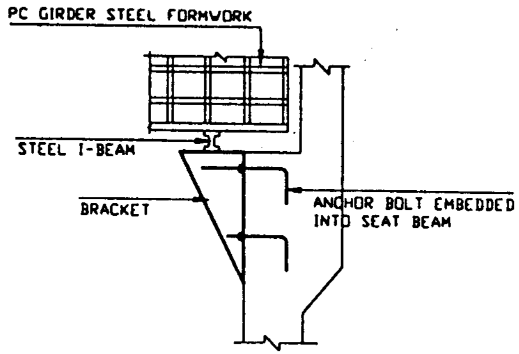
HALF ELEVATION SHOWING STEEL FORM FOR PC GIRDER

FIG. 14.1(b)

DPM-C3\LGED\FORM1 1:90



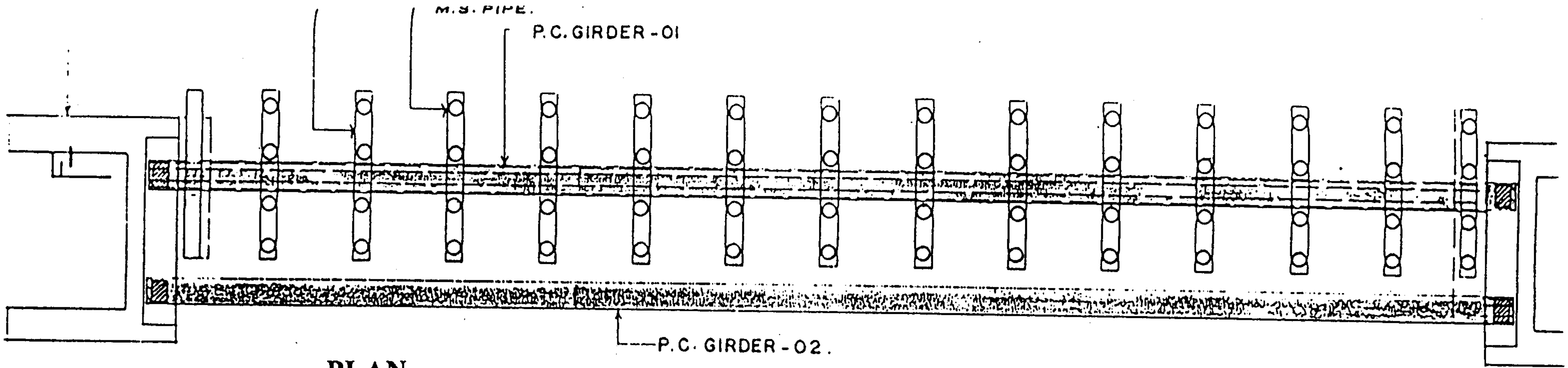
ELEVATION SHOWING STEEL TRUSS SUPPORT FOR FORMWORK (TYPICAL)



DETAIL 'A'

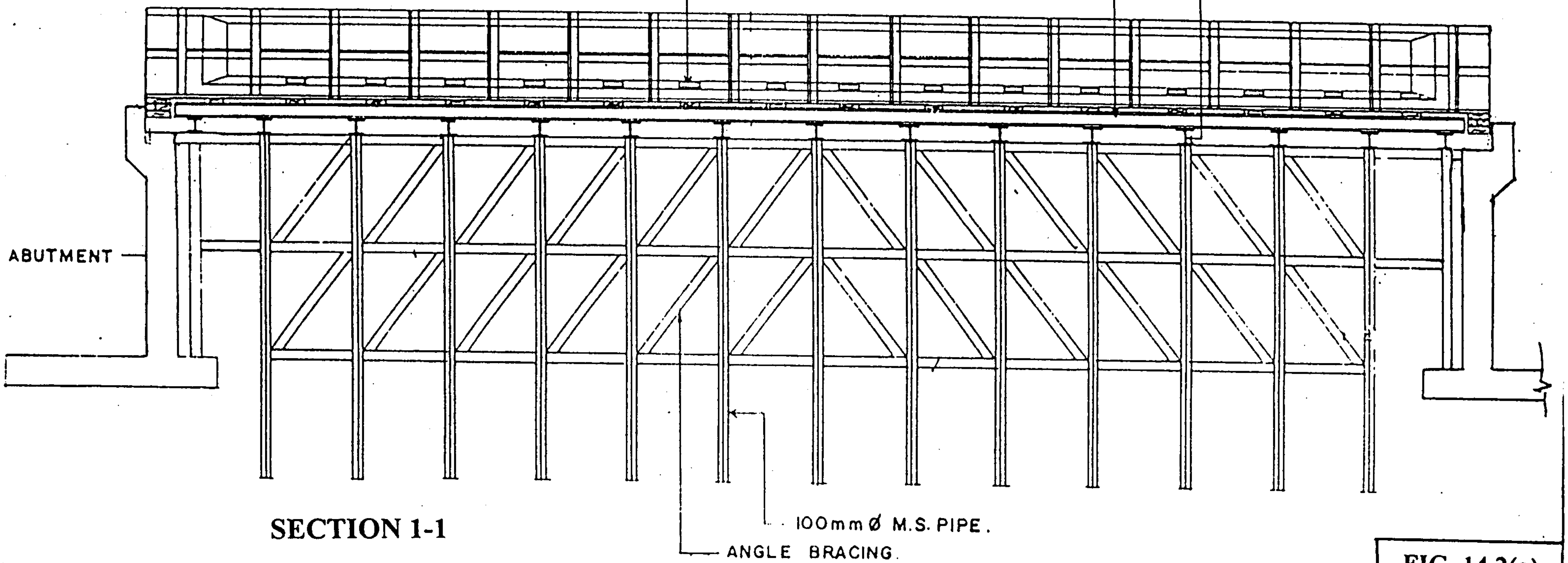
FIG. 14.1 (C)

DPM-C3\LGED\FORM2 1:90



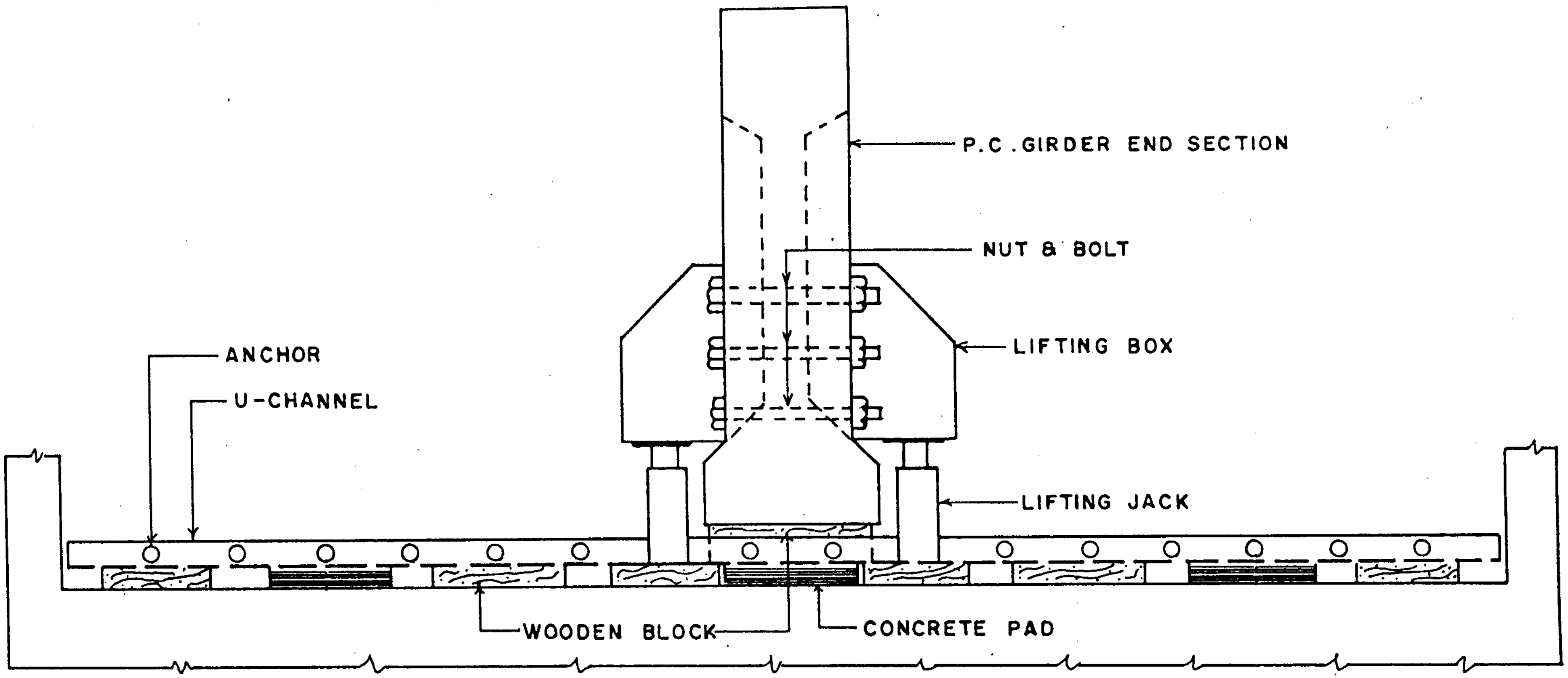
PLAN

PRECASTING GIRDER ADJACENT TO SPAN

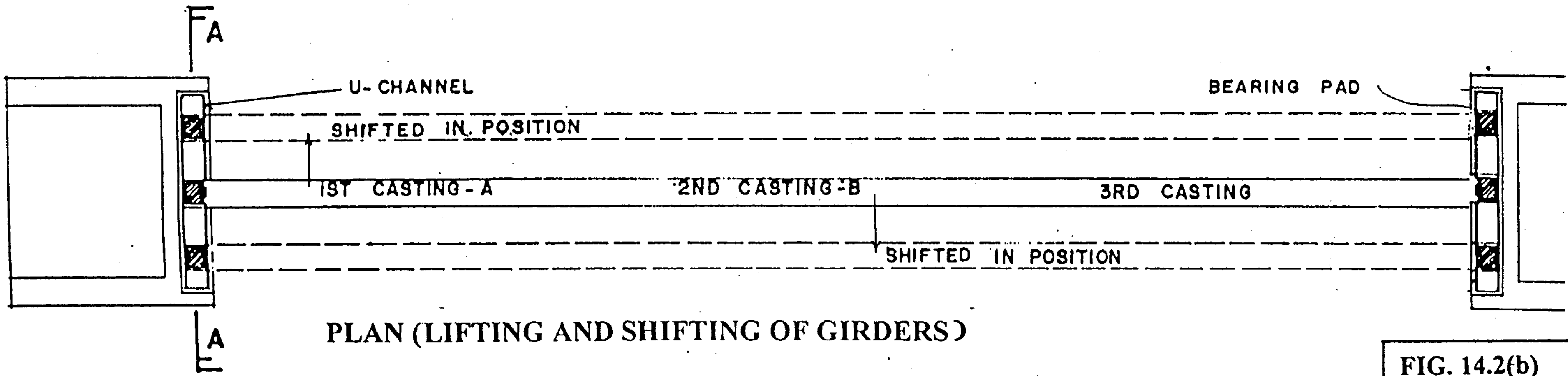


SECTION 1-1

FIG. 14.2(a)

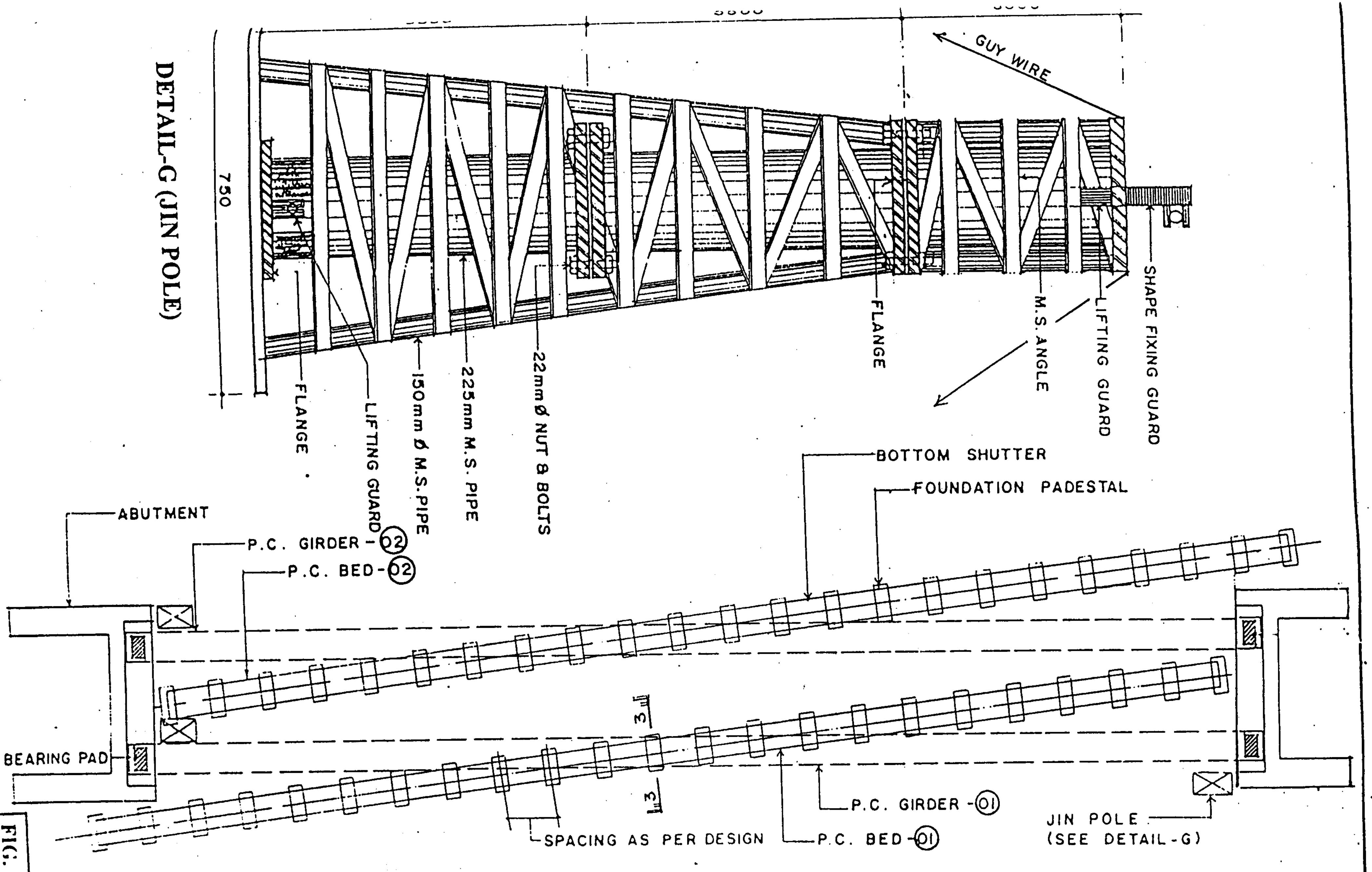


SECTION A-A (LIFTING OPERATION)



