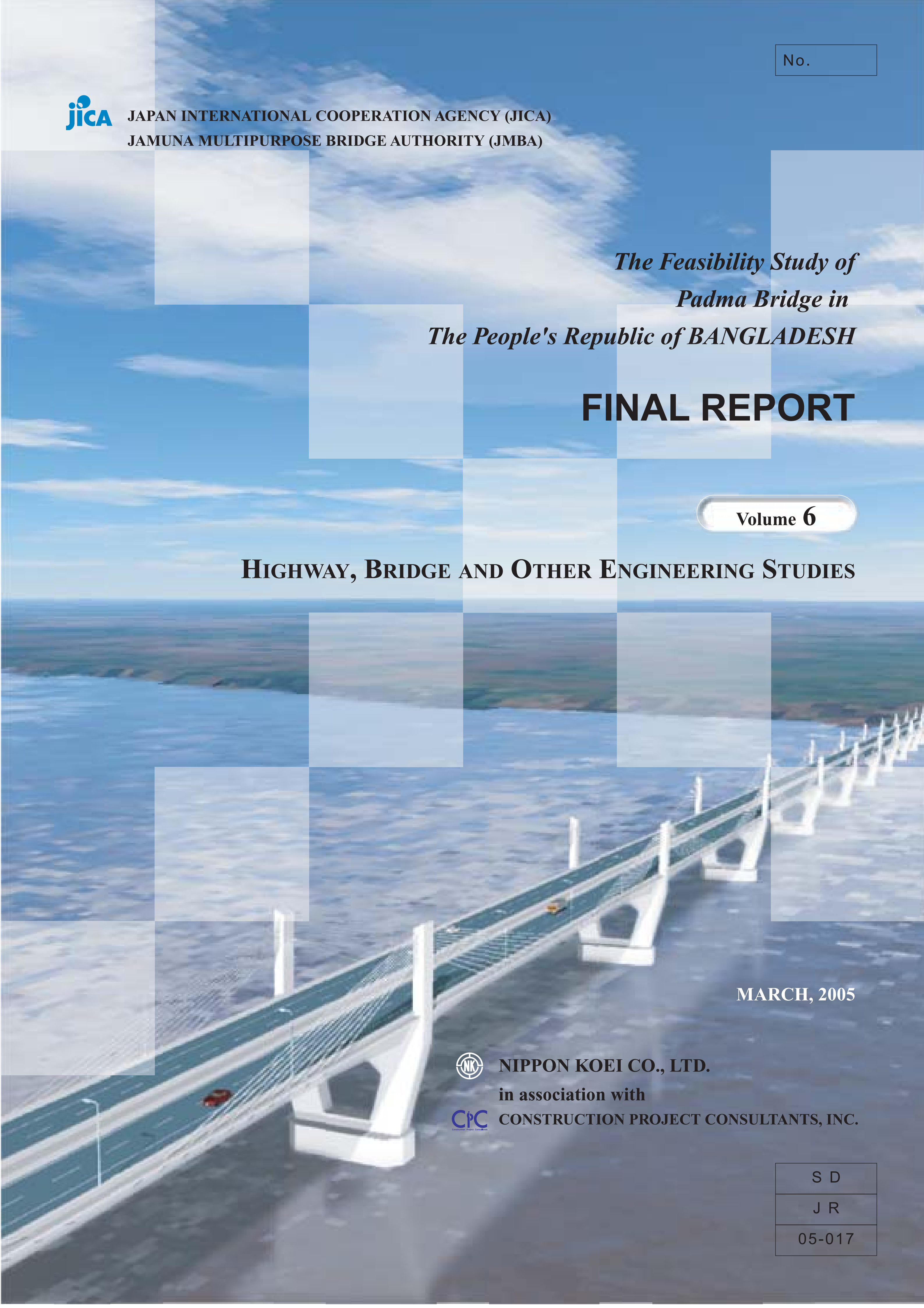

No.

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)
JAMUNA MULTIPURPOSE BRIDGE AUTHORITY (JMBA)

The Feasibility Study of Padma Bridge in The People's Republic of BANGLADESH

FINAL REPORT

Volume 6

HIGHWAY, BRIDGE AND OTHER ENGINEERING STUDIES

MARCH, 2005

NIPPON KOEI CO., LTD.
in association with
CONSTRUCTION PROJECT CONSULTANTS, INC.

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STRUCTURE OF FINAL REPORT

The Feasibility Study of Padma Bridge
Final Report
(Vol. 4 Topographic Survey and Geotechnical Investigation)

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Appendix 8 Bridge Engineering

Chapter 1 Preliminary Technical Considerations for Bridge Planning

1.1 INTRODUCTION

Data collection and preliminary studies have continued through September 2003 and are on going. The major task in this period was to select two prospective sites from four alternative sites for bridge crossings. The technical assumptions adopted for this preliminary planning stage for the bridge were

- Recent river opening need to be maintained.
- Guide bunds would be constructed on both riverbanks at all the alternative sites.
- Although any subsoil exploration has not been conducted yet by the Study Team, foundation type and sizes could be predicted by using available subsoil data in the vicinity of the respective alternative sites.

1.2 CONFIGURATION OF THE BRIDGE AT THE ALTERNATIVE LOCATIONS

Bridge lengths and foundation types and sizes were regarded as the main factors for selecting two prospective sites from the four alternative crossing sites.

Indicative bridge lengths were estimated for the four alternative crossing sites. It was assumed that the length would be a function of the recent river openings and that each crossing site will have guide bunds similar to and constructed in a similar way to the Bangabandhu (Jamuna) Bridge.

As a means of verifying the proposed bridge lengths, checks were made using:

1) the relationship between design discharge and length of bridges constructed over the major rivers in Bangladesh, and
2) conventional formula to estimate required bridge length for design discharge.

Subsoil exploration will be conducted by the JICA Study Team during the low water season, after October 2003. Therefore, predictions of possible foundation types and sizes have been made by using the existing subsoil data which had been obtained in the pre-feasibility study of the Padma Bridge together with borehole data provided by BWDB obtained during ground water surveys they had conducted.

1.2.1 Bridge Length

(1) Indicative Bridge Length

Indicative bridge lengths were used for a comparison of the alternative crossing sites. As with the Bangabandhu Bridge, sufficient distance from the recent riverbanks in low water seasons would be required for construction of the guide bunds whichever alternative bridge site was considered.

Indicative bridge lengths were determined by summing:

a) The maximum river width in low water seasons in the past 30 years.
b) The space required for construction of the protection bunds allowing for excavation of side slopes assuming the guide bunds would have a profile 1:6 to predicted scour depth and a 1:4 side slope to the ground forming the temporary dam to the river that will be 170 m wide at water level. (see Figure 1.2.2).
c) Viaduct length from guide bund to abutment, which should be determined based on stability of approach road embankment, but assumed to be 60 m at this time.

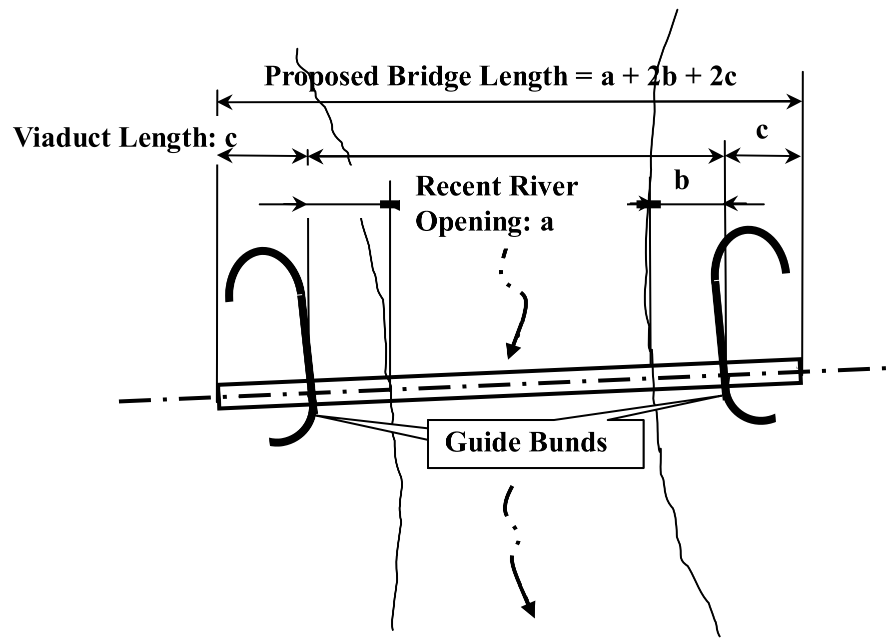


Figure 1.2.1 Dimensional Factors for Proposed Bridge Length

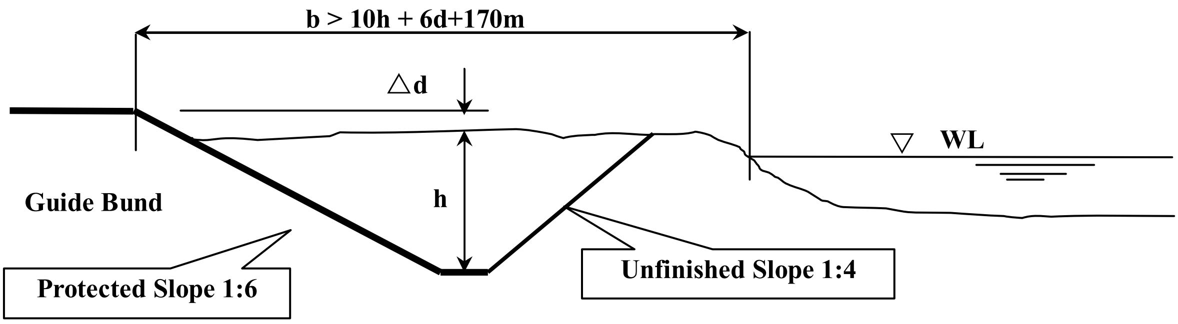


Figure 1.2.2 Construction Space for Guide Bund Works

Indicative bridge lengths are summarized in Table 10.2.1.

Table.1.2.1 Indicative Bridge Lengths by Alternative Crossing Sites

	Design Discharge (m^3/sec)	River Opening a (m)	Construct. Space b (m)	Viaduct Length c (m)	Indicative Bridge Length (m) = a+2b +2c
Site-1: Patria - Goalundo	151,400	5,000	480	60	6,080→6,100
Site-2: Dohar - Charbhadrasan	(147,000)	8,480	490	60	9,580→9,600
Site-3: Mawa - Janjra	134,400	4,920	500	60	6,040→6,100
Site-4: Chandpur - Bhedarganj	(162,400)	9,600	510	60	10,740→10,800

Remarks: Design discharges given in brackets are estimated from flow data obtained remote from the site as no flow data is available at these locations.

(2) Verification of Bridge Lengths by Relationship between Design Discharge and Bridge Length in Major Bangladesh Rivers

Regression equation of trend line for the completed or on-going bridges over the major rivers in Bangladesh is shown in Figure 1.2.3, which includes Bangabandhu bridge over the Jamuna, Paksey bridge over the Ganges, Bhairab, Meghna and Meghna-Gumuti bridges over the Meghna, Arial Khan bridge over the Arial Khan, and Rupsa bridge over the Rupsa.