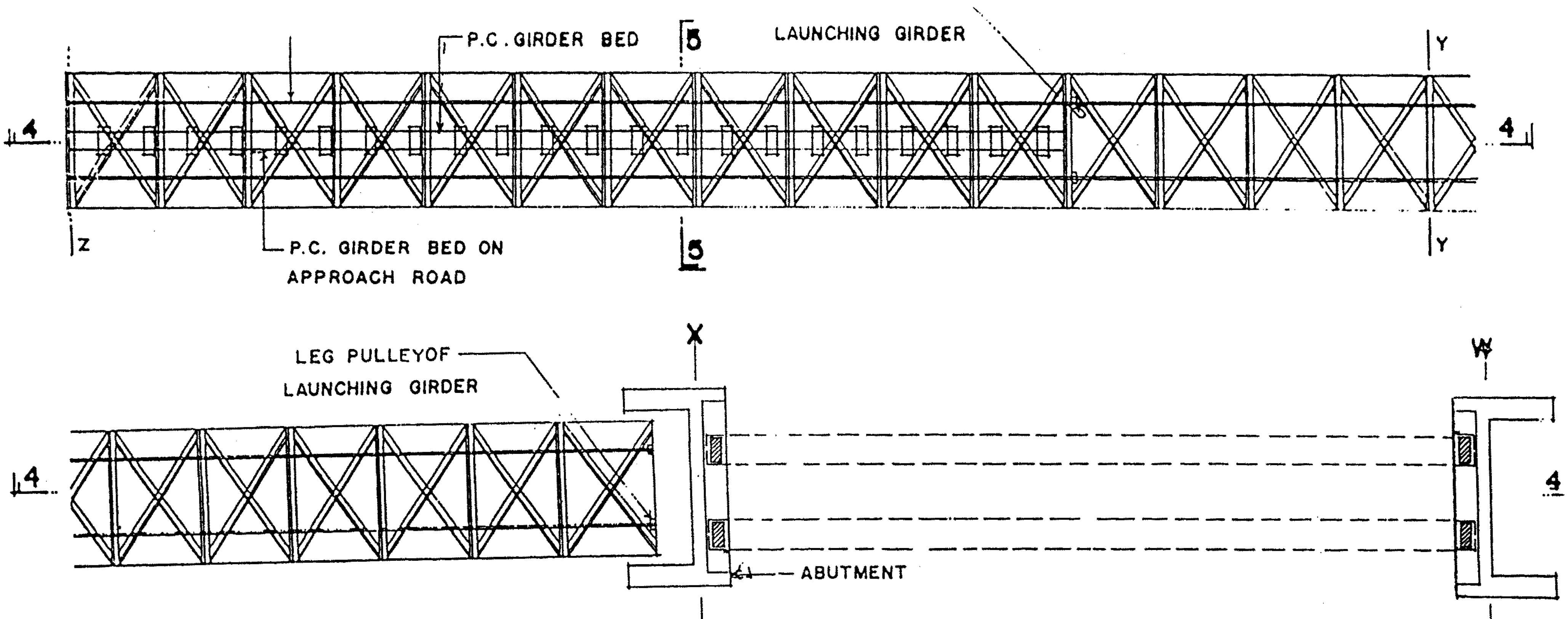
PLAN (LIFTING AND SHIFTING OF GIRDERS)

FIG. 14.2(b)

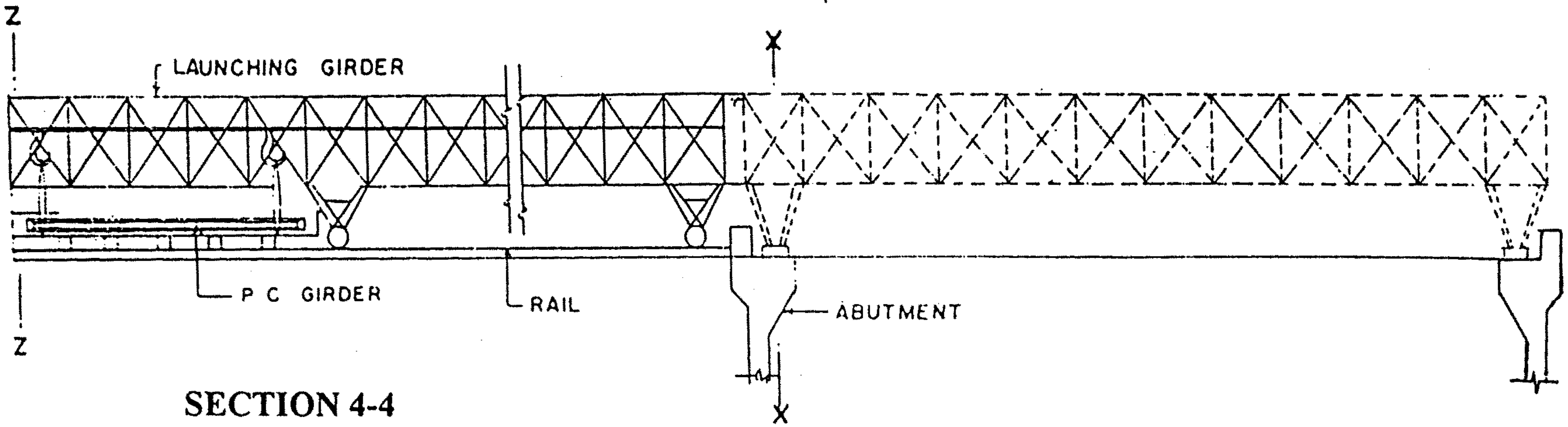


DETAILS OF IN-SITU ERECTION METHOD

FIG. 14.3

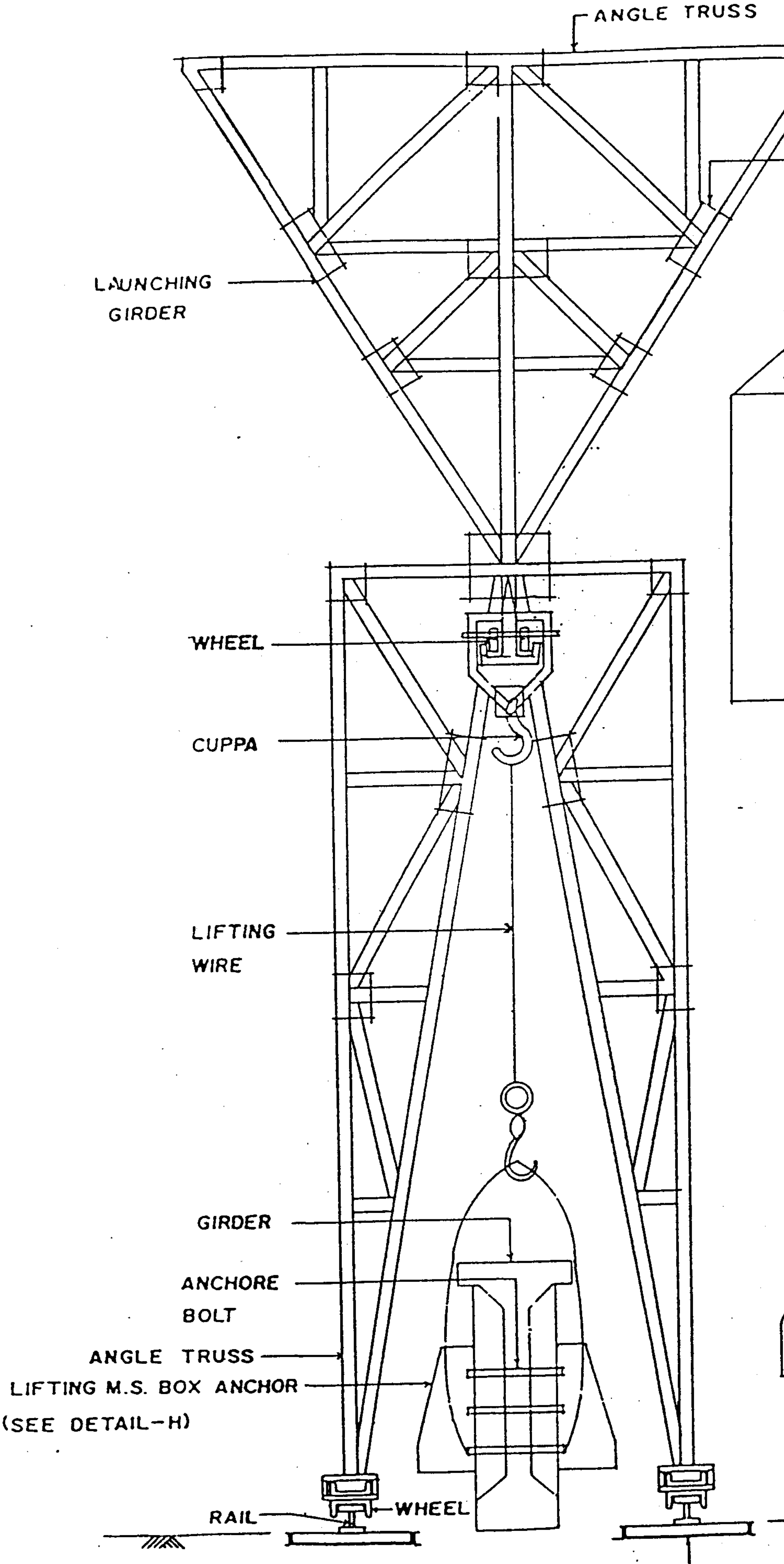


PLAN OF LAUNCHING GIRDER SYSTEM

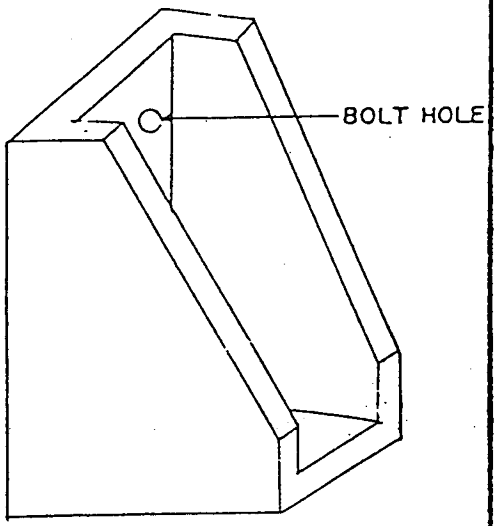


SECTION 4-4

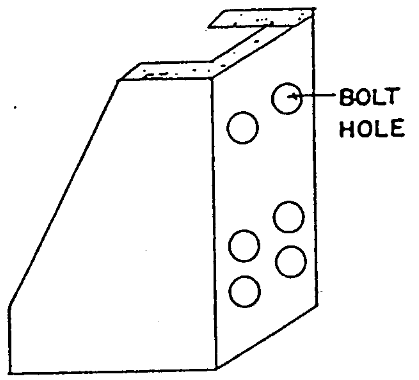
FIG. 14.4



SECTION 5-5

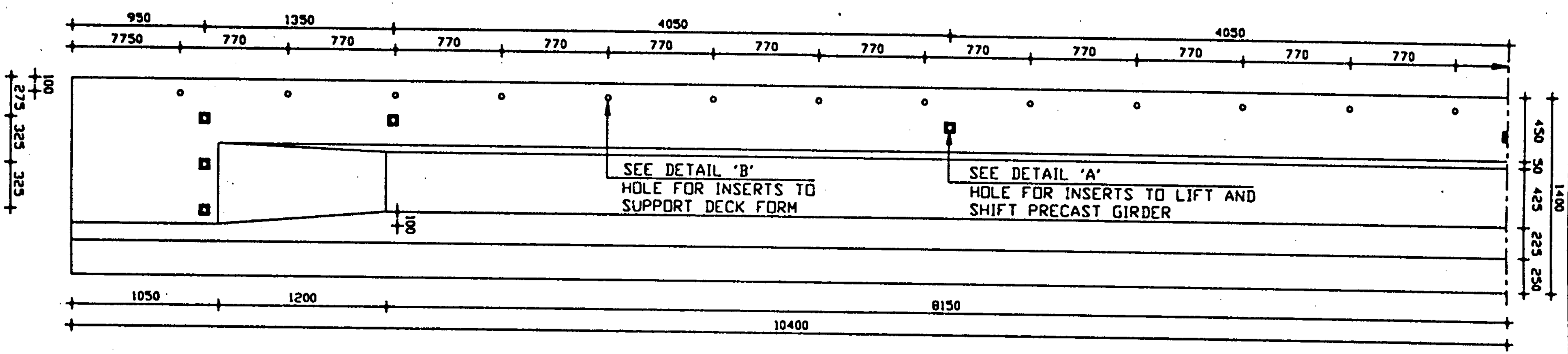


DETAIL-H
FRONT VIEW



DETAIL-H
BACK VIEW

FIG. 14.5



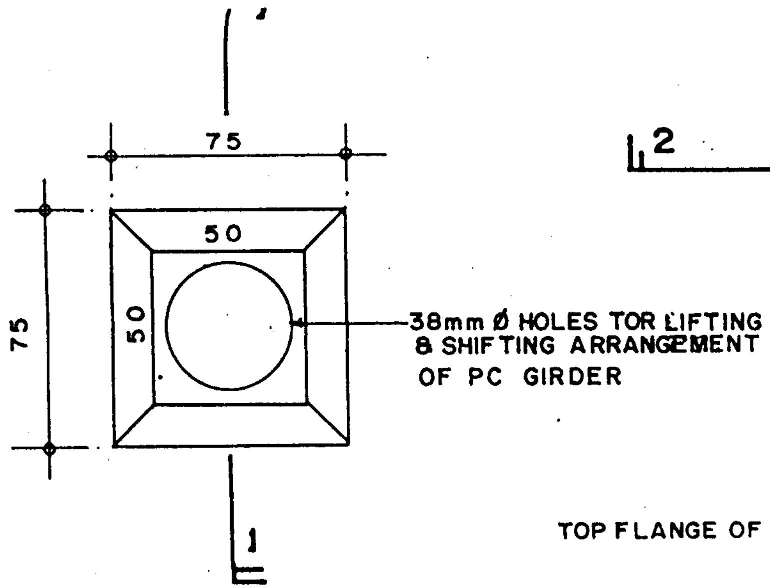
HALF ELEVATION OF A 20.80 m LONG PC GIRDER SHOWING INSERT DETAILS FOR SUPPORTING DECK FORM (TYPICAL)

NOTE:

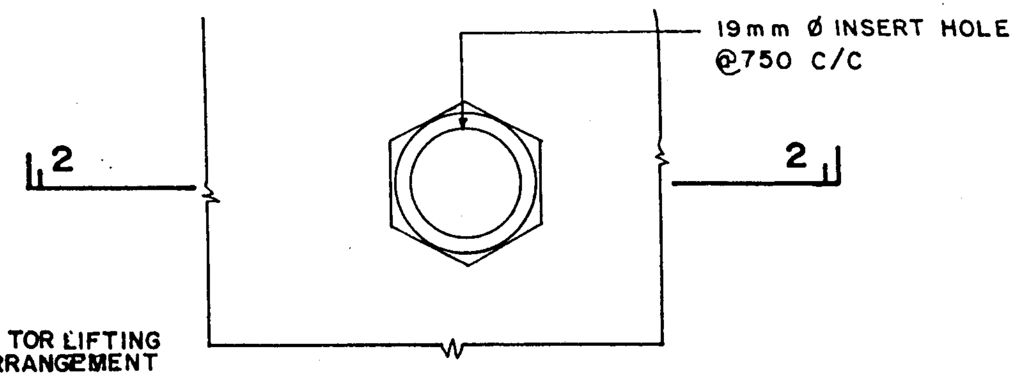
ALL TEMPORARY HOLES AND GROOVES WILL BE FILLED BY PRESSURE GROUTING USING NON-SHRINK ADMIXTURE.

FIG. 14.6(a)

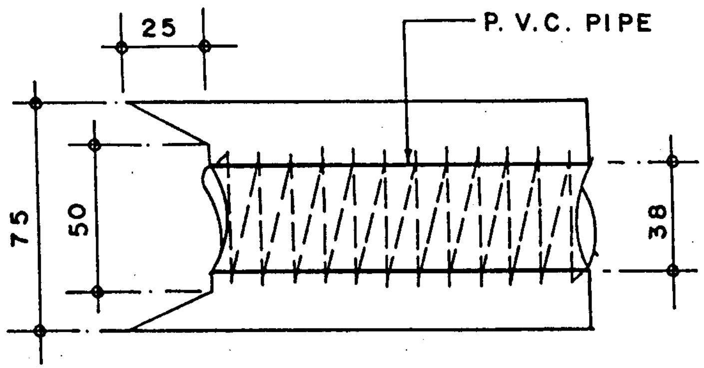
DPM-C3\lged\FORM3.DWG 1:120



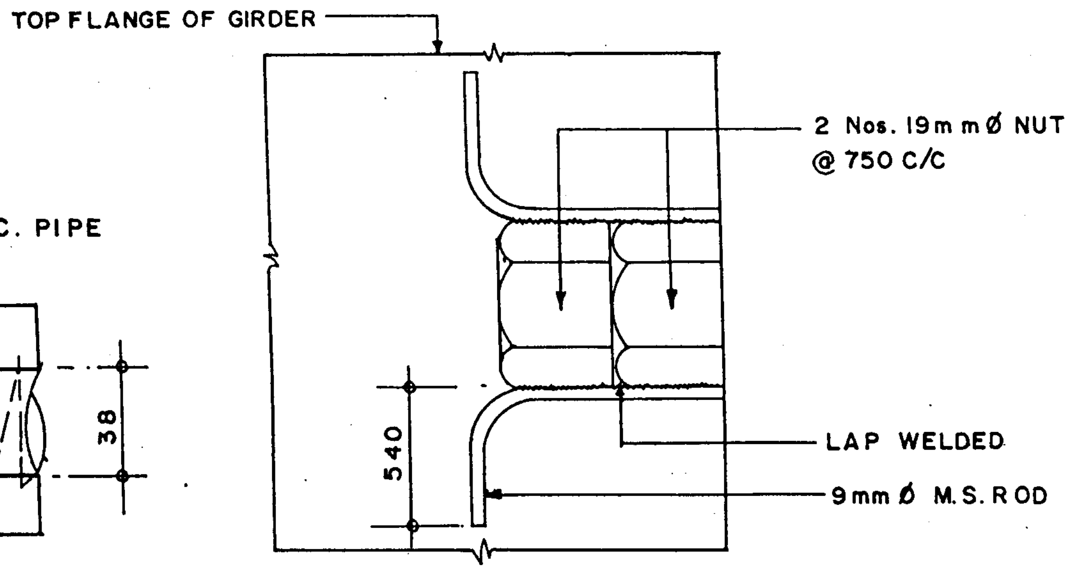
(a) TYPICAL HOLE DETAIL FOR
GIRDER SHIFTING (DETAIL-A)



(b) TYPICAL INSERT DETAIL FOR
DECK FORM SUPPORT (DETAIL-B)



SECTION ON. 1-1



SECTION ON. 2-2

TYPICAL DETAILS OF INSERTS IN TOP FLANGE FOR LIFTING AND
SHIFTING OF PC GIRDER AND DECK FORM SUPPORT

FIG. 14.6(b)

CHAPTER 15

CONSTRUCTION OF CAST-IN-SITU BORED PILES

15.1 General

This work shall consist of constructing cast-in-place reinforced concrete pile for bridge foundation as per drawing and specification and to provide all labour, materials, equipment including boring equipment and incidentals necessary to complete the work as per direction of the Engineer-in-charge.

15.2 Material and Work Specification

15.2.1 Concrete

The concrete for cast-in-place pile shall be manufactured as per specification with the minimum proportion of 1:1½:3 having minimum cylinder concrete strength of 20 N/mm^2 at 28 days. The concrete shall be placed by using a tremie pipe which shall be sufficiently long to reach the bottom level of pile and gradually withdrawn as the placing of concrete proceeds.

15.2.2 Casing Pipe

Temporary steel casing pipe of required diameter shall be used at least for the upper 3m from ground level during drilling to stabilise the hole. Permanent steel casing pipe if provided in the schedule of items of work, shall be of the required diameter, length and thickness and shall be lowered upto the designed depth immediately after the drilling of hole is completed. At the option of the contractor the permanent casing pipe, may be lowered simultaneously with the progress of drilling and the use of temporary casing pipe may be eliminated. Casing pipes may be transported to site at suitable lengths of pieces and shall be welded as per specification to fabricate the design length. The permanent steel casing pipes shall be free from rust, pits or any other deformity and the inside of the pipe shall be free from paint, grease or any other deleterious substance that may affect the concrete.

15.2.3 Reinforcement

The reinforcement for cast-in-place concrete piles shall meet the requirement of the specification for reinforcing steel for structures. All turns of spirals shall be tack welded at 3 places and lapping of vertical reinforcement shall be allowed if shown on the drawings.

15.2.4 Welding

a) Electrodes

All arc welding electrodes shall conform to AWS standards and the electrodes shall be rods of size and classification number as recommended by their manufacturers.

b) Quality

Welding of reinforcing bars shall be performed by skilled and experienced welder and the connections shall be made in accordance with AWS.

15.3 Method of Construction

a) Preparation

Before starting drilling operation the contractor shall establish levels, grades and alignment of all piles with reference to bench marks (BM) previously established at site. The Contractor shall have all casing and ready for reinforcing bars fabricated as per design and ready for lowering after the completion of drilling. All necessary equipment such as pump, welding set etc. and materials for concrete work including tremie pipe shall be made available before the start of drilling operation.

b) Drilling

The drilling method and equipment to be used shall be approved by the Engineer. The Contractor shall prepare suitable cofferdam/artificial island/staging or any other approved means, if required, for the drilling operation and concreting the piles and piers in water. Bentonite slurry, if required, shall be used to stabilise the hole.

c) Pile Cluster

Where there are more than (4) four piles in a cluster, the centre pile shall be installed first. All piles in a cluster shall be of the same depth.

d) Tolerance for Drilling Holes

Bore holes shall be accurately drilled in the locations shown in the contract drawing. All piles shall be drilled with a lateral tolerance of not more than 75 mm from the point specified. Piles that deviate more than 75 mm in lateral location or piles whose slope deviates from the vertical by more than 2 (two) percent shall be rejected. Additional piles shall then be furnished and installed by the contractor in such locations as the Engineer may direct. All costs of such additional piles required to suit changed pile locations shall be borne by the contractor.

e) Obstruction During Drilling

When obstructions make it extremely difficult to drill certain boreholes in the locations shown on the drawings and to the proper bearing strata, the contractor shall adopt all necessary measures to install piles as required including jetting, as approved by the Engineer. If the contractor is unable to complete properly any pile restoring to such technically sound methods, the Engineer may order additional hole(s) drilled at another location(s) at the Contractor's expense.

f) Depth of Hole

The depth of borehole shall be checked by the Engineer-in-charge by lowering suitable drop to determine the length of pile. Immediately after approval of the bore hole, the steel casing pipe shall be provided upto the design depth, if shown in the Schedule of Items of Works, and then the reinforcement cage shall be lowered.

15.4 Measurement and Payment

The cast-in-place bored pile of the required size shall be paid as per length of pile cast measured in linear m from the bottom of the pile to the cut-off level and paid at the contract unit price shown in the Schedule of Items of Works and shall be the full compensation for providing all equipment and incidentals necessary to complete the work, except reinforcing steel which shall be paid separately. Permanent steel casing pipe, cofferdam/crossdam/ring bundh/ artificial island/staging in water shall be paid separately if shown in the drawing and Schedule of Items of Works.

No payments shall be made for temporary casing pipe or bentonite slurry used for stabilizing the drilled hole.

15.5 Load Test on Piles

i) This item shall consist of the application of a test load on a pile selected by the Engineer-in-charge to determine the potential bearing capacity and adequacy of the pile by observation of its settlement behaviour under test load.

ii) The Contractor shall submit to the Engineer-in-charge for approval of all plans and procedures for the load tests including platform, weights or jacks, gauges, set-up for surveying and loading and unloading sequence. The Contractor shall furnish dependable gauges and drives (sensitivity 0.025 mm) as approved by the Engineer-in-charge for measuring the settlement and shall furnish the Engineer-in-charge with a recent verification of the gauge calibration by a reliable agency. The contractor shall also furnish the Engineer-in-charge with adequate facilities for making load and settlement readings 24 hours per day except such engineering instruments and apparatus as are included in an engineer's regular equipment, e.g., surveying equipment, etc..

iii) The test load shall be concentrically applied by such a method that the test load acting on the pile at any time may be definitely determined and controlled.

iv) The load shall be applied to the pile as near to the ground surface as possible. The test loading will not be considered satisfactory if the pile fails internally during the test loading due to improper installation or procedure by the Contractor.

v) The head of the pile shall be cut off, levelled and capped with a steel plate in such a manner as to produce a horizontal plane of bearing surface. The total test load shall be twice the anticipated working load on the pile and shall be applied in increments amounting to 25, 50,75, 100, 125, 150, 175 and 200 per cents of the anticipated working load. The first application of the load shall be made after the curing period is over and concrete achieves the required strength but not earlier than 72 hours after the driving of the pile to be tested. Increment of load shall be applied not earlier than 2 hours after all measurable settlement (0.25 mm or less in 20 minutes) caused by the previous loading. A careful reading of settlement shall be made immediately before and after the application of such increment of loads and at the intermediate interval of 20 minutes apart. On completion of full test loading, the load shall remain on the pile 24 hours or more if the necessity for that is indicated by the rate of the settlement of the pile. The settlement reading shall be taken during and at the end of the period.

vi) The pile shall be unloaded in stipulated decrements as directed by the Engineer-in-charge. During unloading of the pile, the rebound shall be measured when the load remaining on the pile amounts to 75, 25,

10 and 0 per cent of the full test load at not more than half hour intervals with measurements of the rebound being immediately before and after each decrement. The final rebound shall be recorded 24 hours after the entire test load has been removed.

vii) A check for the accuracy of the settlement shall be made from a fixed reference point to determine adequacy of the whole settlement attachment. Any pile or stake serving or fixed reference shall be driven at a distance of 2.44 m or more from the nearest point of the test pile. If it is necessary to remove and reapply the load, it shall be reapplied gradually and not suddenly.