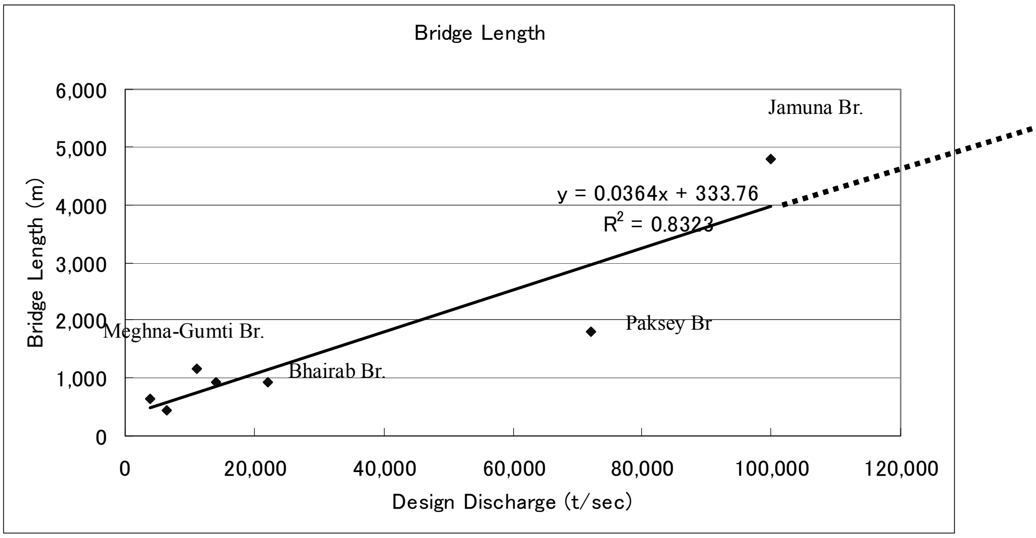


Figure 1.2.3 Design Discharges and Bridge Lengths in Major Bangladesh Rivers

The distribution is assumed to be linear for the purposes of this comparison. The bridge length for each alternative site has been extrapolated from above data using the equation for the trend line.

Table 1.2.2 shows the bridge lengths obtained using the above regression equation (verification mean) vs. those proposed in the previous paragraph (1).

Table 1.2.2 Checking of Proposed Bridge Length by Design Discharge – Bridge Length Relationship

	Design Discharge (m^3/sec)	Bridge Length (m)		Judgment
		Obtained from the Regression Equation	Indicative using water opening plus guide bunds	
Site-1: Patria - Goalundo	151,400	5,612	6,100	OK
Site-2: Dohar - Charbhadrasan	(147,000)	5,612	9,600	OK
Site-3: Mawa - Janjra	134,400	5,139	6,100	OK
Site-4: Chandpur - Bhedarganj	(162,400)	6,340	10,800	OK

From the above table, the proposed bridge lengths for the respective sites fulfill the requirements of the design discharge – bridge length relationship.

(3) Verification of Bridge Length by Lacy's Formula

According to Lacy's formula, the required bridge length is obtained based on the following:

$Ws = CQ^{1/2}$

Where, Ws: River opening (bridge width) in meter

C: Coefficient, which varies from 3.3 to 4.9 in m-sec unit.

Q: Design Discharge in cu.m/sec.

The coefficient of C ranging 3.3 to 4.9 is used for ordinary rivers, but not applicable for braided rivers like the Padma, Jamuna, Meghna in Bangladesh. The actual coefficients of the major bridges completed or under construction in Bangladesh are in a range of 6.3 (Bhairab bridge over the Upper Meghna) to 15.2 (Bangabandhu bridge over the Jamuna).

The coefficient of C for the proposed bridge lengths and design discharges by alternative crossing sites are summarized in Table 1.2.3.

Table 1.2.3 Checking of Proposed Bridge Lengths by Coefficient of Lacy's Formula

	Design Discharge (m3/sec)	Proposed River Opening (m)	Coefficient C of Lacy's Formula (m-sec unit)	Judgment (10<C<15)
Site-1: Patria - Goalundo	151,400	5,980	15.4	OK
Site-2: Dohar – Char Bhadrasan	(147,000)	9,480	24.7	OK
Site-3: Mawa – Janjira	134,400	5,980	16.3	OK
Site-4: Chandpur - Bhedarganji	(162,400)	10,680	26.5	OK

Note: Proposed River Opening = (Proposed Bridge Length) – (Viaduct Length)

All the alternative sites have values of more than 15, which might be applicable to braided rivers.

1.2.2 Subsoil Conditions along the Padma River

Subsoil exploration by mechanical borings will begin after October 2003 when river water recedes. Specific sites for the subsoil exploration will follow the selection of prospective alternative crossing sites.

Options of conceivable foundation types and sizes will be discussed in the Subsection 10.2.3 based on the available subsoil data below.

(1) Available Subsoil Data

Currently available data for predicting the foundation types and sizes.

- Padma Bridge Study, Prefeasibility Report Volume III, Annex B: Geotechnical and Preliminary Seismic Study, Rendel Palmer & Tritton, Nedeco in association with Bangladesh Consultants Ltd., February 2000
- This report includes 3 no. of 115m long boring logs along with SPT values at Site-3, and 1 no. of the same at Site-1.
- Boring Logs in BWDB Archive
- In the BWDB archive, deep boring longs (140m to 300m) in the vicinity of Site-1 to 4, which were obtained for ground water survey, are available. Although these boring logs exclude NPT values, they provide useful details about the subsoil conditions of the alluvial plain along the Padma River.
- Geology Map of Bangladesh, Geological Survey of Bangladesh and US Geological Survey, 1990

(2) Bearing Strata

All the alternative crossing sites are situated in deltaic alluvial soil of the Padma River deposits. For all the bridge foundation, subsoil layers below the scour level (approx. 25m) are significant. Table 1.2.4 indicates typical subsoil conditions below the scour level.

Table 1.2.4 Predicted Subsoil Condition by Alternative Crossing Sites

	Typical Subsoil of Bearing Strata		Indicative Foundation Dept
Site-1: Patria - Goalundo	From 25 to 90m: Below 90m:	Silty fine sand Silty fine to medium sand	90m
Site-2: Dohar – Char Bhadrasan	From 25 to 70m: Below 70m:	Clay, silt, fine sand Medium to coarse sand	70m
Site-3: Mawa – Janjira	From 25 to 80m: Below 80m:	Silty fine sand Silty fine to medium sand	80m
Site-4: Chandpur - Bhedarganji	From 25 to 80m: Below 80m:	Silt Medium to coarse sand	100m

1.2.3 Possible Foundations of the Padma Bridge

As the foundations are to be constructed in extremely deeper layers, conceivable types of the foundations are limited to the following:

(1) Site-1: Paturia-Goalundo

Foundation Depth: 90m or more

Foundation Type:
a) Large diameter cast-in-place RC piles, which are world-wide popular in case of deep foundation.
b) Large diameter tubular steel pipe driven piles, which were used for the Bangabanndhu (Jamuna) Bridge.
c) RC open caissons by jack-down method with cable anchors. This method, developed in Japan, was adopted on the New Nizammudin Bridge over the Yamuna River in Delhi, and is now often used in India.

(2) Site-2: Dohar-Charbadrasan

Foundation Depth: 70m or more

Foundation Type:
a) Large diameter cast-in-place RC piles
b) RC open caissons by jack-down method with cable anchors.
c) Large diameter tubular steel pipe driven piles.

Note) Relatively thicker layers of cohesive soil (clay and silt) are found in the existing boring logs by BWDB for the ground water survey at Faridpur and Hariampur. The large diameter tubular steel pipe driven piles might face at difficulties for penetration because of higher skin friction.

(3) Site-3: Mawa-Janjra

Foundation Depth: 80m or more

Foundation Type:
a) Large diameter cast-in-place RC piles
b) Large diameter tubular steel pipe driven piles
c) RC open caissons by jack-down method with cable anchors.

(4) Site-4: Chandpur-Bhedarganji

Foundation Depth: 100m or more – records from the BWDB indicate channel depths of up to 65m in this area (BWDB measurements made in 2002). No suitable foundation has been identified for these conditions.

1.3 UTILITIES TO BE INSTALLED

Information of public utilities, viz. electric power transmission line, natural gas and telecom fiber optics, are summarized in Table 1.3.1, which shows whether the agencies concerned have interests in the respective sites.

Table 1.3.1 Bridge Provision for Public Utilities

	Electric Power Transmission Line	Natural Gas	Telecom Fiber Optics
	Bangladesh Power Development Board	Petrobangla	Bangladesh Telegraph & Telephone Board
Site-1	Not suitable	Suitable	Suitable
Site-2	Suitable	Suitable	Suitable
Site-3	Very Suitable (Lowest distance from the future generation site at Meghna Ghat to this site)	Suitable	Suitable
Site-4	Not suitable	Suitable	Suitable
Remarks	Armored high voltage cables would have to be considered in addition to the overhead cables supported by towers.		

1.4 EVALUATION OF ALTERNATIVE BRIDGE CROSSINGS

The preceding information is summarized in Table 1.4.1. From the data Site-1 and Site-3 are regarded as preferred bridge sites because of the shorter bridge length and variety of applicable foundation types which will offer both lower costs and simpler construction planning than Site-2 and Site-4. Though Site-2 is advantageous to installation of public utilities, it is discounted because of longer bridge length which is much more dominant factor than public utilities. At site-4 the channel depth which could affect the foundations is considered to present an unacceptable risk to construction and stability of a structure at that location.

Table 1.4.1 Summary of Preliminary Consideration for Bridge Planning

	Indicative Bridge Length	Predicted Depth of Bearing Strata	Conceivable Foundation Types	Appurtenance for Public Utilities
Site-1	Approx. 6.1km Shortest, as with Site-3	Below 90m Silty fine to medium sand	3 type options: Large diameter cast- in-place RC piles Large diameter steel pipe driven piles RC open caissons	2 utility Natural gas pipe line Telecom fiber optics
Site-2	Approx. 9.6km 2nd longest	Below 70m Medium to coarse sand	32 type options Large diameter cast- in-place RC piles Large diameter steel pipe driven piles RC open caissons	3 utilities Electric power transmission line Natural gas pipe line Telecom fiber optics
Site-3	Approx. 6.1km Shortest, as with Site-1	Below 80m Silty fine to medium sand	3 type options: Large diameter cast- in-place RC piles Large diameter steel pipe driven piles RC open caissons	3 utilities Electric power transmission line Natural gas pipe line Telecom fiber optics
Site-4	Approx. 10.8km Longest	Below 100 m recorded channel depth of 65 m Medium to coarse sand	No suitable foundation types identified	2 utility Natural gas pipe line Telecom fiber optics

Chapter 2 Preparatory Study for Preliminary Design of Bridge and Highway

2.1 GENERAL

Preliminary design of the Padma Bridge and connecting roads will be conducted from May 2004 for the selected bridge site. In advance, preparatory study was conducted for providing reference data for selecting a final bridge site as well as for conducting the preliminary design of the Padma Bridge and connecting roads.