CHAPTER 16

SPECIFICATIONS AND INSTALLATION OF ELASTOMERIC BEARING AND EXPANSION JOINT

16.1 Elastomeric Bearing

16.1.1 General

An elastomeric bridge bearing is a device constructed partially or wholly from elastomer. The purpose of this is to transmit loads and accommodate movements between a bridge superstructure and its supporting structure. The bearings may be plain pads (consisting of elastomer only) and reinforced bearings (consisting of alternative layers of steel or fabric reinforcement and elastomer, bonded together). Tapered elastomer layers in reinforced bearings are not permitted.

Bearings should be furnished with dimensions, material properties, elastomer grade and type of laminates required by the drawings. These should conform to minimum 50±5 Shore A hardness in accordance with ASTM Designation : D 2240.

16.1.2 Properties of the Elastomer

The raw elastomer shall be either virgin Neoprene (polychloroprene) or virgin natural rubber (polyisoprene). The elastomer compound shall be classified as being of low temperature grade 0. The testing requirements of the materials should conform to AASHTO standard specifications for Highway Bridges, 1992, Div. I-A, Table 18.2.3.1A for neoprene quality control tests and Table 18.2.3.1B for natural rubber quality control tests.

16.1.3 Steel Laminates

Steel laminates used for reinforcement shall be made from rolled mild steel conforming to ASTM A36, A570 or equivalent unless otherwise specified by the Engineer-in-charge. The laminates shall have a minimum thickness as shown on the drawings. Holes in plates for manufacturing purposes will not be permitted unless they have been accounted for in the design and shown on the drawing.

16.1.4 Bond

The vulcanised bond between elastomer and reinforcement shall have a minimum peel strength of 5.2 kN/m(30 lb/in.). Steel laminated bearings shall develop a minimum peel strength of 6.9 kN/m (40 lb/in.). Peel strength test shall be performed in accordance with ASTM D 429 Method B.

6.1.5 Manufacture

Bearing with steel laminates shall be cast as a unit in a mould and shall be bounded and vulcanised under heat and pressure. The mould finish shall conform to standard shop practice. The internal steel laminates shall be sand blasted and cleaned of all surface coatings, rust, mill scale, and dirt before bonding, and shall be free from sharp edges and burrs. Bearings that are designed to act as a single unit with a given shape factor must be manufactured as a single unit.

16.1.6 Fabrication Tolerances

Plain pads and laminated bearings shall be built to the specified dimension within the following tolerances:

Overall Height

- Design Thickness 32mm (1¼ in.) or less	- 0, + 3mm
Design Thickness over 32mm	- 0, + 6mm
- Overall Horizontal Dimension 914mm (36 in.) or less	- 0, + 6mm
- Thickness of Individual Layers of Elastomer (Laminated Bearings only) At any point within the bearings	± 20% of design value but not more than ± 3mm
- Parallelism with opposite face Top and bottom sides	0.005 radian
	0.02 radian
- Edge Cover Embedded Laminates	- 0, + 3mm
- Thickness Top and bottom cover Layer	- 0, the smaller of + 1.5mm and + 20% of the nominal cover layer thickness.

16.1.7 Marking and Certifying

The manufacturer shall certify that each bearing satisfies the requirements of the drawings and these specifications, and shall supply a certified copy of material test results. Each reinforced bearing shall be marked with indelible ink or flexible paint the bearing identification number, elastomer type and grade number. The marking shall be on the face that is visible after erection of the bridge.

16.1.8 Testing

i) General

Materials for elastomeric bearings and the finished bearings themselves shall be subjected to the tests described in this section. Material tests shall be in accordance with the appropriate Table 18.2.3.1A or Table 18.2.3.1B of AASHTO Bridge Standards, 1992.

ii) **Ambient Temperature Tests on the Elastomer**

The elastomer used shall atleast satisfy the limits prescribed in the appropriate Table 18.2.3.1A or B for durometer hardness, tensile strength, ultimate elongation, heat resistance, compression set, and ozone resistance. The bond to the reinforcement, if any, shall also satisfy the requirement of Article 16.1.4 above. The shear modulus of the material shall be tested at 23°c (73°F) using the apparatus and procedure described in annex A of ASTM D4014. It shall be either equal to or greater than the value shown on the drawing.

iii) **Visual Inspection of the Finished Bearing**

Every finished bearing shall be inspected for compliance with dimensional tolerances and for overall quality of manufacture. In steel reinforced bearings, the edges of the steel shall be protected everywhere from corrosion.

iv) **Short Duration Compression Tests on Bearings**

The bearing shall be loaded in compression to 1.5 times the maximum design load. The load shall be held constant for 5 minutes, removed, and re-applied for another 5 minutes. The bearing shall be examined visually while under the second loading. If the bulging pattern suggests laminate parallelism or a layer thickness that is outside the specified tolerances, or poor laminate bond the bearing shall be rejected. If there are three or more separate surface cracks that are greater than 0.2mm (0.08 in.) wide and 0.2mm (0.08 in.) deep, the bearing shall be rejected.

v) **Long Duration Compression Tests on Bearings**

The bearing shall be loaded in compression to 1.5 times its maximum design load for a minimum period of 15 hours. If, during the test, the load falls below 1.3 times the maximum design load, the test duration shall be increased by the period of time for which the load is below this limit. The bearing shall be examined visually at the end of the test while it is still under load. If the bulging pattern suggests laminate parallelism or a layer thickness that is outside the specified tolerances, or poor laminate bond the bearing shall be rejected. If there are three or more separate surface cracks that are greater than 0.2mm (0.08 in.) wide and 0.2mm (0.08 in.) deep, the bearing shall be rejected.

vi) **Shear Modulus Tests on Bearing Material**

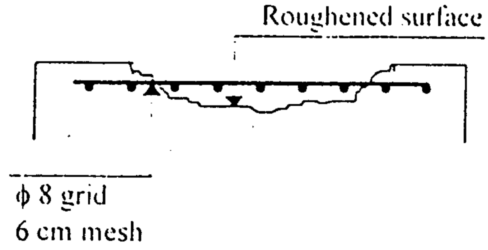
The shear modulus of the material in the finished bearing shall be evaluated by testing a specimen cut from it using the procedure given in annex A of ASTM D4014. If the test is conducted on finished bearings, the material shear modulus shall be computed from the measured shear stiffness of the bearing, taking due account of the influence on shear stiffness of bearing geometry and compressive load.

16.1.9 Installation

Bearings shall be placed on surfaces that are plane to within 1.5mm (1/16 in.) and horizontal to within 0.01 radian. Any lack of parallelism between the top of the bearing and the underside of the girder that exceeds 0.01 radian shall be corrected by grouting or as otherwise directed by the Engineer-in-charge.

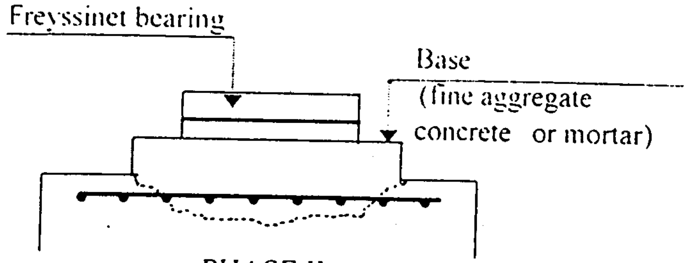
In order to function correctly the elastomeric bearings must be placed with particular care.

As a general rule, the bearings must be placed on a perfectly plane and horizontal surface. A 30 to 40 mm high mortar pad or for larger height the RC pad is cast onto the base concrete, which must be previously carefully roughened. The dimensions of the pad must be by 30 to 50 mm larger than those of the bearing.



PHASE I

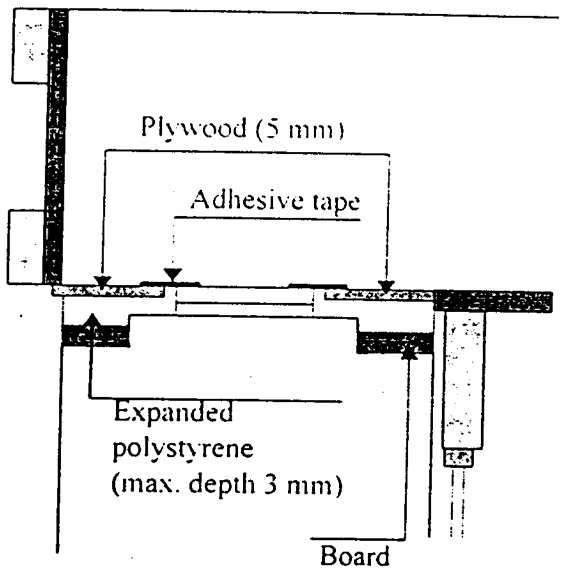
When minimum loads are small, it is recommended to glue the bearing to the support with an epoxy resin.



PHASE II

The bottom face of the precast unit, in contact with the bearing, must be perfectly plane and horizontal.

For cast-in-situ structures, the bearing is placed on the support and the structure cast on the bearing. The shuttering surrounding the bearing must be removable at the time of stripping the forms and at the same time it must be strong enough to resist the weight of concrete. If the base shuttering fails, the bearing will be partially embedded in the concrete and the distortion hindered. Future lifting of the structure may become more difficult. The sketch shows the standard details of the Freyssinet recommended shuttering around bearing.



PHASE III

16.2 Installation of Expansion Joint

The expansion joint shown on the drawing in Part-A consists of MS angle nosings connected to the concrete of the deck slab and back wall by MS anchor bars. The movable part consists of steel plate fixed by twin black bolts with one side nosing. The structural steel conforms to BS 449 St.43 or equivalent.

During concreting of the deck slab and back wall the nosing angles with the welded anchor bars and the threaded twin black bolts will be embedded in the green concrete exactly as shown on the drawing. A steel template will be used to maintain alignment and position of the black bolts. The steel movable plate will be fixed with the bolts by nuts. A Φ20 MS bar will then be welded at the edge of the other side nosing angle. The gap between the plate and the welded bar will then be filled by bituminous filler material.

CHAPTER 17

COMPUTER SOFTWARE AND USER'S GUIDE

17.1 General

The computer softwares have been developed for analysis and design of the various components of the standard structures of the present Manual which includes bridge superstructure, substructure and pile foundations. Each program has three components:

- Main execution program;
- Data File; and
- Output file

Main execution files are in machine language, so these cannot be read or altered. If somehow any changes occur, then the program may become unuseable. So execution programs should not be changed. The user can change the data files only following Art. 17.3 . The output files will be automatically generated by the program with the given names.

17.2 List of the Software

17.2.1 Analysis and Design

The names of the main programs for analysis and design of different components of a bridge are furnished below as :

for PC Girder	:	DPM-PCG.EXE
for Deck Slab	:	DPM-DECK.EXE
for Elastomeic Bearing	:	DPM-BRG.EXE
for Abutment-Wing Wall	:	DPM-ABUT.EXE
for Pile Cap	:	DPM-PCAP.EXE
for Pile Load & Design	:	DPM-LOAD.EXE
for Length of Pile	:	DPM-PILE.EXE

A brief outlilne of the software packages are given in the subsequent paragraphs.

PC Girder

The computer program for PC girder has been made following the design steps given in Art. 8.5, Flow Chart for Design of PC Girder. The general program and file names are DPM-PCG.EXE, DPM-PCG.DAT and DPM-PCG.OUT. For a particular span say, 30 m c/c brg. span girder the data and output file may be re-named as DPM-PC30.DAT and DPM-PC30.OUT respectively. The program calculates stresses for self weight of girder, prestressing forces, dead load of deck slab, cross girder, live load with its impact, creep modified differential shrinkage and all losses of prestress. It also calculates deflection and ultimate moment and shear capacity of the PC girder and checks adequacy of girder cross-section.

Deck Slab

The computer program for analysis and design of deck slab has been made following the design steps given in in Art. 9.4, Flow Chart for Design of Deck Slab. The program and file names are DPM-DECK.EXE, DPM-DECK.DAT and DPM-DECK.OUT. It calculates the maximum positive and negative bending moments at critical locations of deck slab and ultimate moment and shear capacity of the concrete section. It thereby checks the adequacy of slab thickness and reinforcement.