2.2 BASIC CONDITION ANALYSIS FOR PRELIMINARY BRIDGE DESIGN

2.2.1 Preconditions for Preliminary Design of Bridge

Those mentioned below are major preconditions to be taken into account for preliminary design of the bridge over the Padma River at the selected site.

(1) Soil

There is a remarkable difference of geological profile between Mawa Janjira site and Paturia Goalundo site; N-value at Mawa Janjira site is much larger than that at Paturia Goalundo site below the maximum scour level about –40m PWD.

Depth of supporting strata is estimated to be at –80m PWD for both sites according to the geotechnical test conducted by the study team.

The soils are generally soft and the sands when exposed to the river are likely eroded. Furthermore comparatively high content of mica was observed; it is evaluated to range from 3.5 to 17.2 %. In Jamuna Bridge Project, it is reported that some flow sliding occurred during excavation works for guide bund foundation of which soil contains some mica. Judging from these failure accidents, also in Padma Bridge Project, enough attention needs to be paid to excavation works. Also the geologist of the team proposes to reduce design values of internal friction angle by four degrees from the estimated value in consideration of the effect of mica content.

Regarding potential for liquefaction, some countermeasures are judged to be needed for bridge substructure as a result of the analysis made by the geologist.

(2) Navigation Clearance

BIWTA controls the clearances for navigation to be provided over rivers in Bangladesh.

The regulatory minimum clearance is to provide the rectangle formed by the minimum vertical (18.29m) and minimum horizontal (76.22m) clearances above the SHWL.

(3) Characteristics of the River

The Padma River is classified as meandering river, and scours the riverbed to a large extent. Estimated scour depths at Mawa- Janjira Site (MJ-site) and Paturia Goalundo Site (PG-site) are as follows:

Table 2.2.1 River Characteristics at MJ-site

At mid-river width		
	Riverbed elevation (PWD)	Depth below DHWL
Without Pier	-23.1m	32.8m
With Pier (3.0m dia.)	-28.6m	38.3m
At river bank		
Without Pier	-33.8m	43.5m
With Pier (3.0m dia.)	-39.3m	49.0m

Table 2.2.2 River Characteristics at PG-site

At mid-river width		
	Riverbed elevation (PWD)	Depth below DHWL
Without Pier	-26.4m	33.8m
With Pier (3.0m dia.)	-31.9m	39.3m
At river bank		
Without Pier	-37.5m	44.9m
With Pier (3.0m dia.)	-43.0m	50.4m

(4) Ship Impact

Ship impact is an important threat to the bridge.

River traffic is made up of numerous large and small vessels: the various data available indicate that vessels of up to 4,000DWT use the river.

The impact load determined as appropriate for the Padma Bridge in the Pre-feasibility Report is 23.3 MN, whereas 5 MN was used for the Jamuna Bridge, 14.6 MN and 7.3 MN for a head-on impact and sideway impact respectively of the Bhairab Bridge.

The impact of 23.3 MN will be verified by the Study at the next stage.

(5) Seismic Force

Bangladesh has experienced several large earthquakes in the last 130 years. Among them seven have had a magnitude greater than 7.0.

Earthquake effects are also important in design. For long spans seismic load effects dominate rather than ship impact. This change occurs when the ship impact loads, which is constant, is exceeded by the loading caused by the seismic acceleration of the mass of the bridge span.

Seismic events in addition to causing horizontal loading can induce liquefaction in the top of the supporting ground.

According to the Prefeasibility Study, acceleration at Goalundo and Mawa are established as 0.15g and 0.125g respectively, which will be required to be reviewed.

(6) Future Traffic Volume

According to the traffic demand with bridge cases forecasted by the team, the traffic volume will be 19,850 and 41,550 vehicles a day at Paturia-Goalundo and Mawa-Janjira sites in 2025 respectively.

The bridge will have to be designed to accommodate the traffic volumes.

(7) Public Utilities

Like the Jamuna Bridge, a number of utilities are considered to be loaded on the proposed bridge.

There are needs for 400 kV transmission line over Padma, 30" diameter of natural gas pipeline and some fiber optic cable for telecommunication, though these issues are not finalized at this stage of the Study.

The load intensity of the utilities is estimated approximately to be 1.0 ton per linear meter.

2.2.2 Policy for Preliminary Bridge Design

In order to satisfy the preconditions, the preliminary design follows the policy mentioned below.

1) 4-lane configuration is adopted for both Paturia-Goalundo and Mawa-Janjira sites to cope with the future traffic volumes.
2) Span lengths must be longer than 100 meters to ensure the smooth and safe navigation.
3) The Padma River will shift the main flow over the entire width for the long design life of the bridge, and thus navigation span cannot be fixed, therefore all the spans over the river must satisfy the navigation limit and be navigable.
4) Foundations must be so designed as to be suitable to characteristics of the subsoil obtained from the geotechnical investigations for the PG and MJ sites as discussed in Chapter 6, and the structures must be selected taking into account scour and liquefaction. Careful consideration must be given to abnormal horizontal loads such as ship impact and earthquake.
5) An electric power cable with huge capacity greater than ever installed, gas pipeline and telecom cables are expected to cross along the bridge. The bridge design must consider not only the load, but also the location and the installation details.

2.2.3 Methodology for Preparatory Study

The preparatory study aims at presuming an appropriate range of span length, and then recommending superstructure and foundation in consideration of the said preconditions.

(1) Superstructure

In selecting an appropriate superstructure type under specific conditions, span length and major materials are essential factors affecting construction cost. Supporting system of whether continuous or simply supported affects not only the cost, but the constructability, drivability and maintainability.

There are various superstructure types, but each type has a suitable possible range of span length appropriate to apply in terms of economics.

Steel and concrete are materials usually utilized for major elements of bridges, and each material has advantages and disadvantages.

(a) Minimum span length

In order to keep the navigation limit of the Padma River, span length must be greater than 100 meters as seen in the following Figure 2.2.1.

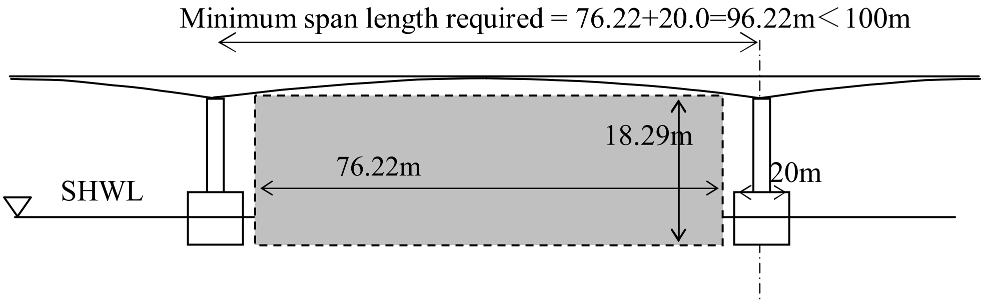


Figure 2.2.1 Navigation Clearance

(b) Superstructure types applicable to span longer than 100m

Figure 2.2.2 and Figure 2.2.3 show the superstructure types and applicable range of span lengths for concrete and steel superstructures.

These types are usually applicable to the span ranges in terms of economics.

(c) Selection of materials

Usually concrete and steel are utilized for major elements of superstructure.

To be brief, steel is strong and light-weighted compared with concrete, but some countermeasures must be taken against the corrosion. Concrete is cheaper and easier to obtain in most countries.

More details of advantages and disadvantages of these materials are indicated by the Table 2.2.3.

(d) Continuity vs. simply supported construction

Continuity and simply supported construction of superstructure show different behavior to safety against catastrophes, differential settlement, constructability and so on. Out of the superstructure types shown in Figures 2.2.2 and 2.2.3, the three arch types are simply supported, and all others are continuous.

Figure 2.2.4 compares characteristics of these two systems.

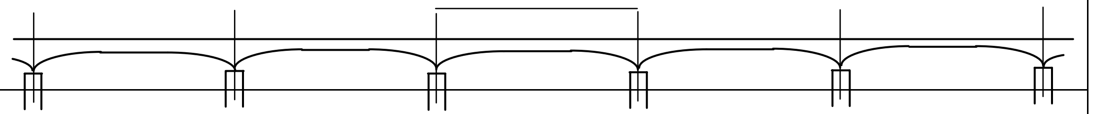
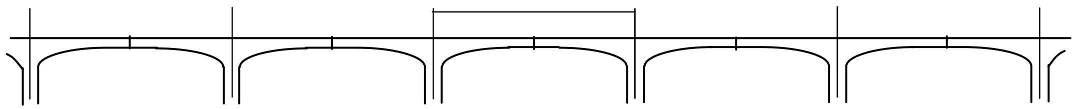
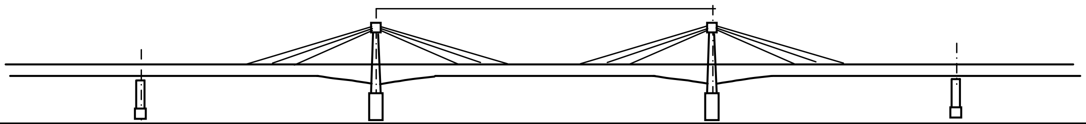
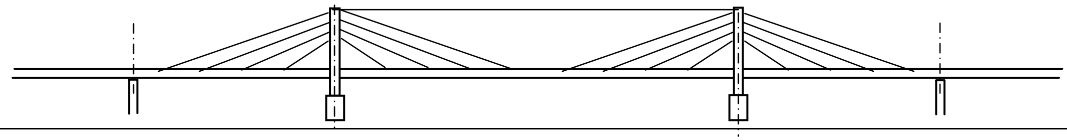
	Type	Applicable Span Length (m)	Configuration
1	Box Girder Bridge (Continuous)	50＜L＜110	
2	Box Girder Bridge (Rigid Frame)	40＜L＜140	
3	Box Girder Bridge (Hinged)	60＜L＜180	
4	Extra-dosed Girder Bridge	100＜L＜200	
5	Cable Stayed Girder Bridge	50＜L＜250	