Elastomeic Bearing

The computer program for designing the reinforced elastomeric bearing has been developed following the design steps given in Art. 12.4, Flow Chart for Design of Reinforced Elastomeric Bearing. The general program and file names are DPM-BRG.EXE, DPM-BRG.DAT and DPM-BRG.OUT. For a particular bridge span say, 30 m c/c brg. span, the data and output files may be re-named as DPM-BRG30.DAT and DPM-BRG30.OUT respectively.

Abutment-Wing Wall

The computer program for analysis and design of abutment-wing wall and tie wall has been made following the design steps given in Art. 10.4, Flow Chart for Design of Abutment-Wing Wall. The general program and file names are DPM-ABUT.EXE, DPM-ABUT.DAT and DPM-ABUT.OUT.

RC Pile Cap

The computer program for analysis and design of abutment pile cap has been made following the design steps given in Art. 11.3, Flow Chart for Design of Pile Cap. The program and file names are DPM-PCAP.EXE, DPM-PCAP.DAT and DPM-PCAP.OUT . For a particular bridge of 8.0m high abutment and 'F' type foundation , the data and out files may be renamed as PCAP-8F.DAT AND PCAP-8F.OUT respectively.

Cast-in-Situ Bored Pile

The service load on each pile group has been calculated for all applicable AASHTO load groups. Similarly factored axial and horizontal / longitudinal loads have been calculated for all AASHTO load groups. The program has further incorporated the parameters for structural design of piles based on the critical load group which gives the factored minimum axial and maximum longitudinal load. The computer program has been developed following the design steps given in Art. 11.2, Flow Chart for Design of Cast-in-Situ Bored Pile. The file names are DPM-LOAD.EXE, DPM-LOAD.DAT and DPM-LOAD.OUT. For a particular abutment say, for a 40 m c/c brg. span PC girder, 10 m high abutment, foundation type 'F' comprising 20 number piles, the data and output files may be re-named as 40-10A20.DAT and 40-10A20.OUT respectively.The program calculates both service and factored vertical and longitudinal loads on each pile. DPM-PILE.EXE, DPM-PILE.DAT and DPM-PILE.OUT have been developed to calculate the pile length based on the subsoil types and properties in accordance with the flow chart given in Fig. 2.2 in Part-A of this Manual. The design includes both the shaft resistance and end bearing of piles. The program calculates the allowable bearing capacity of pile by dividing the ultimate capacity by the factor of safety 3.

17.2.2 Preparation of Drawings in AutoCAD

AutoLISP programs in AutoCAD environment have been developed for preparation of the drawings in Part A of this manual. The AutoLISP programs are as follows:

Program Name	Title of Drawing
DPM-GA.LSP	General Arrangement Drawing
DPM-DECK.LSP	Concrete Outline Details of Deck Slab
DPM-PC1.LSP	Concrete Outline Details of PC Girder
DPM-PC2.LSP	Prestressing Cable Details of PC Girder
DPM-ABUT.LSP	Concrete Outline Details of Abutment-Wing Wall and Pile Cap

17.2.3 Bar Bending Schedule

The bar bending schedules for the standard structures of the Part A of this Manual has been prepared by the spread sheet program made in LOTUS 1-2-3 . The comprehensive program gives the bar diameter, type, cut length of bars, total lengths and weights. The shape code of bars prepared by using AutoCAD have been given in separate drawing sheet (Part-A, Dwg. No.: PC-SHC-01). The bar bending schedules of non-prestressed reinforcement for different components of the bridges are furnished as follows :

PC-20-04 to PC-40-04	:	Bar Bending Schedules for PC Girder
SIA20 to SIIB40	:	Bar Bending Schedules for Deck Slab
P-6A-01 to P-10I-02	:	Bar Bending Schedules for Pile
C-SC-01 to C-SC-03	:	Bar Bending Schedules for Pile Cap
A-6A-01 to A-10I-06	:	Bar Bending Schedules for Abutmen-Wing Wall
RA-00-02	:	Bar Bending Schedule for Precast Railing and Rail Post

17.2.4 Bill of Quantities (BOQ)

The bill of quantities consisting of standard item description conforming to the LGED's standard rate schedule have been prepared by using Lotus 1-2-3 spread sheet. This is shown in Table 3.1 of Part A of this Manual. The following spread sheets have been prepared to comply with the requirements of the rate schedule :

SQ-SP-01	:	BOQ for Superstructure
SQ-SB-01 to SQ-SB-09	:	BOQ for Substructure
SQ-PF-01	:	BOQ for Pile Foundation

17.3 User's Guide

The design softwares are simple and easy to operate. But the data input must be done very carefully. The given data are so sensitive that even a small mistake can result in a wrong output. Moreover the decimel points should be put carefully. For the real values of data sheet decimel (.) must be given but for the integer values decimel must be avoided. Otherwise the program will show an error like this - 'Format does not match with input data'. Another important thing is the field width of the data. This width has been limited to 12 digits. If some real data become larger than this, exponential form should be used. Suppose a real value is 1524000000.00 which has a field width of 13 digits but the width is restricted upto

12 digits in program. So the above value can be written as 15.24E+08 in the exponential form which has a field width of 9 digits and is acceptable.

Any change in the sequence of input data will result in an useless output the program will show an error like this - ' Data error in input field'. For each structure the data file should be saved carefully and should be given individually distinct name so that it could be retrieved later easily for future work. The data and output file names should not be more than 8 characters and extension DAT or OUT should be used respectively. A dot (.) must be given in between the main name and extension. For convenience main names of data and out files should be same. As for example, the data and out file names may be used as
LOAD8F40.DAT AND LOAD8F.OUT respectively.

SAMPLE DATA FILE FOR DESIGN OF PC GIRDER

FILE : DPM-PCG.DAT
PROGRAM : DPM-PCG.EXE
PROJECT : MANUAL ON PC BRIDGES
CLIENT : LGED

```
**************************INPUT DATA IN kN &/ m*******************************
******************WRITE ALL DATA FROM COL.51 & LINE 10**********************
```

LENGTH OF GIRDER C/C BRG..L.m	=30.0
OVERHANG PORTION OF GIRDER. XBEAR.m	=0.40
DIST. OF TEST X-SEC. FROM C.L. OF BRG..X.m	=15.0
TYPE 'EG' FOR EXTERIOR & 'IG' FOR INTERIOR	=EG
LANE TYPE:WRITE 'SL'FOR SINGLE OR'DL'IN DOUBLE	=SL
NUMBER OF CROSS BEAM ,NCB	=3
X-GIRDER LOAD,FCB(I),(kN)INPUT 1/LINE NCB	=12.944
	=12.944
	=12.944
UNIT WT.OF CONCRETE,WCONC.kN/m^3	=24.00
UNIT WT. OF WEARING COURSE ,WWCONC.kN/m^3	=23.0
THICKNESS OF WEARING COURSE .HWC,m .	=0.04
DIST. OF EXTERIOR GIRDER FROM CURB EDGE.X1.m	=0.58
DIST. OF WHEEL FROM CURB EDGE(AASHTO).XLL.m	=0.61
LOAD FROM FOOT PATH,CURB,WFP.kN	=6.06
LOAD FROM RAILING,kN/m	=0.734
FOOT PATH LL(REF.AASHTO),FPLL.kN/m^2	=2.88
WIDTH OF FOOT PATH,BFP,m	=0.40
INITIAL PRESTRESSING FORCE,PF.kN	=555.0
MODULUS OF ELASTICITY OF GIRDER CONC..EBEAM,kN/m^2	=25906000.0
MODULUS OF ELASTICITY OF DECK CONC..EDECK,kN/m^2	=21152000.0
NUMBER OF MAIN GIRDERS,NG	=2
C/C DISTANCE BETWEEN GIRDERS, S,m	=2.50
DISTANCE OF EG EDGE FROM CURB,X2,m	=0.388
CONCRETE STRENTH ,FCI,kN/m^2	=30000.0
CREEP MODIFIED DIFF. SHRINKAGE STRAIN(NORMALLY 100E-06).ESH	=0.0001
DECK WIDTH,BDECK,m	=2.218
DECK THICKNESS,HDECK,m	=0.20
LONGER HOR.L.OF W-1 OF GIR.AT MID SPAN.m,B(1,1)	=0.400
SMALLER HOR.L.OF W-1 OF GIR.AT MID SPAN.m,D(1,1)	=0.400
HEIGHT OF W-1 OF GIRDER AT MID SPAN,m.H(1,1)	=0.450
FOR WEDGE-2 .B(2.1)	=0.090
FOR WEDGE-2 .D(2.1)	=0
FOR WEDGE-2 .H(2.1)	=0.045
FOR WEDGE-3 .B(3.1)	=0.090
FOR WEDGE-3 .D(3.1)	=0
FOR WEDGE-3 .H(3.1)	=0.045
FOR WEDGE-4 .B(4.1)	=0.220
FOR WEDGE-4 .D(4.1)	=0.220
FOR WEDGE-4 .H(4.1)	=1.50
FOR WEDGE-5 .B(5.1)	=0.215

FOR WEDGE-5 ,D((5,1)	=0
FOR WEDGE-5 ,H(5,1)	=0.215
FOR WEDGE-6 ,B(6,1)	=0.215
FOR WEDGE-6 ,D(6,1)	=0
FOR WEDGE-6 ,H(6,1)	=0.215
FOR WEDGE-7 ,B(7,1)	=0.650
FOR WEDGE-7 ,D(7,1)	=0.650
FOR WEDGE-7 ,H(7,1)	=0.250
FOR WEDGE-1 AT END SPAN.B(1,2).m	=0
FOR WEDGE-1 AT END SPAN.D(1,2).m	=0
FOR WEDGE-1 AT END SPAN.H(1,2).m	=0
FOR WEDGE-2 B(2,2)	=0
FOR WEDGE-2 D(2,2)	=0
FOR WEDGE-2 H(2,2)	=0
FOR WEDGE-3 B(3,2)	=0
FOR WEDGE-3 D(3,2)	=0
FOR WEDGE-3 H(3,2)	=0
FOR WEDGE-4 B(4,2)	=0.400
FOR WEDGE-4 D(4,2)	=0.400
FOR WEDGE-4 H(4,2)	=1.95
FOR WEDGE-5 B(5,2)	=0.225
FOR WEDGE-5 D(5,2)	=0
FOR WEDGE-5 H(5,2)	=0.225
FOR WEDGE-6 B(6,2)	=0.225
FOR WEDGE-6 D(6,2)	=0
FOR WEDGE-6 H(6,2)	=0.225
FOR WEDGE-7 B(7,2)	=0.650
FOR WEDGE-7 D(7,2)	=0.650
FOR WEDGE-7 H(7,2)	=0.250
TOTAL DEPTH OF GIRDER,HTOTAL,m	=2.20
YIELD STRESS.FY,kN/m^2	=1377000.0
NUMBER OF STAGES OF PRESTRESSING.STAGE	=1
NO.OF CABLES IN STAGE-I,N(I)	=9
ORD.OF CABLE-J IN STAGEI AT MID SPAN COMID(I,J)	=0.075
ORD.OF CABLE-J IN STAGEI AT END SPAN COEND(I,J)	=0.20
	=0.075
	=0.20
	=0.075
	=1.00
	=0.075
	=0.20
	=0.075
	=0.20
	=0.175
	=1.70
	=0.175
	=2.05
	=0.175
	=2.05
	=0.175
	=1.35
	=0.3
CURVATURE EFFECT FACTOR./rad	=0.175
LENGTH EFFECT FACTOR.KL.,'m	=0.007

WEDGE PULL-IN OF ANCHORAGE, DELTA,m	=0.008
MODULUS OF ELASTICITY OF STEEL ,ES,kN/m^2	=205000000.0
AREA OF PRESTRESSING STEEL,APS,m2	=0.000462
MODULUS OF ELASTICITY OF CON.DURING STRESING,ECI,kN/m^2	=21900000.0
RELATIVE HUMIDITY IN %	=60.0
ULTIMATE STRENGTH OF PRESTRESSING STEEL,FSP,kN/m^2	=1620000.0
DIAMETER OF TENSION REINFORCEMENT,DTR,m	=0.01
TOTAL NO. OF TENSION BAR,NTB	=2
YIELD STRENGTH OF NON-PRESTRESSED STEEL,FSY,kN/m^2	=275000.0
DIAMETER OF WEB REINFORCEMENT,DWR,m	=.01
TOTAL NO. OF STIRRUP LEGS	=2
LENGTH OF SPLAYED PORTION,SPL,m	=1.200

SAMPLE DATA FILE FOR DESIGN OF DECK SLAB

FILE : DPM-DECK.DAT

PROGRAM : DPM-DECK.EXE

PROJECT : MANUAL ON PC BRIDGES

CLIENT : LGED

DECK TYPE: IA

*********************ALL INPUT DATA ARE IN 'kN' &/ 'm'****************************

*****************DATA SHOULD BE GIVEN FROM LINE 20 & COLUMN 45*******************

THICKNESS OF DECK SLAB,HDECK, m	0.20
THICKNESS OF WEARING COURSE,HWC, m	0.04
WIDTH OF SIDEWALK,WSW, m	0.40
C/C DISTANCE OF GIRDER,SPAN, m	2.50
FOOTPATH LIVE LOAD,FLL, kN/m^2	2.88
CONCRETE STRENGTH,FCCONC, kN/m^2	20000.0
REINF. STRENGTH,FYSTEL, kN/m^2	275000.0
LIVE LOAD, P18, kN	71.172
GAMMA (FACTOR)	1.3
BETAD	1.0
BETAL	1.0
S1	0.38
LOAD OF RAIL, kN/m	0.734
LEVER ARM FOR RAIL & RAIL POST,LA1, m	1.155
LOAD OF RAIL BASE, kN/m	1.56
LEVER ARM FOR RAIL BASE,LA2, m	1.193
LOAD OF FOOTPATH, kN/m	6.48
LEVER ARM FOR FOOTPATH,LA3, m	0.937
DIAMETER OF POSITIVE MAIN REINF., DMP, m	0.016
SPACING OF POSITIVE MAIN REINF., SMP, m	0.135
DIAMETER OF NEGATIVE MAIN REINF., DMN, m	0.016
SPACING OF NEGATIVE MAIN REINF., SMN, m	0.090
DIA. OF DISTRI. REINF.ON BOTT.LAYER, DD, m	0.012
DIA. OF TEMP. REINF ON TOP LAYER, DT, m	0.012

SAMPLE DATA FILE FOR DESIGN OF ELASTOMERIC BEARING

FILE : DPM-BRG.DAT
PROGRAM : DPM-BRG.EXE
PROJECT : MANUAL ON PC BRIDGES
CLIENT : LGED

*****************ALL VALUES MUST BE WRITTEN FROM COL-50 & LINE-10**********************
******************************LIST OF INPUT DATA***

BRIDGE SPAN IN ,SPAN ,mm	=20000.0
GROSS LENGTH OF BEARING ,L ,mm	=250.0
GROSS WIDTH OF BEARING ,W ,mm	=400.0
THICKNESS OF INNER ELASTOMER LAYER ,HRI ,mm	=10.0
THICKNESS OF OUTER ELASTOMER LAYER,HO ,mm	=5.0
THICKNESS OF STEEL REINFORCEMENT LAYER,HS, mm	=3.0
CLEAR COVER ,COVER, mm	=6.0
NO. OF INTERNAL LAYERS OF ELASTOMER.NI (INTEGER)	=2
NO. OF STEEL LAYERS ,NS (INTEGER)	=3
SHEAR MODULUS OF ELASTOMER AT 73deg F ,G, N/mm^2	=0.8
DEAD LOAD REACTION ON BEARING ,VDL. N	=365408.0
LL REACTION ON BEARING ,VLL (WITHOUT IMPACT),N	=25639.34
MODIFYING FACTOR ,BETA	=1.0
COEFFICIENT OF TEMPERATURE, ALPHAT.PER deg C	=0.000012
TEMPERATURE ,T ,deg C	=35.0
SHRINKAGE STRAIN , ESH	=0.0003
CONSTANT DEPENDENT ON ELASTOMER HARDNESS ,KBAR	=0.75
DEFLECTION OF BRIDGE AT MID SPAN,DEFLEC,mm	=6.12
ALLOWABLE STRESS OF STEEL ,FS, N/mm^2	=100.0
NO. OF LANES,NLANE (INTEGER)	=1
NO. OF GIRDERS ,NG (INTEGER)	=2
TOTAL ROTATION AT SUPPORT,rad	=0.00343

SAMPLE DATA FILE FOR DESIGN OF ABUTMENT-WING WALL

FILE : DPM-ABUT DAT
PROGRAM : DPM-ABUT.EXE
PROJECT : MANUAL ON PC BRIDGES
CLIENT : LGED

*****************DATA MUST BE WRITTEN FROM COLUMN-50 & LINE-10****************
***************************ALL DATA MUST BE IN kN &/ m****************************
************************************COMMON BLOCK**********************************

HEIGHT OF REGION-1.Z1. m	=0.75
IF THERE IS ANY TIE WAL WRITE Y & IF NOT WRITE N	=Y
TRANSVERSE SPAN OF ABUTMENT.LXABUT.m	=5.28
THICKNESS OF BASE SLAB.ZBASE. m	=0.8
TOTAL HEIGHT OF ABUTMENT FROM FDN.ZTOTAL. m	=8.0
TOE LENGTH FROM ABT.WALL(W/F).B3. m	=1.50
DIS. OF ABUT. SEAT FROM BACKWALL.B4. m	=0.725
THICKNESS OF ABUTMENT WALL.H1. m	=0.6
THICKNESS OF BACKWALL.H2. m	=0.4
THICKNESS OF TIE WALL.H3. m	=0.25
THICKNESS OF WING WALL AT EDGE.H4. m	=0.3
THICKNESS OF W/W AT TIE WALL. H5. m	=0.3
THICKNESS OF W/W AT ABUT. JN..H6. m	=0.6
HEIGHT OF SURCHARGE.HSUR. m	=0.61
UNIT WT. OF WATER .WTW. kN/m^3	=10.0
UNIT WT. OF SOIL.WTSAT. kN/m^3	=19.00
COEFFICIENT OF ACTIVE EARTH PRESSURE. Ka	=0.333
YOUNG'S MODULUS OF CONCRETE.EC. kN/m^2	=19560000.0
DEPTH OF WATER TABLE.ZW. m	=0.75
YIELD STRENGTH OF REINF. STEEL.FY. kN/m^2	=276000.0
ALLOWABLE STEEL STRESS.FALLOW. kN/m^2	=100000.0
28 DAYS CYLINDER STRENGTH OF CONCRETE.FCI. kN/m^2	=17000.0
CLEAR COVER TO REINFORCEMENT. m	=0.06

***************************BLOCK-1***
USE THIS BLOCK ONLY FOR ABUTMENT WITH TIE WALL
****

DIS.OF TOP OF TIE WALL FROM TOP OF R-1.Z5. m	=0.25
DIS.OF BOTTOM OF T.WALL FROM TOP OF BASE SLAB.Z6	=0.25
NO. OF TIE WALL FOR WING WALL (INTEGER VALUE)	=2
WIDTH OF WING WALL FLG.PORTION.B2. m	=2.7
MAX.WIDTH OF TIE WALL AT BASE.BCOUNT. m	=5.28
HOR. BAR DIA FOR R-1 OVER TIE WALL .DOCR1. m	=0.012
PROVIDED -VE HOR STEEL FOR R-1 OVER T.WALL.AOCR1. m^2/m	=0.001130
-VE HOR.STEEL SPACING FOR R-1 OVER T WALL.SOCR1	=0.1
HOR.BARDIA FOR REGION-1. DHOR1. m	0.012
PROVIDED +VE HOR.STEEL FOR REGION-1.AHP1	0.000753
PROVIDED +VE HOR.STEEL SPACING FOR REGION-1.SHP1	0.15
PROVIDED -VE HOR.STEEL FOR REGION-1.AHN1	0.001130
PROVIDED -VE HOR.STEEL SPACING FOR REGION-1.SHN1	0.1
HOR.BAR DIA FOR REGION-2. DHOR2. m	0.012
PROVIDED -VE HOR.STEEL FOR REGION-2.AHN2	0.001130
PROVIDED -VE HOR.STEEL SPACING FOR REGION-2.SHN2	0.1
HOR.BARDIA FOR REGION-2.DHOR3. m	0.012
PROVIDED +VE HOR.STEEL FOR REGION-3.AHP3	0.000753

PROVIDED +VE HOR.STEEL SPACING FOR REGION-3,SHP3	0.15
PROVIDED -VE HOR.STEEL FOR REGION-3,AHN3	0.000753
PROVIDED -VE HOR.STEEL SPACING FOR REGION-3,SHN3	0.15
PROVIDED +VE VER.STEEL FOR REGION-3,AVP3	0.000753
PROVIDED +VE VER.STEEL SPACING FOR REGION-3,SVP3	0.15
PROVIDED -VE VER.STEEL FOR REGION-3,AVN3	0.001130
PROVIDED -VE VER.STEEL SPACING FOR REGION-3,SVN3	0.1
PROVIDED VER.STEEL FOR TIE WALL ,ACOUNT	0.000753
PROVIDED VER.STEEL SPACING FOR TIE WALL ,SCOUNT	0.15

-----------------------END OF BLOCK-1-----------------------------

***********************BLOCK-2***

USE THIS BLOCK ONLY FOR ABUTMENT WITH NO TIE WALL.

****

BAR DIA FOR R-1,DHOR1,m

PROVIDED -VE HOR.STEEL AREA FOR R-1,AHN1, m^2

PROVIDED -VE HOR.STEEL SPACING FOR R-1,SHN1, m

HOR.BAR DIA FOR R-2,DHOR2, m

PROVIDED -VE HOR.STEEL AREA FOR R-2,AHN2, m^2

PROVIDED -VE HOR.STEEL SPACING FOR R-2,SHN2, m

-----------------------END OF BLOCK-2-----------------------------

***********************COMMON BLOCK***

FOLLOWING DATA ARE REQUIRED FOR ABUTMENT WALL.

****

HOR.BARDIA FOR ABUTMENT, DHORA, m	0.012
PROVIDED -VE HOR.STEEL FOR ABT.WALL,AHNA	0.00113
PROVIDED -VE HOR.STEEL SPACING FOR ABT.WALL,SHNA	0.1
PROVIDED +VE HOR.STEEL FOR ABT.WALL,AHPA	0.000753
PROVIDED +VE HOR.STEEL SPACING FOR ABT.WALL,SHPA	0.15
PROVIDED -VE VER.STEEL FOR ABT.WALL,AVNA	0.001130
PROVIDED -VE VER.STEEL SPACING FOR ABT.WALL,SVNA	0.1
PROVIDED +VE VER.STEEL FOR ABT.WALL,AVPA	0.000753
PROVIDED +VE VER.STEEL SPACING FOR ABT.WALL,SVPA	0.15

SAMPLE DATA FILE FOR PILE LOAD CALCULATION AND PILE DESIGN

FILE : DPM>30-8A20.DAT
PROGRAM : DPM-LOAD.EXE
PROJECT : MANUAL ON PC BRIDGES
CLIENT : LGED

NO.OF PILES=20 WIDTH OF BRIDGE=5.88 m

*******************WRITE DATA FROM LINE 15 & COL 51**********************
*******************DON'T USE DECIMEL FOR INT.VALUE***********************

NO. OF WEDGE , N (INT)	13
TOTAL NO. OF GIRDERS, NGIRD (INT)	2
DL REACTION FROM EACH GIRDER ON ABUT., kN	618.7
LL REACTION FROM EACH GIRDER ON ABUT., kN	110.0
LEVER ARM OF GIRDER REACTION FROM TOE,LADL, m	2.0
DISTANCE OF TOE FROM ABUT. FACE. DIS. m	1.70
COEFF. OF EARTH PRESSURE, Ka	0.33
EL. OF ABUTMENT TOP, m	8.0
EL. OF ABUTMENT BOTTOM, m	0.0
HIGH FLOOD LEVEL, HFL, m	4.3
TOTAL LENGTH OF GIRDER,LGIRD, m	30.8
GIRDER LENGTH BETWEEN BEARING C/C. m	30.0
DEPTH OF GIRDER, DGIRD, m	2.20
DEPTH OF SLAB, DSLAB, m	0.20
DEPTH OF KERB, DKERB, m	0.25
WIDTH OF BRIDGE, WBRDG, m	5.88
WIDTH OF FOOTPATH, BFP, m	0.80
HEIGHT OF ABUTMENT, HABUT, m	8.0
DEPTH OF SCOUR FROM PILECAP BOTTOM,DS, m	1.0
HEIGHT OF SURCHARGE, HSUR, m	0.61
BASIC WIND SPEED, VBWIND, m/s	66.67
AREA OF BEARING, ABRG, m^2	0.100
ELASTOMER STIFFNESS FACTOR, G, kN/m^2	800.0
ELASTOMER THICKNESS FACTORT, TQ, m	0.04
NO. OF TIE WALL/COUNTERFORT, NCF (INT)	1
TOTAL NO. OF RAIL POSTS, NPOST (INT)	42
N0. OF RAIL BAR, NRAIL (INT)	2
NO. OF LANE, NLANE (INT)	1
DEPTH OF RAIL, DRAIL, m	0.100
WIDTH OF RAIL, BRAIL, m	0.100
HEIGHT OF RAIL POST, HPOST, m	0.80
WIDTH OF RAIL POST, WPOST, m	0.25
BRAKING FORCE FACTOR, DF	1.0
SHRINKAGE COEFFICIENT, ESH, per deg C	520.0E-06
TEMP. COEFFICIENT, ALPHA, per deg C	12.0E-06
DESIGN TEMPERATURE, TEMP, CELCIUS	35.0
K1	1.0
STREAM VELOCITY, VSTRM, m/s	1.5