Figure 2.2.2 Applicable Concrete Superstructure Types over Span Length of 100m

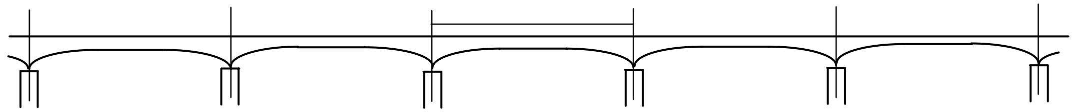
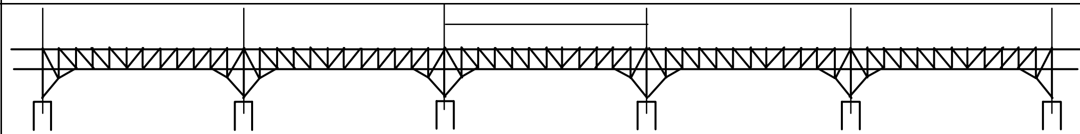
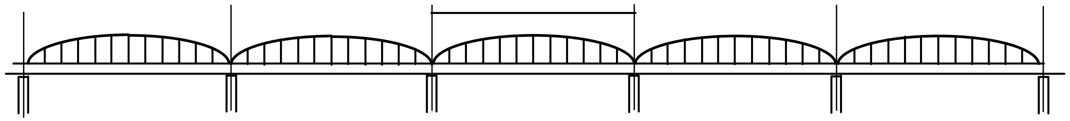
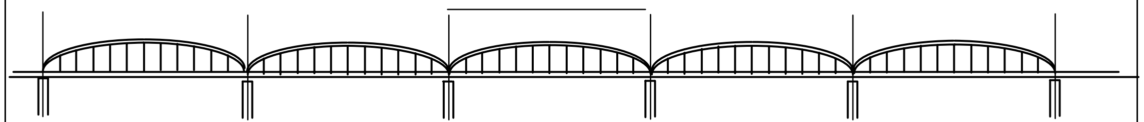
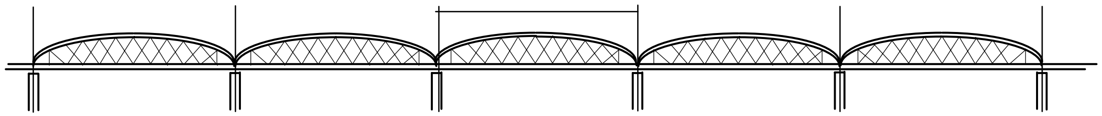
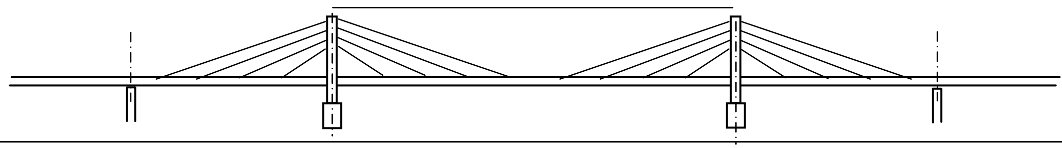
	Type	Applicable Span Length (m)	Configuration
1	Continuous Box Girder (with steel deck)	30＜L＜150	
2	Continuous Truss Bridge	60＜L＜110	
3	Arch Bridge (Langer Type)	50＜L＜120	
4	Arch Bridge (Lohse Type)	80＜L＜150	
5	Arch Bridge (Nielsen Type)	100＜L＜170	
6	Cable Stayed Bridge	130＜L＜500	

Figure 2.2.3　Applicable Steel Superstructure Types over Span Length of 100m

Table 2.2.3 Comparison of Material Characteristics

Items	Steel Superstructure	Concrete Superstructure
Weight of Materials	- Comparative light weight taking the strength into consideration	- Approximately five times heavier than steel taking a ratio of the material weight to the strength
Ease of Handling	- Easy to handle due to the light-weight and strength against tensile and compressive force	- Heavy lifting and transporting facilities require due not only to the weight but also the vulnerability to tensile force
Maintenance	- Repaint required at regular intervals, although there's weathering steel requiring no paint - Whole life cycle cost increased	- No paint required
Inspection	- Possible to observe the structure directly	- More difficult to inspect concrete structures where reinforcing bars and tendons are hidden from the direct view.
Availability	- All prefabricated steel segments have to be imported and no contribution to Bangladesh industries expected	- Precast segments may be prepared near the site, or all works carried out at site for cast-in-place
Substructure/foundation	- Relatively small substructure and foundation is sufficient due to the lightweight superstructure.	- Large substructure and foundation needed to bear the heavy superstructure.

Notes
In addition to steel superstructures and concrete superstructures, there is variety of composite structures.

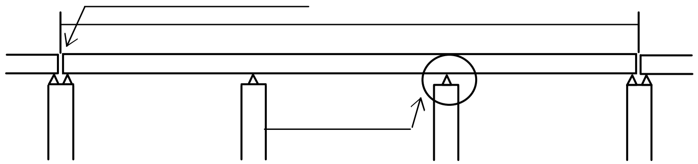
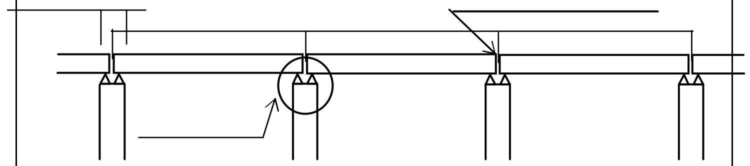
Items	Continuous Superstructure	Simply Supported Superstructure
Sketch of structure		
Safety against Catastrophes	- Greater survivability and safety for users in the face of ship collision, earthquake and other impact forces	- Less survivability - Installation of linkages across joint of superstructures
Differential Settlement of Foundation	- Superstructure imposed by permanent bending moment due to differential settlement of foundation	- No bending moment imposed even under differential settlement
Constructability	- Variety of alternatives of erection method including most methods applicable to simple structures and additionally cantilever balanced erection and so on	- Limited alternatives of erection method due to inability to transfer bending moment across the deck joint - False work (temporary supports) erection, large block erection (complete lift) by jack or crane, and gantry erection applicable
Installation/Maintenance of Expansion Joint	- Less number of expansion joints required	- Expansion joint required on every pier - Frequent joints lead passengers to a poor ride quality. - Difficulty and cost on installation and maintenance of many expansion joints

Figure 2.2.4 Comparison between Continuous and Simply Supporting System

(e) Characteristics of Each Superstructure Types

i) Concrete Bridge

Box Girder Bridge

This type can be classified into three. They are Continuous Box Girder Bridge, Continuous Rigid Frame Girder Bridge and Hinged Box Girder Bridge, having similar appearances and structural characteristics. But their connecting details provide different structural characteristics.

This type is can be applied to a span length of up to 180m, but most is less than 150m.

Most of major bridges in Bangladesh belong to this category.

Extradosed Girder Bridge

In appearance the extradosed girder bridges are similar to cable stayed girder bridges, but they differ in the following two respects. Typically the tower height to span length ratio is approximately 1/10, and the optimum girder depth is 1/30 to 1/35 of the main span length.

The applicable span ranges between 100m and 200m.

Cable Stayed Girder Bridge

This is more suitable for longer span than other concrete bridges. Usually they are applied for a span up to 250m. The tower is as high as one fifth of the span length, which is much higher than that of extradosed girder bridge. Furthermore it enables the girder to be less deep and thus provide spacious navigation clearance beneath the bridge by the stayed cables inclined sharply.

Girders and towers for cable stayed girder bridge can be constructed of both concrete and steel.

ii) Steel Bridge

Box Girder Bridge

Steel box girder bridge has a similar appearance and characteristics to that of concrete. For the Jamuna Bridge project the consultants developed a steel box girder scheme in parallel with the concrete one.

Steel has different maintenance requirements from concrete such as repainting.

Continuous Truss Bridge

If the configuration includes a railway, either at day one or with future provision, then this type can be planned as a double decked continuous truss arrangement.

One particular weakness is corrosion due to the vast surface area, large number of slender elements and complex structure of the nodes, thus careful maintenance is required.

Arch Bridge

Langer arch, Lohse arch and Nielsen arch belong to a tied arch family. They are simply supported structures and applied mostly up 170m. Its relative lack of popularity lies in the difficulty of its erection due to the structural characters of simple support system and complex structure with good number of elements, which might cause local corrosion.

Cable Stayed Girder Bridge

Steel cable stayed bridges can be used for spans in the range between 200m and 800m. They are most often used in situations where high and wide navigation clearances are required or where an owner decides that a "landmark" structure is justified. They are invariably relatively expensive, often costing in the region of 25 to 50 percent than girder bridges.

(f) Candidate superstructure types

Among the above-mentioned superstructure types, some are deleted from a list of candidates for the subsequent cost estimating due to crucial disadvantages of constructability and/or maintainability.

Table 8.2.4 shows disadvantages of each type, where "X" indicates the disadvantage of the type in constructability or maintainability.

Table 2.2.4 Disadvantages of Bridge Type

	Type	Constructability	Maintainability
PC Bridge	Box Girder		
	Extradosed		
	Cable Stayed		
Steel Bridge	Continuous Box		
	Continuous Truss		X
	Arch	X	X
	Cable Stayed		

Thus the steel continuous truss and steel arch are excluded from the succeeding procedure.

The remaining five types are usually applied within the span ranges shown in Table 8.2.5.

Table 2.2.5 Applicable Span Length of Candidate Types

	Type	Applicable Span (100m – 150m – 200m – 250m)
PC Bridge	Box Girder	150m (frequently applied), 180mm (applicable)
	Extradosed	200m
	Cable Stayed	250m
Steel Bridge	Continuous Box	150m
	Cable Stayed	500m

: Frequently applied span length, :Applicable span

(2) Substructure/Foundation

(a) Loading

As the primary stage of the foundation design it is necessary to identify the loading and circumstances that will cause adverse conditions that the foundations must be designed to resist.

The following may be divided into one of four general categories.

- Structural Loads: Weight of the structure, surfacing and effects from structural behavior.
- Live Loads: The loads from traffic on the deck.
- Environmental Loads: Wind loads, loads from the river (stream flow, buoyancy, wave forces).
- Abnormal Loads: Ship impact or earthquake loads.

(b) Scour and Flow

Scour influences the following aspects of substructure and foundation design;

- The design height of the substructure: That is the distance from scour riverbed level to bearing level.
- Overturning effects of the foundation: As scour increases the lever arm at which forces act increases.
- Loads form water flow increases.
- The stiffness of the substructures becomes important: For slender members vibration and buckling must be checked.

(c) Subsoil

The subsoil conditions along the river are described in Chapter 6.

Below the scour depths projected at the preferred bridge sites the soil is fine to medium sand with traced of mica. It is the engineering properties of these strata that will govern the foundation design for vertical and particularly horizontal load capacity. The resistance of the ground to lateral loads and movement will be the most important design consideration. The presence of mica can have a marked effect on the soil properties.

(d) Types of Pile

The three main issues of loading from ship impact, depth of scour and the capacity of the ground to resist horizontal loading will govern the design of the foundations. The foundations will have to be strong enough to carrying the loads and suitable distribute the loads through the ground to ensure the soil strength is not exceeded.

Four types of foundation have been identified as potentially suited for use for the Padma Bridge. They are shown in the Figure 2.2.5 and Figure 2.2.6.

i) Driven steel tubular piles

One option for the construction of piled foundations for the Padma Bridge would be to use tubular steel piles, which is a same foundation type as used for the Jamuna Bridge. Using steel piles would be the quickest way to construct a foundation and the piles could be installed in a raked or vertical orientation. One advantage of steel piling is the strength of the pile section and that steel will work equally well in tension and compression. For the Padma Bridge it is expected that piles will subject to tension loading.

ii) Insitu concrete piles

Large diameter bored reinforced concrete piles could be adopted for the Padma Bridge foundations. This option would limit the foundation to a vertical piles arrangement only. Bored pile construction uses a sequence of operations. This method is slower than driven steel pile installation and concrete piles will be weaker under the tensile loading expected on the piles of this bridge.

iii) Concrete caissons

Two types of caisson have been considered for primary design purposes, a tube (well) caisson and a large diameter caisson. Both types were assessed for their suitability as foundations in the Padma bridge environment.