BETA IN DEGREE	45.0
EARTHQUAKE COEFF., KQ	0.06
WIND LOAD FACTOR, S1	1.0
WIND LOAD FACTOR, S2	1.0
WIND LOAD FACTOR, S3	1.0
UNIT WT. OF CONCRETE, WTCONC, kN/m^3	24.0
UNIT WT. OF MOIST SOIL, WTSOIL, kN/m^3	19.0
DIA. OF PILE, PDIA, m	0.6
CLEAR COVER OF PILE, CCOVER, m	0.075
DIA. OF MAIN BAR, BARDIA, m	0.025
DIA. OF STIRRUP, TIEDIA, m	0.01
MODULUS OF ELASTICITY, EMOD, kN/m^2	21.152E+06
COEFF.OF SUBGRADE MODULUS, NH, kN/m^3	1500.00
NRX (NO.OF PILE ROW PARALLEL TO X-AXIS), (INT)	4
NRY (NO.OF PILE ROW PARALLEL TO Y-AXIS), (INT)	5
NVPX(1) [NO.OF PILE IN ROW1 PARL.TO X-AXIS], (INT)	5
NVPX(2)	5
NVPX(3)	5
NVPX(4)	5
NVPY(1) [NO.OF PILE IN ROW1 PARL.TO Y-AXIS](INT)	4
NVPY(2)	4
NVPY(3)	4
NVPY(4)	4
NVPY(5)	4
DPY(1) [DIST.OF ROW 1 ALONG Y-AXIS, m]	0.6
DPY(2)	2.4
DPY(3)	4.2
DPY(4)	6.0
DPX(1) [DIST.OF ROW 1 ALONG X-AXIS, m]	0.6
DPX(2)	2.4
DPX(3)	4.2
DPX(4)	6.0
DPX(5)	7.8
H(1) [HEIGHT OF WEDGE 1, m]	3.72
H(2)	0.900
H(3)	2.58
H(4) [AVG. HEIGHT OF WEDGE 4, m]	0.0
H(5)	0.0
H(6)	7.2
H(7) [HEIGHT OF FLAG WALL END, m]	1.00
H(8)	0.8
H(9)	7.2
H(10)	0.25
H(11)	6.95
H(12)	0.25
H(13) [HEIGHT OF TRIANGULAR WEDGE, m]	0.35
T(1) [THICKNESS OF ABUTMENT WEDGE 1, m]	0.6
T(2)	1.125
T(3)	0.4
T(4)	0.0
T(5)	0.0
T(6)	4.5
T(7)	3.75
T(8)	7.05
T(9)	4.75
T(10)	0.30
T(11)	0.25

T(12)	8.375
T(13)	0.525
L(1) [LENGTH OF ABUTMENT WEDGE 1, m]	5.88
L(2)	5.88
L(3)	5.88
L(4)	0.0
L(5)	0.0
L(6)	0.45
L(7)	0.3
L(8)	8.4
L(9)	5.88
L(10)	0.15
L(11)	5.88
L(12)	0.25
L(13)	5.88

SAMPLE DATA FILE FOR DESIGN OF PILE CAP

FILE : PCAP-8F.DAT
PROGRAM : DPM-PCAP.EXE
PROJECT : MANUAL ON PC BRIDGES
CLIENT : LGED

ABUT. HEIGHT : 8.0 m FOUNDATION TYPE : F GIRDER LENGTH : 40.8 m

******************************LIST OF INPUT DATA(IN 'kN' &/OR 'm')*******************
****************************WRITE ALL DATA FROM LINE 15 & COL.51*****************

PILE ARRANGEMENT- S FOR STAGGERED; R FOR RECTANGULAR	R
UNIT WT. OF CONCRETE, WTCONC, kN/m^3	24.0
UNIT WT. OF SOIL, WTSOIL, kN/m^3	19.0
DISTANCE OF TOE FROM ABUT. FACE, DIS, m	1.70
COEFF. OF EARTH PRESSURE, Ka	0.33
HT. OF SURCHARGE, HSUR, m	0.61
LENGTH OF ABUTMENT, LABUT, m	5.88
HEIGHT OF ABUTMENT, HABUT, m	8.00
THICKNESS OF ABUTMENT WALL,TABUT, m	0.60
LENGTH OF PILE CAP, LPCAP, m	8.40
WIDTH OF PILE CAP, WPCAP, m	7.05
THICKNESS OF PILE CAP, TCAP, m	0.80
BASE WIDTH OF WING WALL(AVG.), WWING, m	0.50
WIDTH OF TIE WALL OR COUNTERFORT, T11, m	0.25
DIA. OF PILE, PDIA, m	0.60
CONCRETE STRENGTH, FCCONC. kN/m^2	20000.0
REINF. STRENGTH, FYSTEL, kN/m^2	275000.0
NRX(NO.OF PILE ROW PARL.TO ABUT./X-X)	4
NRY (NO.OF PILE ROW PARALLEL TO WINGWALL/Y-Y)	5
PR(1)(PILE REACTION OF ROW1 PARL.TO ABUT. X-X)	679.0
PR(2)	636.0
PR(3)	593.0
PR(4)	549.0
NVPX(1)[NO.OF PILE IN ROW1 PARL.TO X-X]	5
NVPX(2)	5
NVPX(3)	5
NVPX(4)	5
NVPY(1)[NO.OF PILE IN ROW1 PARL.TO Y-Y/WINGWALL] 4 NVPY(2)	4
NVPY(3)	4
NVPY(4)	4
NVPY(5)	4
DPY(1) [DIST.OF ROW 1 ALONG Y-AXIS]	0.60
DPY(2)	2.40
DPY(3)	4.20
DPY(4)	6.0
DPX(1) [DIST.OF COLUMN 1 ALONG X-AXIS]	0.60
DPX(2)	2.40
DPX(3)	4.20
DPX(4)	6.00
DPX (5)	7.80

SAMPLE DATA FILE FOR CHECKING LENGTH OF PILE

FILE	: DPM-PILE.DAT
PROGRAM	: DPM-PILE.EXE
PROJECT	: MANUAL ON PC BRIDGES
CLIENT	: LGED

ASSUMED TOTAL LENGTH OF PILE= 15 m

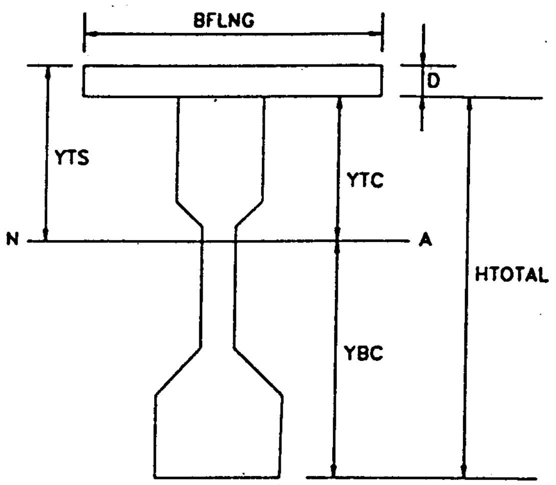
ALL SPT VALUES GIVEN HERE ARE FIELD RECORDED VALUES. FOR 'NC' SOIL AT PILETIP, L=LTOT-DSCOUR ; FOR 'C' SOIL AT PILETIP, L=LTOT-DS-1*PDIA
NUMBER OF LAYERS= L/1.5 = LOWER INTEGER

WRITE C FOR COHESIVE SOIL & WRITE NC FOR NON-COHESIVE SOIL .

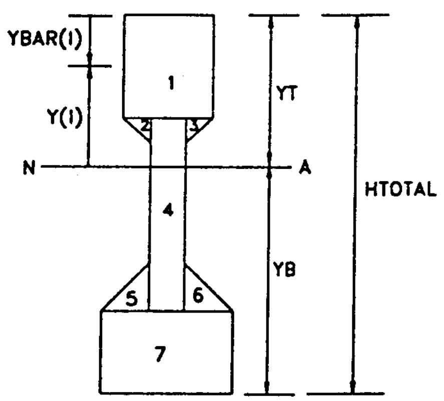
********************ALL VALUES MUST BE WRITTEN FROM COLUMN-51 & LINE-25*********

********************************LIST OF ALL INPUT DATA********************************

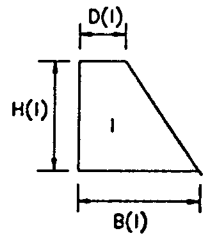
TOTAL LENGTH OF PILE.LTOT, m	=15.0
DEPTH OF SCOUR BELOW PILE CAP.DSCOUR.m	=3.4
DESIGN PILE LOAD,PLOAD,kN	=1388.93
SUBMERGED UNIT WT. OF SOIL,WTSUB,kN/m^3	=10.0
UNIT WT. OF PILE CONCRETE.WTCONC,kN/m^3	=24.0
COEFFICIENT OF HORIZONTAL SOIL STRESS.KS	=0.7
ADHESION FACTOR ,ALPHA	=0.5
BEARING CAPACITY FACTOR FOR COHESIVE SOIL.NC	=9.0
DIAMETER OF SHAFT,DSHAFT,m	=0.9
DIAMETER OF BASE ,DBASE,m	=0.9
NO. OF LAYERS,NLAYER (INTEGER VALUE)	=7
SOIL TYPE OF LAYER-1,SOIL(1)	=NC
FIELD RECORDED SPT VAL.OF LAYER-1,N(1) (INTEGER)	=37
SOIL TYPE OF LAYER-2.SOIL(2)	=NC
FIELD RECORDED SPT VALUE OF LAYER-2,N(2)	=38
SOIL TYPE OF LAYER-3,SOIL(3)	=NC
FIELD RECORDED SPT VALUE OF LAYER-3,N(3)	=34
SOIL TYPE OF LAYER-4,SOIL(4)	=NC
FIELD RECORDED SPT VALUE OF LAYER-4,N(4)	=35
SOIL TYPE OF LAYER-5,SOIL(5)	=NC
FIELD RECORDED SPT VALUE OF LAYER-5,N(5)	=45
SOIL TYPE OF LAYER-6,SOIL(6)	=NC
FIELD RECORDEDE SPT VALUE OF LAYER-6,N(6)	=52
SOIL TYPE OF LAYER-7.SOIL(7)	=NC
FIELD RECORDED SPT VALUE OF LAYER-7,N(7)	=54



PC BEAM CROSS SECTION (COMPOSITE)



PC BEAM CROSS SECTION (NON-COMPOSITE)

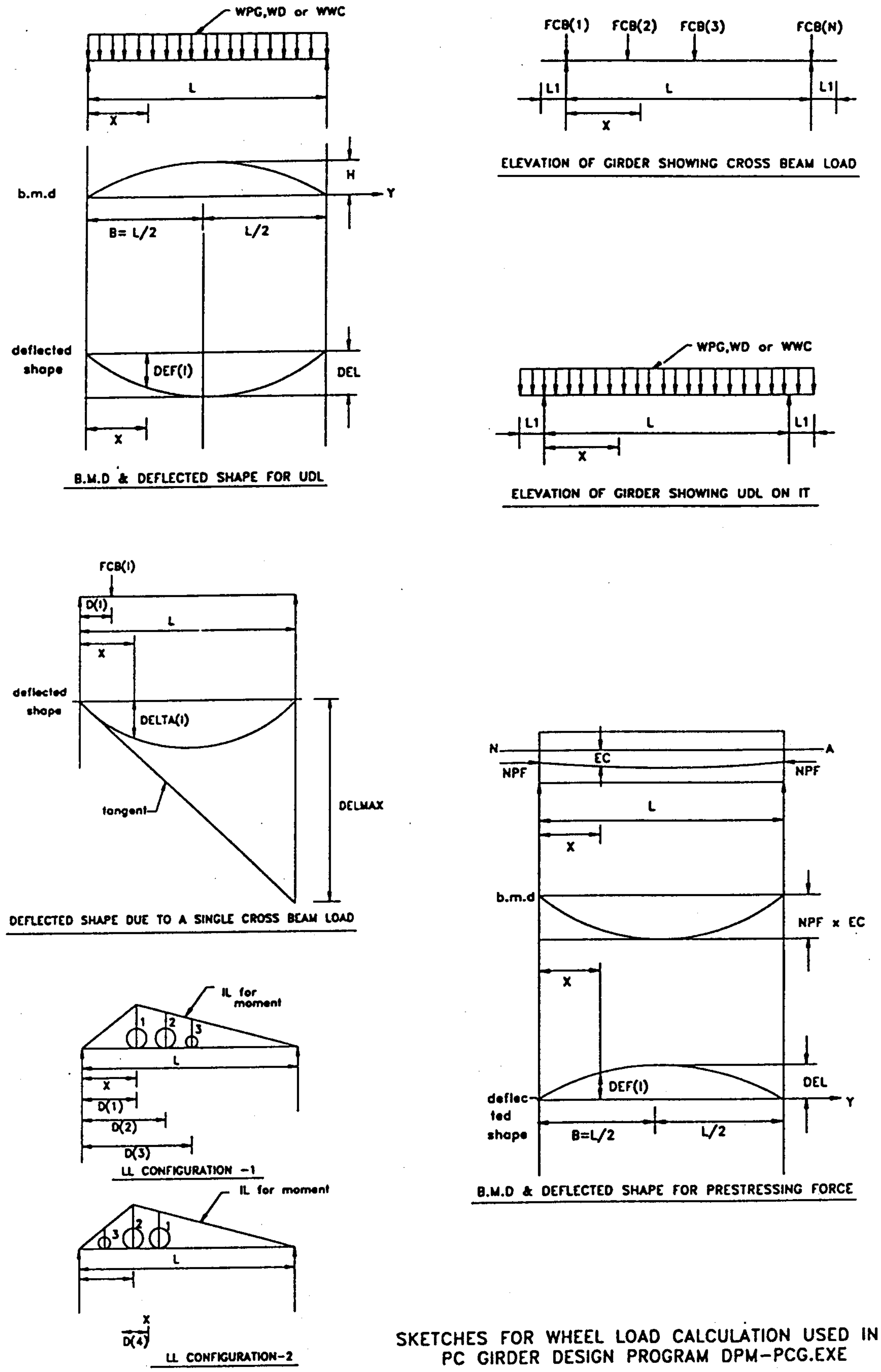


WEDGE

D(I) = SMALLER HORIZONTAL DIMENSION OF A WEDGE
B(I) = LONGER HORIZONTAL DIMENSION OF A WEDGE
H(I) = VERTICAL HEIGHT OF A WEDGE
I = 1,2,3,4,5,6 & 7