<u>Tube caissons (or Well foundations)</u>

A tube or well caisson could be used to provide the foundation beneath each bridge pier. During the works on Aricha Power Conveyor where caissons were installed one caisson suffered a blow out during sinking operations, this would be a risk if this type of foundation were adopted for the Padma Bridge.

Well caissons are suitable to carry the loading and distribute into the ground. However the large deflections at bearing level under ship impact load and the works required to install such caissons make them a costly form of construction.

<u>Large diameter caissons</u>

Use of large diameter caissons was considered to check if a shorter caisson height could be used to provide the bridge foundations. For this case it was assumed bearing capacity under the base would resist the vertical loads and horizontal loads would be resisted by base friction. Outline calculations show it is not a suitable foundation unless taken to a considerable depth due to the nature of the supporting ground.

It is considered there are no advantages from this type of caisson as a foundation for the bridge.

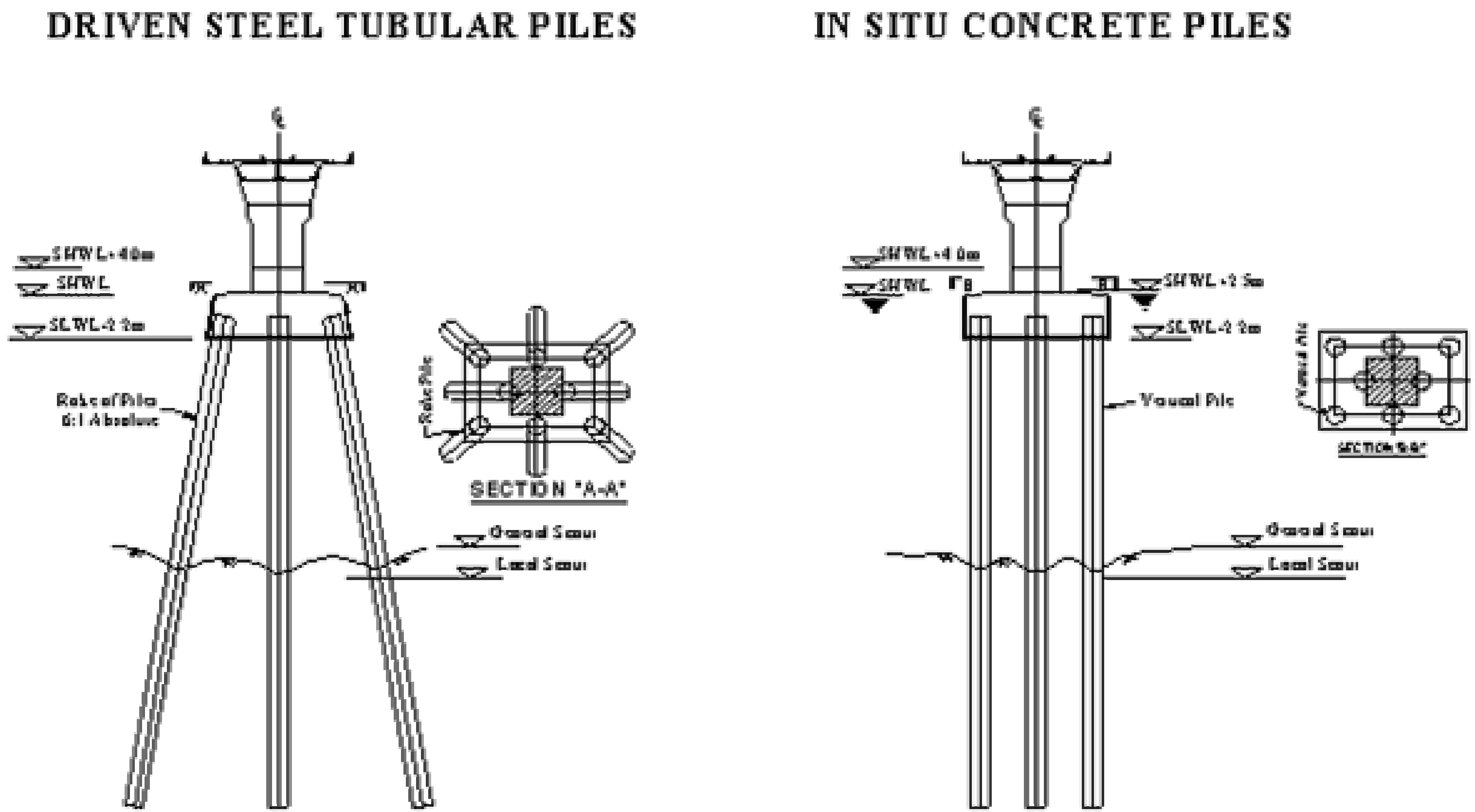


Figure 2.2.5 Driven Steel Tubular Piles and In-Situ Concrete Piles

TUBE (WELL) CAISSON

LARGE DIAMETER CAISSON

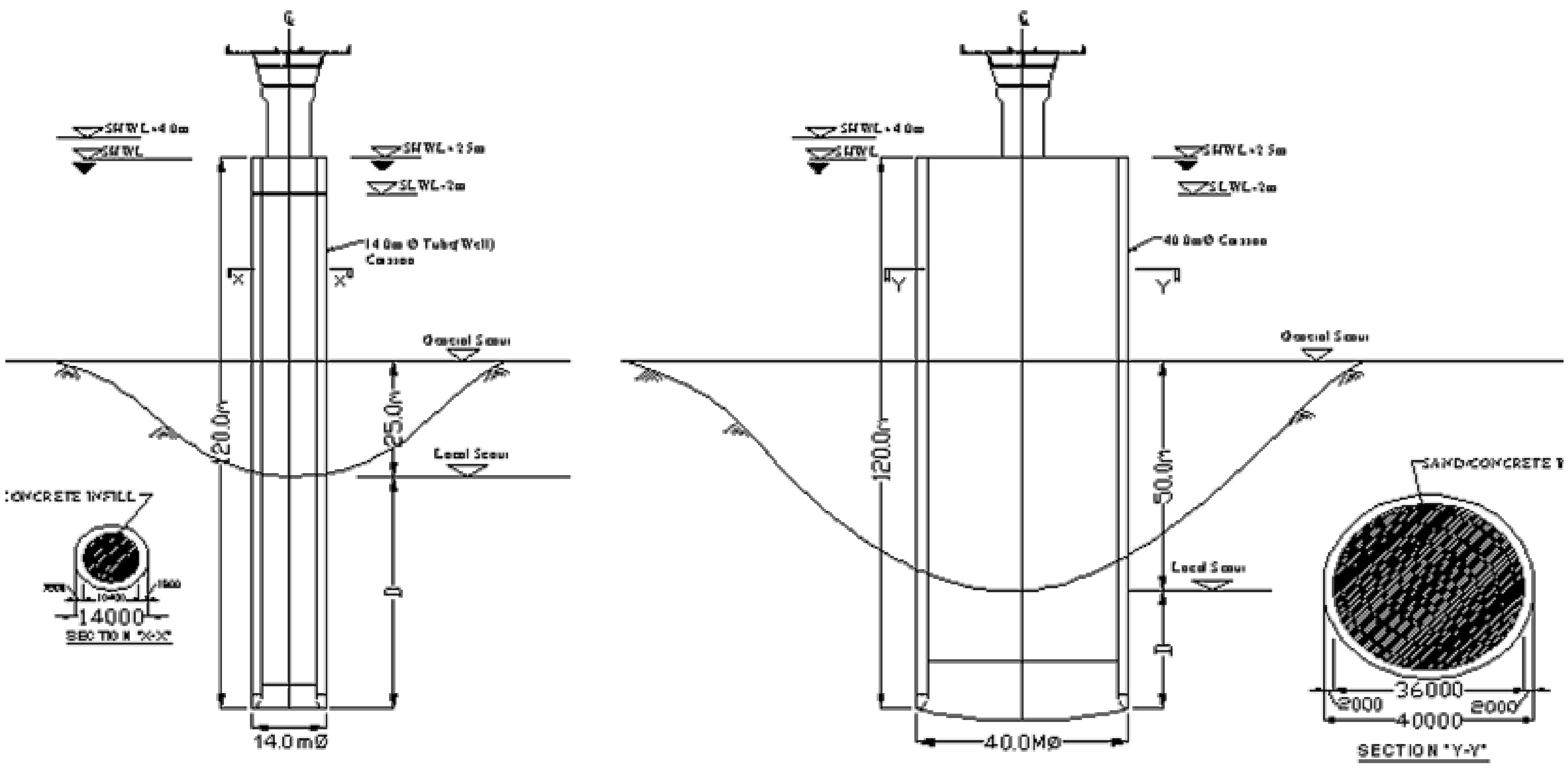


Figure 8.2.6 Tube (Well) Caisson and Large Diameter Caisson

(e) Foundation type in preparatory study

As a result of the examination of each type described above, foundation by "Driven Steel Tubular Piles" is considered as representing foundation type for the selection of a final bridge site.

2.2.4 Selection of Target Ranges of Span Lengths

(1) Methodology

In order to grasp a tendency of relation between span lengths and the construction costs, outline estimates were carried out for both concrete and steel bridges.

The methodology taken for the estimate is as follows.

1) The span ranges from 100m through 250m. Estimate was made in increments of 25m within the above range, in other word, seven cases of 100, 125, 150, 175, 200, 225 and 250m for both materials.
2) Cross Section
   The cross-section used for selecting a final bridge site is shown by the Figure 8.2.7, which was proposed by the Pre-feasibility Study of the Padma Bridge as 4-lane configuration. This cross Section might be changed after review in the course of preliminary design stage.

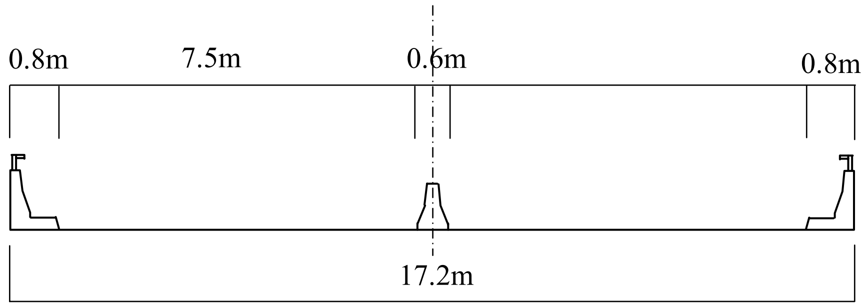


Figure 2.2.7 4-Lane Cross-section

3) Superstructure types for each span of each material were taken as shown below, which are based on the Table 2.2.5.

PC Superstructures

Type	Continuous Box		Extradosed		Cable Stayed Girder		
Span (m)	100	125	150	175	200	225	250

Steel Superstructures

Type	Steel Deck Continuous Box			Cable Stayed Girder			
Span (m)	100	125	150	175	200	225	250

Typical views of Continuous Box Girder Bridge, Extradosed Girder Bridge and Cable Stayed Girder Bridge are shown in the Figure 2.2.8 as samples for span length of 150m.

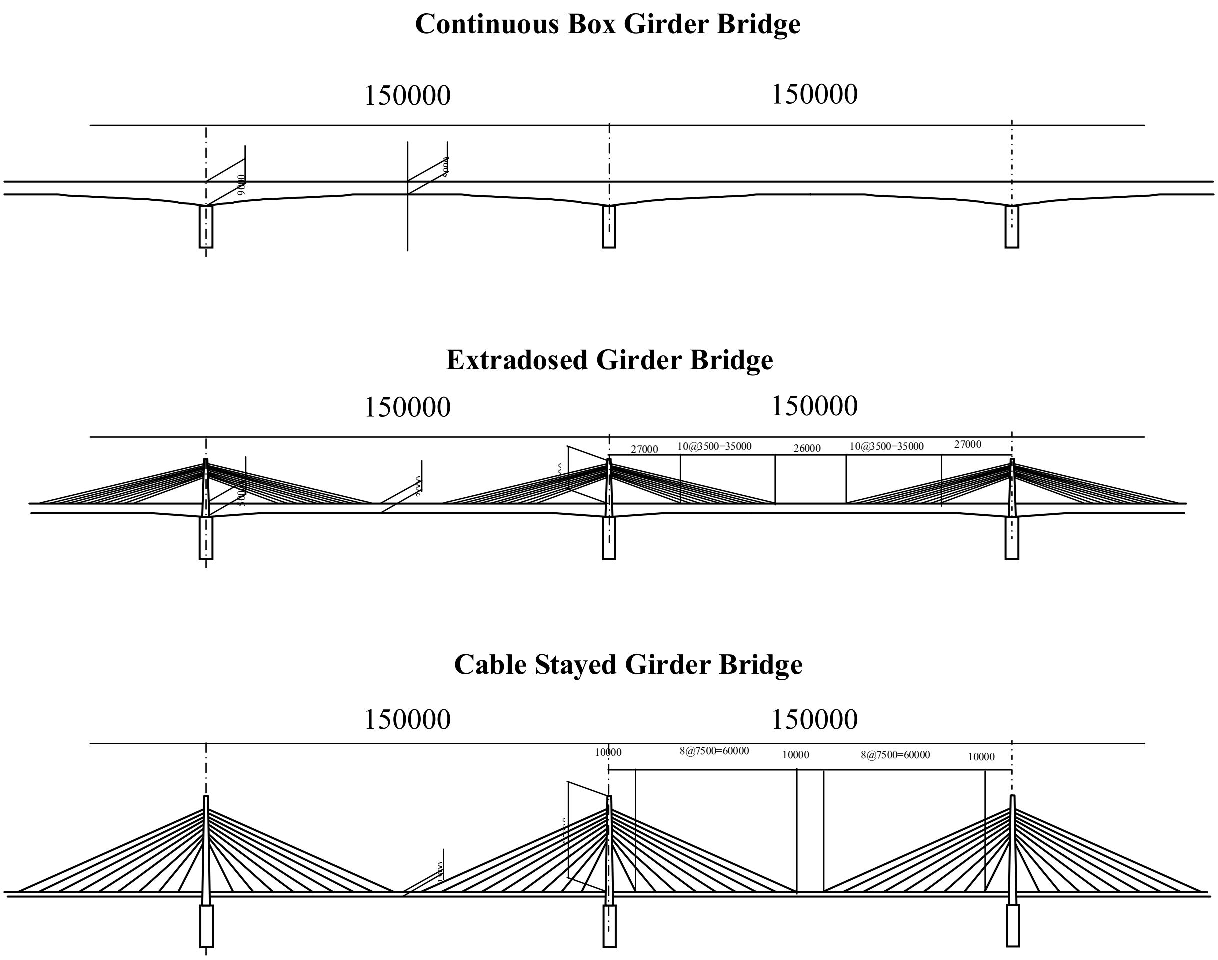
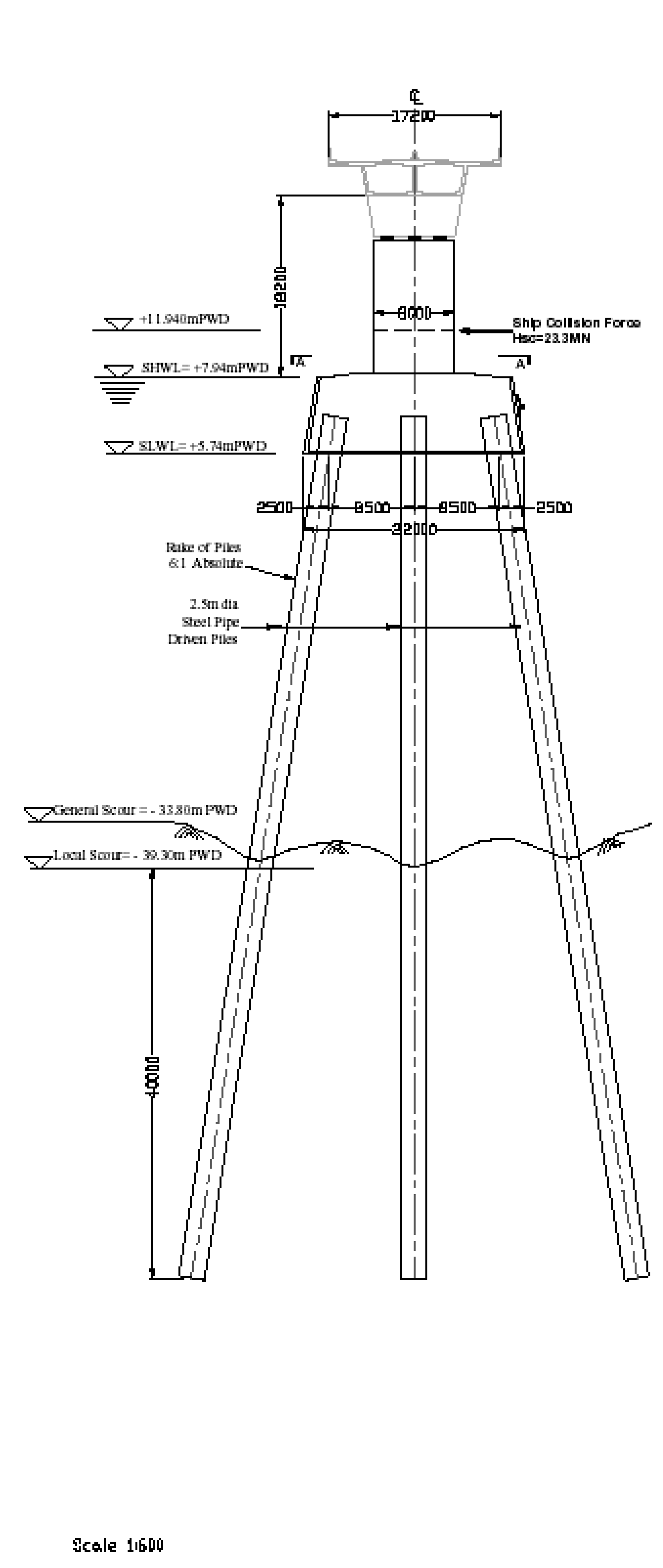


Figure 2.2.8 Side Views by Bridge Types

4) Substructure/foundations for estimating indicative costs in selecting a final bridge site were as shown in Figure 2.2.9 and 2.2.10.
5) The indicative construction costs consist of direct cost for constructing the superstructure and the substructure.

PADMA BRIDGE AT PATURIA-GOALUNDO
SUBSTRUCTURE - RAKING (STEEL) PILE



External Forces developed by Prestressed Concrete Type Bridges — Unit=tf & m

	Total Vertical Force transmitted to Pilecap Bottom	Horizontal Force: Ship Collision Force	Moment transmitted to Pilecap Bottom	Pile Nos.
PC.Box L=100m	11540	2350	14570	12
PC.Box L=125m	13210	2350	14570	12
PC.Extradose L=150m	15467	2350	14570	12
PC.Extradose L=175m	17565	2350	14570	13
PC.Cable Stay L=200m	18623	2350	14570	13
PC.Cable Stay L=225m	20860	2350	14570	14
PC.Cable Stay L=250m	23605	2350	14570	15

External Forces developed by Steel Type Bridges — Unit=tf & m

	Total Vertical Force transmitted to Pilecap Bottom	Horizontal Force: Ship Collision Force	Moment transmitted to Pilecap Bottom	Pile Nos.
St.Box L=100m	8769	2350	14570	12
St.Box L=125m	9418	2350	14570	12
St.Box L=150m	10145	2350	14570	12
Cable Stay L=175m	10651	2350	14570	12
Cable Stay L=200m	10786	2350	14570	13
Cable Stay L=225m	11493	2350	14570	13
Cable Stay L=250m	12193	2350	14570	14