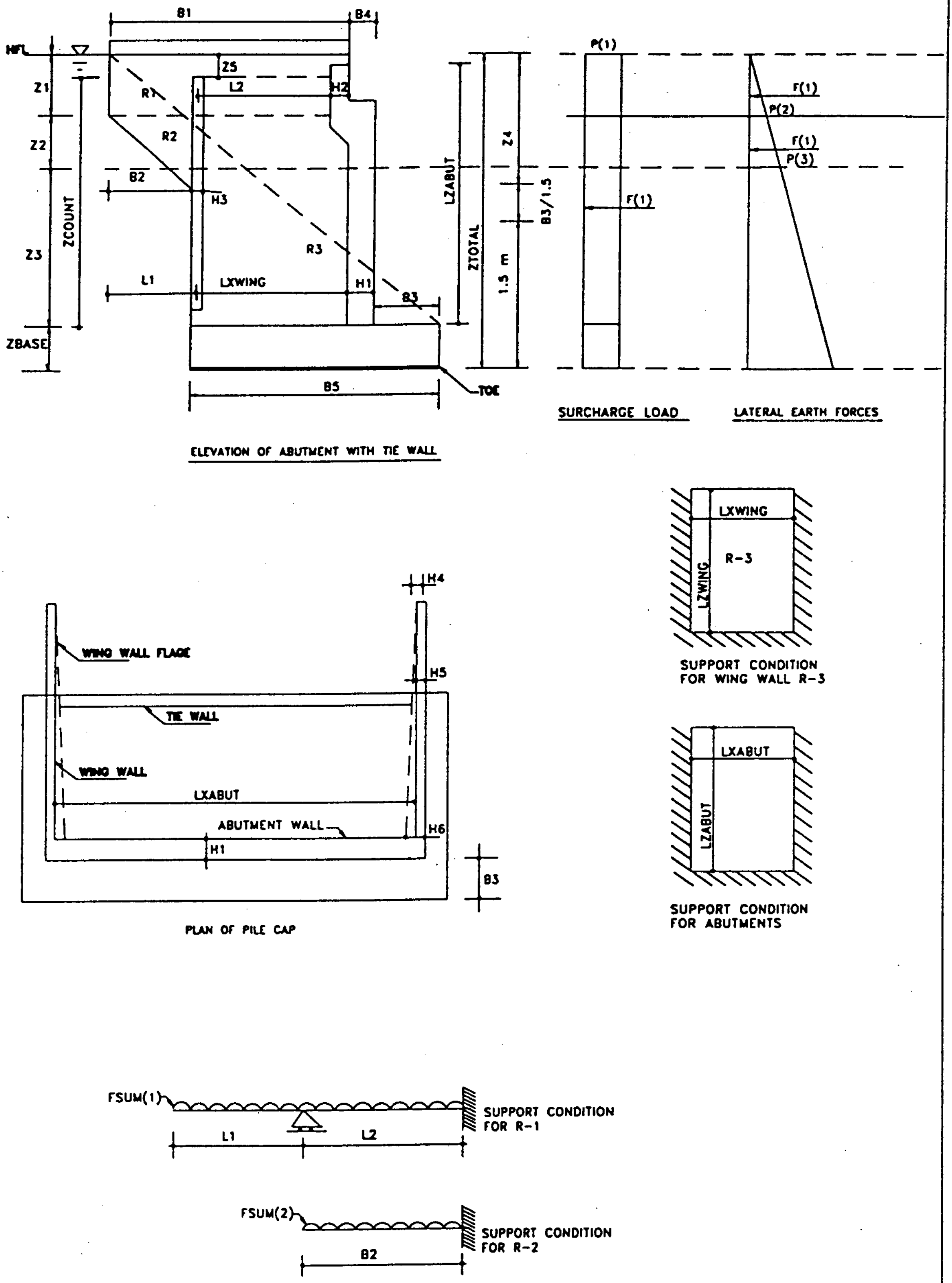
NOTATIONS USED IN DATA FILE DPM-PCG.DAT FOR
DESIGN PROGRAM OF PC GIRDER, DPM-PCG.EXE

FIG. 17.1



SKETCHES FOR WHEEL LOAD CALCULATION USED IN PC GIRDER DESIGN PROGRAM DPM-PCG.EXE

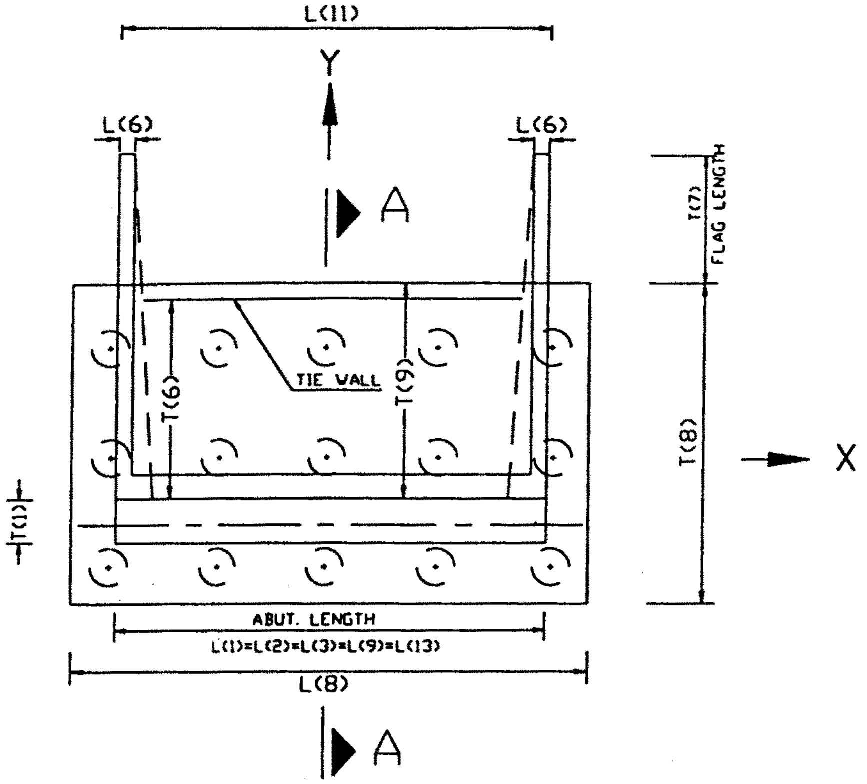
FIG. 17.2



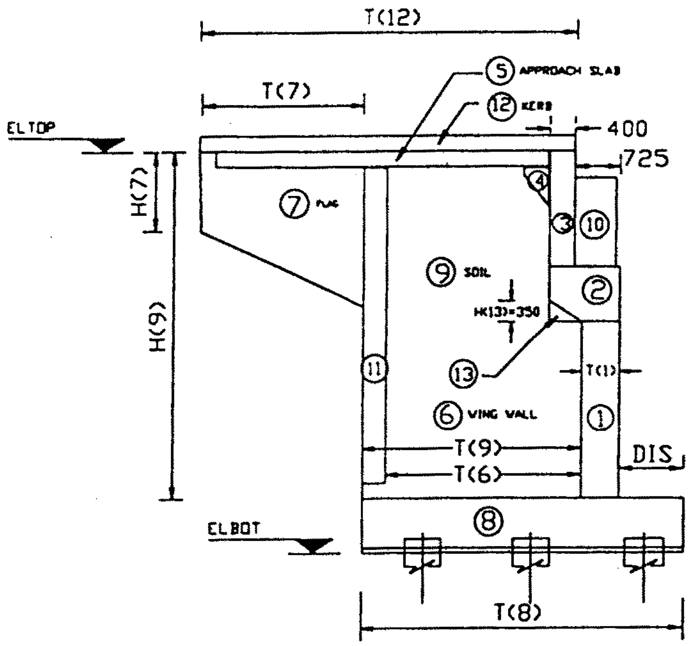
SKETCHES SHOWING NOTATIONS USED IN PROGRAM DPM-ABUT.EXE FOR DESIGN OF ABUTMENT-WING WALL

FIG. 17.3

DPM>FOR-ABT2 1:90



PLAN OF ABUTMENT-WING WALL WITH PILE CAP
(TYPICAL)

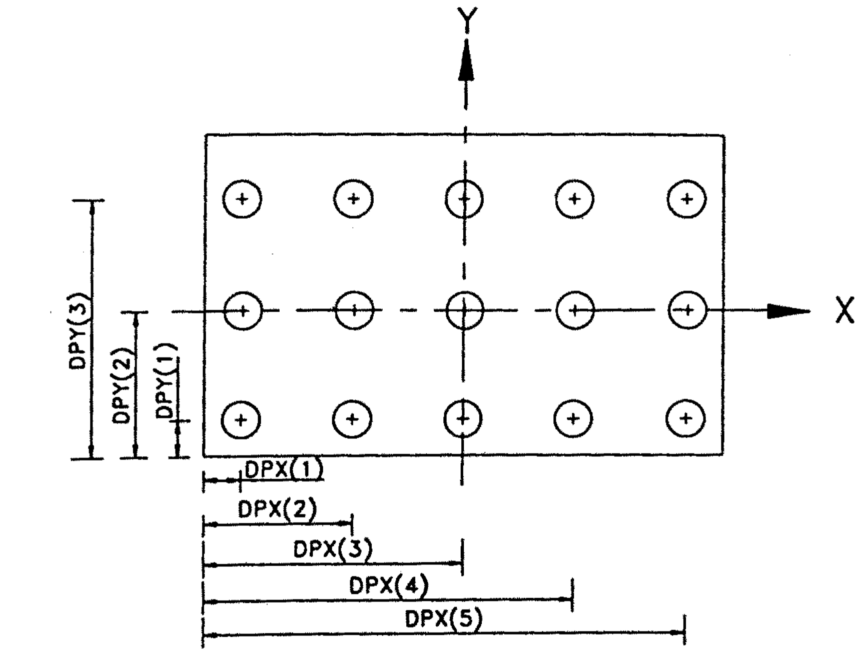


SECTION A-A
(SHOWING DIFFERENT WEDGES OF ABUTMENT)

SKETCHES SHOWING THE WEDGE NUMBERS AND NOTATIONS USED IN DATA FILE DPM-LOAD.DAT FOR PILE DESIGN PROGRAM DPM-LOAD.EXE

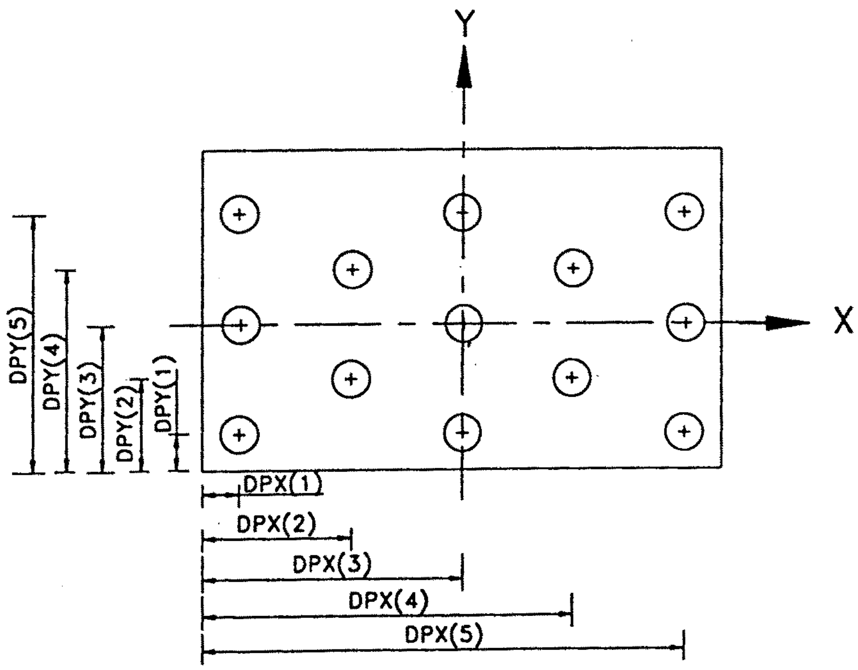
FIG. 17.4

DPM>FOR1LOAD 1:90



PLAN OF PILE FOUNDATION

PILE ARRANGEMENT : RECTANGULAR (R)
NRX = NO. OF PILE ROW PARALLEL TO X-X= 3
NRY = NO. OF PILE ROW PARALLEL TO Y-Y= 5
NVPX(1)=NO. OF PILES IN ROW (1) PARL. TO X-X= 5
NVPX(1) = NVPX(2) = NVPX(3)
NVPY(1)=NO. OF PILES IN ROW (1) PARL. TO Y-Y= 3
NVPY(1)=NVPY(2)=NVPY(3)=NVPY(4)=NVPY(5)



PLAN OF PILE FOUNDATION

PILE ARRANGEMENT : SCATTERED (S)
NRX = 5
NRY = 5
NVPX(1) = NVPX(3) = NVPX(5) = 3
NVPX(2) = NVPX(4) = 2
NVPY(1) = NVPY(3) = NVPY(5) = 3
NVPY(2) = NVPY(4) = 2

SKETCHES SHOWING THE NOTATIONS USED IN DATA FILE
DPM-LOAD.DAT FOR PILE DESIGN PROGRAM DPM-LOAD.EXE

FIG. 17.5

DPM>FOR2LOAD 1:90