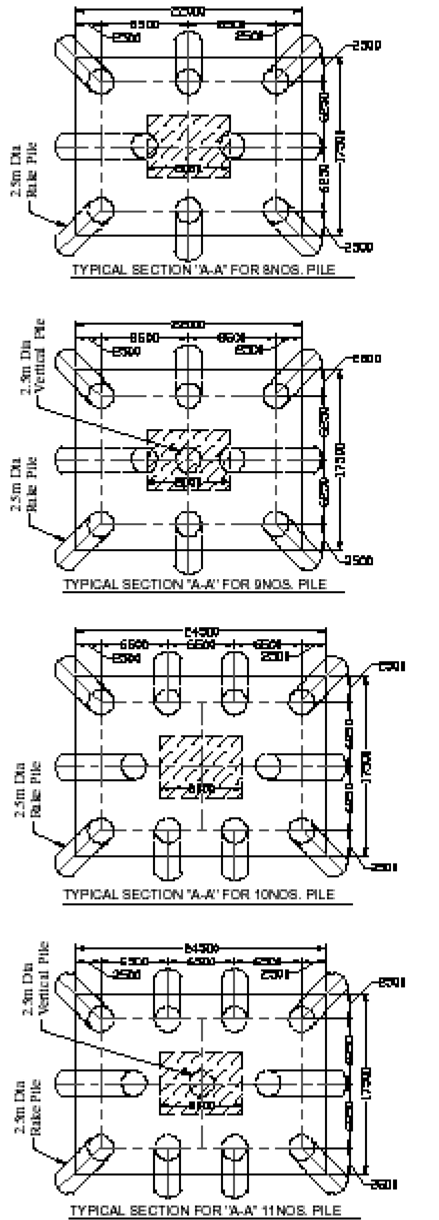


Figure 2.2.9 Paturia – Goalundo Site, Substructure with Raking Piles

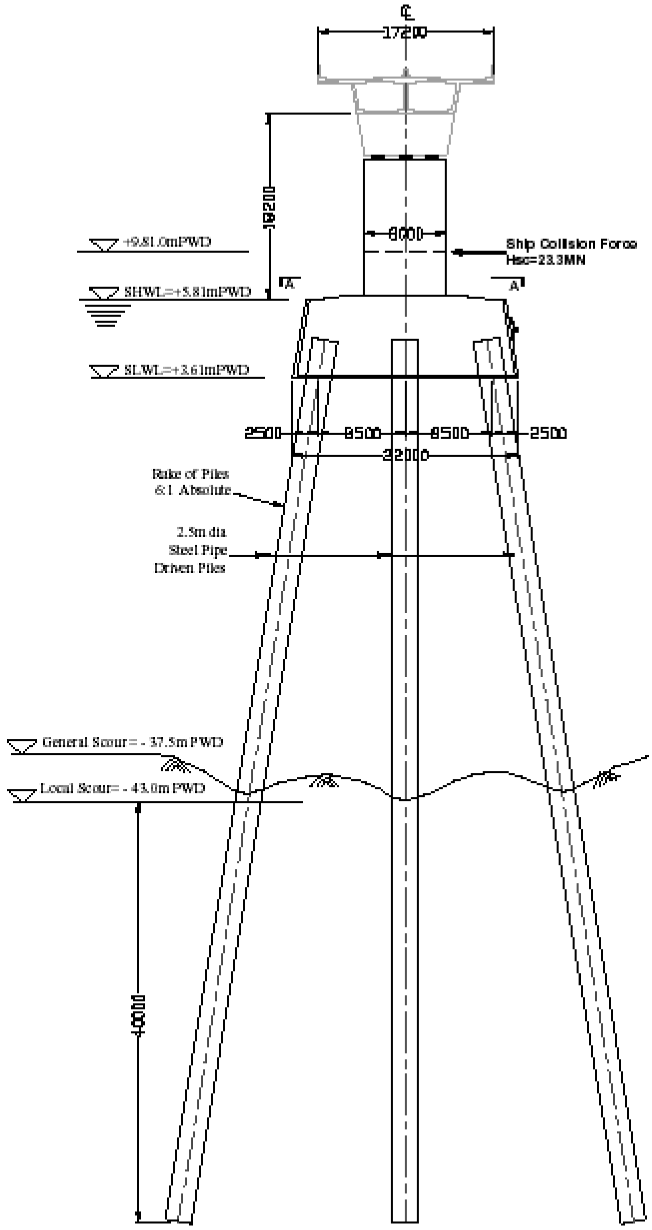
PADMA BRIDGE AT MAWA-JANJIRA
SUBSTRUCTURE - RAKING (STEEL) PILE

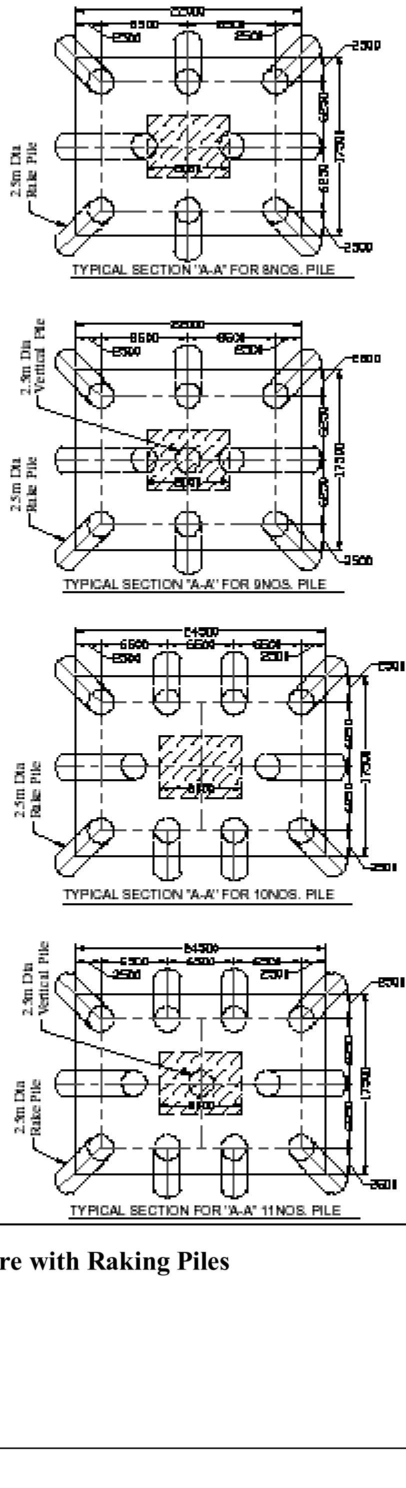
External Forces developed by Prestressed Concrete Type Bridges Unit=tf & m

	Total Vertical Force transmitted to Pilecap Bottom	Horizontal Force: Ship Collision Force	Moment transmitted to Pilecap Bottom	Pile Nos.
PC.Box L=100m	11540	2350	14570	10
PC.Box L=125m	13210	2350	14570	10
PC.Extradose L=150m	15467	2350	14570	10
PC.Extradose L=175m	17565	2350	14570	9
PC.Cable Stay L=200m	18623	2350	14570	9
PC.Cable Stay L=225m	20860	2350	14570	11
PC.Cable Stay L=250m	23605	2350	14570	12

External Forces developed by Steel Type Bridges Unit=tf & m

	Total Vertical Force transmitted to Pilecap Bottom	Horizontal Force: Ship Collision Force	Moment transmitted to Pilecap Bottom	Pile Nos.
St.Box L=100m	8769	2350	14570	12
St.Box L=125m	9418	2350	14570	12
St.Box L=150m	10145	2350	14570	11
Cable Stay L=175m	10651	2350	14570	11
Cable Stay L=200m	10786	2350	14570	11
Cable Stay L=225m	11493	2350	14570	11
Cable Stay L=250m	12193	2350	14570	12





Scale 1:600

Figure 2.2.10 Mawa – Janjira Site, Substructure with Raking Piles

(2) Results of Indicative Bridge Costs

Costs per meter including superstructures, piers and foundations were estimated based on the following for grasping the indicative bridge costs.

- Quantities of superstructures, substructures and foundations are estimated in the preceding sub-paragraphs.
- Unit prices of materials such as concrete, PC tendons, re-bars, etc. are based on those experienced in Paksey Bridge, and that of steel tubular driven piles is based on the Jamuna Bridge.

The following 2 cases (PC and Steel bridges) were conducted for this purpose.

1) Prestressed Concrete Bridges
   The following bridge types were considered for the respective span lengths.
   a) Superstructure Type by Span Length
      From 100m to 125m Span: Continuous Box Girder
      From 150m to 175m Span: PC Extradosed Girder
      From 200m to 250m Span: PC Cable Stayed Girder
   b) Pier and Foundation
      Reinforced concrete piers supported by steel tubular driven piles were considered. The quantities of piles were calculated based on the external forces transmitted from the superstructures, ship collision force, and the subsoil data from the geotechnical investigation for the Paturia-Goalundo and Mawa-Janjira sites.

2) Steel Bridges
   a) Superstructure Type by Span Length
      From 100m to 150m Span: Steel Deck Continuous Box Girder
      From 175m to 250m Span: Steel Cable Stayed Girder
   b) Pier and Foundation
      Same as i) b) of the above.

As a result, the relationship between span length and indicative costs per meter has been obtained as shown in Figure 2.2.11. The minimum costs per meter in Figure 2.2.11 will be incorporated in the subsequent Chapter 11 Indicative Cost Estimate.

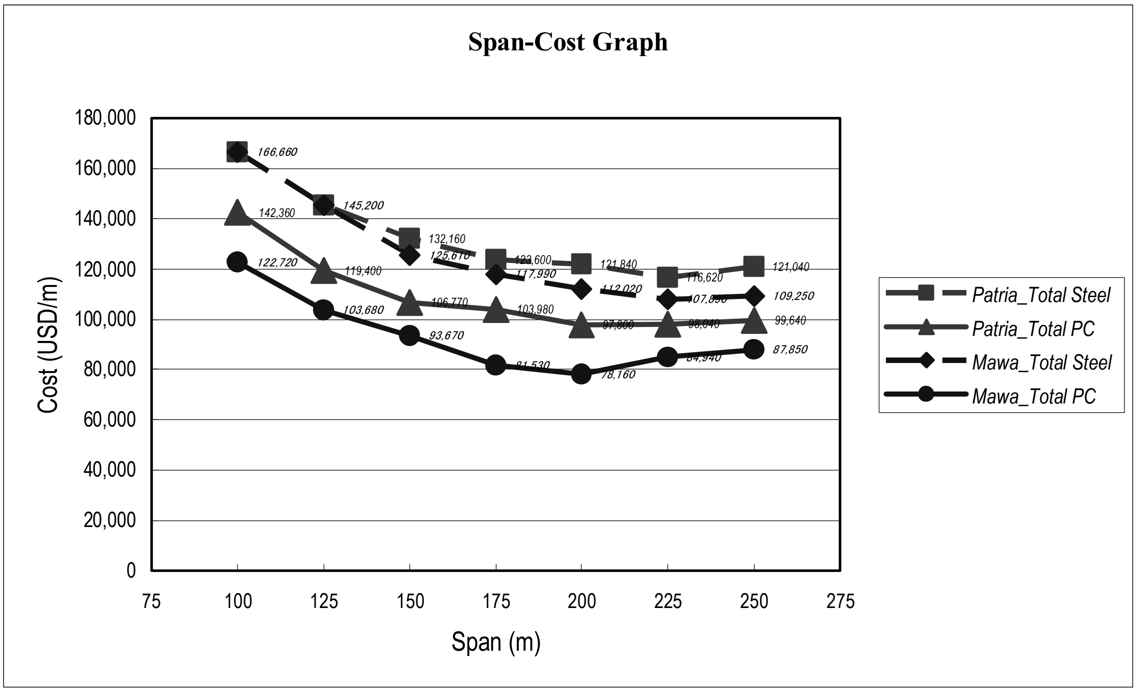


Figure 2.2.11 Span-Cost Relation

2.3 BASIC CONDITION ANALYSIS FOR HIGHWAY DESIGN

2.3.1 Connecting Roads

Site-1 (Paturia)

The Project Highway will be connected to the N5 Highway at the Paturia side and the N7 Highway at the Goalundo side, respectively. Presently, the connecting roads at the assumed connecting sections are two-way, two-lane highways with a fair condition of the surface treatment pavement. The existing road embankment of these highways has 1:1.5 to 1:3.0 slope gradient with the turf or grass protection. The existing ground surface condition is cultivated or swampy land, and the embankment at the connecting sections are assumed to be about 3m to 5 m high above the ground.

Site-3 (Mawa)

The Project Highway will be connected to the N8 Highway on the both river banks (Mawa and Janjira). The N8 Highway is being improved up to an arterial highway with two-lane asphalt concrete surface under a road improvement project financed by ADB. The Project Highway will be connected to this improved N8 project road alignment.

The proposed road structure of the N8 highway project is composed of a 7.3m carriageway plus a 2.7m shoulder and a 1:2.0 embankment slope at both sides. The design height of the N8 embankment near the connecting points will be about 3m to 5 m high above the cultivated or swampy ground in the vicinity.

2.3.2 Control Points

Major Control Points to be considered in the Project Highway design will be as shown in Table 2.3.1.

Table 2.3.1 Major Control Points

Description		Site-1 Payuria-Goalundo		Site-3 Mawa-Janjira	
		Vehicles per day	Composition %	Vehicles per day	Composition %
Year 2015	Light Vehicle	2,410	23.4	3,850	18.1
	Bus	4,880	48.7	13,210	62.1
	Truck	3,010	28.0	4,200	19.8
	Total	10,300	100.0	21,260	100.0
Year 2025	Light Vehicle	4,610	23.2	7,340	17.7
	Bus	9,920	50.0	26,750	64.4
	Truck	5,320	26.8	7,460	18.0
	Total	19,850	100.0	41,550	100.0

2.3.3 Future Traffic

Future traffic in the year of 2015 and 2025 is estimated by the Study Team as follows:

Table 2.3.2 Future Traffic Forecast

Description		Site-1	Site-3
Existing Objects	. Existing Road Network	N5andN7、other minor roads	N8(ADB project)、other minor road
	2. Existing Ferry Route	Tentative alignment assumed downstream of the ferry route	Tentative alignment assumed downstream of the ferry route
	3. Inland river and canal	Existing bridges、culverts	Existing bridges、culverts
	4. Existing facilities	Communities、Railway, Public Utilities、	Communities、 Public Utilities、
	5. Ground condition	Soft ground、Swampy area	Soft ground、Swampy area
Assumed Objects	1. River Training Work	Guide bund、River protection	Guide bund、River protection
	4. Miscellaneous	Toll facility、Others	Toll facility、Others

Note: Average Composition ratio 2015－2025 Site-1=Light Vehicle 23.3%, Bus 48.7%, Truck 28.0%
Site-3=Light Vehicle 17.9%, Bus 63.3%, Truck 18.9%

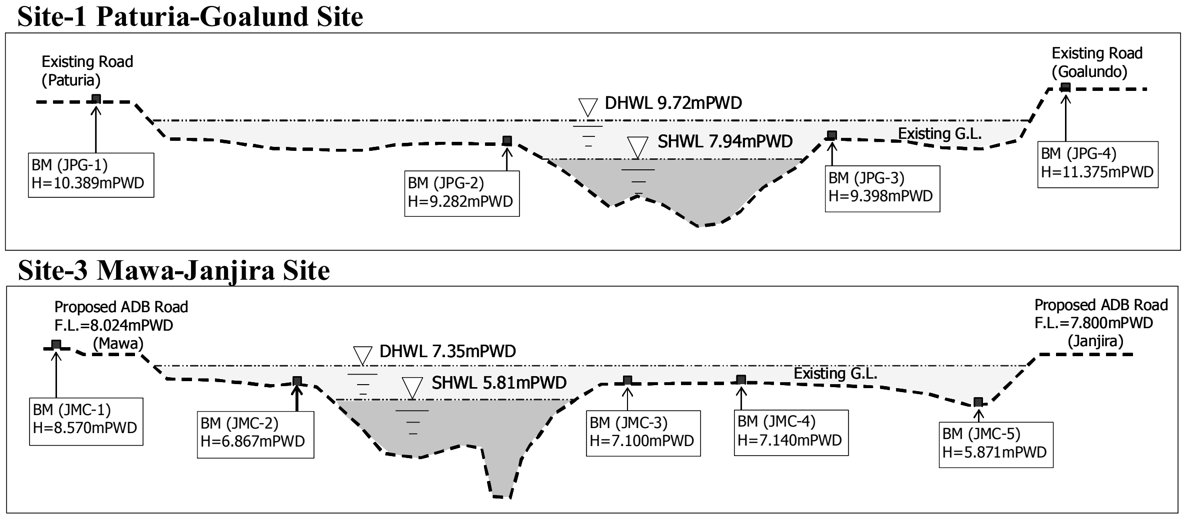
2.3.4 Height of Project Highway

Topographic survey was executed with the Bench Mark establishment along the assumed alignment at the two alternative sites. The survey provides the height of the existing ground and existing/proposed roads for both alternative sites such as follows:

- Site-1: Left bank=10.389m PWD, Right bank =11.375m PWD
- Site-3: Left bank=8.024m PWD, Right bank=7.800m PWD (both ADB project height)

The Design High Water Level (DHWL, 100 years flood level) is set up in the river study of the Study as 9.72m PWD at Site-1 (Paturia) and 7.35m PWD at Site-3 (Mawa) respectively.

As a result of comparison of the existing road level with DHWL, it is found out that the existing roads of both alternative sites (Site-1, 3) are higher than DHWL and this leads to an adequate fulfillment of the required condition for the connecting point between the approach road and the existing road.



Note: Not to Scale
DHWL=Design High Water Level, SHWL=Standard High Water Level , BM=Bench Mark

Figure 2.3.1 Project Height

2.3.5 Geometric Design Conditions

A primary concern to be considered in the course of the Project Highway design would be the selection of applicable design standards. Considering the Bangladesh and other standards, the applicable design criteria and parameters are summarized as shown in Table 2.3.3. Accordingly, the typical cross section of the approach road is shown in Figure 2.3.2.

Table 2.3.3 Proposed Design Condition

Description			Specification
1. Design Speed			80km/hr
2. Curve	Horizontal	Minimum (Absolute Min.)	R=400(280)
		Transition curve Omission	More than Radius 2,000m
	Vertical	Recommend for Crest (Min.)	R=4,500(3,000)
		Recommend for Sag (Min.)	R=3,000(2,000)
3. Longitudinal gradient		Max.	4%
4. Road Cross Section		Crest width	21.6m (RHD Standard Design Type-2)
		Carriageway	7.3m (3.65m x 2)
		Median	1.6m
		Shoulder	2.7m (1.8m hard shoulder + 0.9m turf protection)
5. Slope gradient			1:3

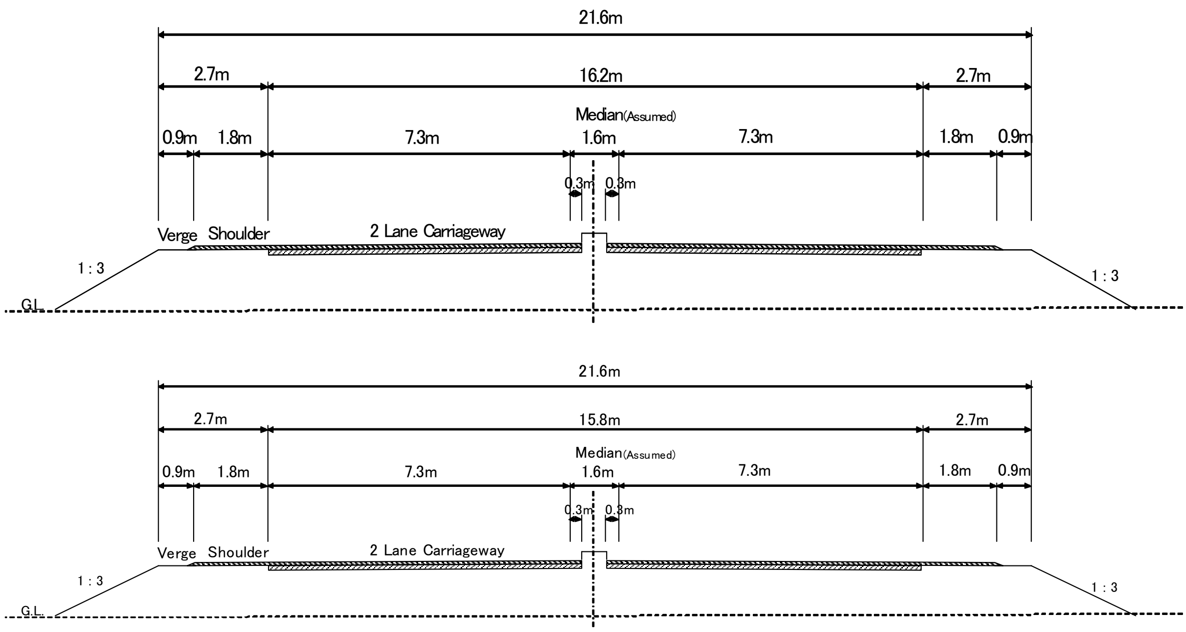


Figure 2.3.2 Approach Road Typical Cross Section

2.3.6 Land Acquisition Area

The initial evaluation of the land acquisition area for the Project HIghway is carried outconducted along the assumed alignment.

The width of the land to be acquired is set to be width is 60m which is composed of a 45m of the physical projection area of the approach road and a 15m of the complementaryadditional area for construction.

The physical projection area comprises a 21.6 m of the approach road crest width and a 24 m of the slope of both sides width (4.0m of assumed average embankment height and 1:3 slope gradient is used for the calculation).

2.3.7 Initial Inventory Study

The initial inventory study was carried out for tentatively confirming the objects to be considered (inland rivers, railways, village roads, houses, etc.) along the assumed alignment of the two alternative sites. The topographic survey route was used for this inventory study in order to calculate the assumed unit volume of major items which can be used for the examination and comparison of alternative alignment.

The results of this inventory study are as follows:

(1) Paturia-Goalundo Site

a) Precondition
   1. Calculation made along the tentative route surveyed for the Initial Inventory Study.
   2. Length of Approach Road Estimated: Left bank= 9.2km, Right bank= 3.4km, Total= 12.6km
   3. Land Acquisition Width: 60m (Assumed value)

b) Land Acquisition Area

Description	Bridge Approach Road						Borrow Pit	Total
	Paddy field	Farm Area	Road	School	House	Total		
Land acquisition area (ha)	0.0	69.7	0.4	0.1	2.9	73.1	36.9	110.0

Note: Borrow Pit (BP) area needed to cover 70% of fill material by 1.5m excavation of BP area (Dredged sand can be used as remaining 30%)

c) Major Structures

Description	Quantity (Nos.)	Remarks
Box culvert	9	7x5type= 3Nos., 5x5type= 6Nos. for Village Road Grade Separation Crossing
Minor bridge	3	Total Length=460m (2Nos. for inland minor river/canal crossing, 1No. for Railway Grade Separation Crossing(10m))
Toll facility	2	for Each direction

d) Earthworks

Description	Quantity (m3)	Remarks
Embankment	600,000	Assumed Embankment Height=4.0m
Replacement	190,000	Assumed unsuitable material =20% total embankment area with 1.5m depth.
Total	790,000	63,000m3/km

(2) Mawa-Janjira Site

a) Precondition

1. Calculation made along the tentative route surveyed for the Initial Inventory Study.
2. Length of Approach Road Estimated: Left bank= 6.8km, Right bank= 12.6km, Total= 19.4km
3. Land Acquisition Width: 60m (Assumed value)

b) Land Acquisition Area (W=60m、assumed)

Description	Bridge Approach Road						Borrow Pit	Total
	Paddy field	Farm Area	Road	School	House	Total		
Land acquisition area (ha)	37.9	67.9	1.0	0.4	7.5	114.7	57.4	172.1

Note: Borrow Pit (BP) area needed to cover 70% of fill material by 1.5m excavation of BP area (Dredged sand can be used as remaining 30%)

c) Major Structures

Description	Quantity (Nos.)	Remarks
Box culvert	22	10x5Type=2Nos., 7x5type=4Nos., 5x5type=16Nos. for Village Road Grade Separation Crossing
Minor bridge	7	Total Length=371m (for inland minor river/canal crossing)
Toll facility	2	for Each direction

d) Earthworks

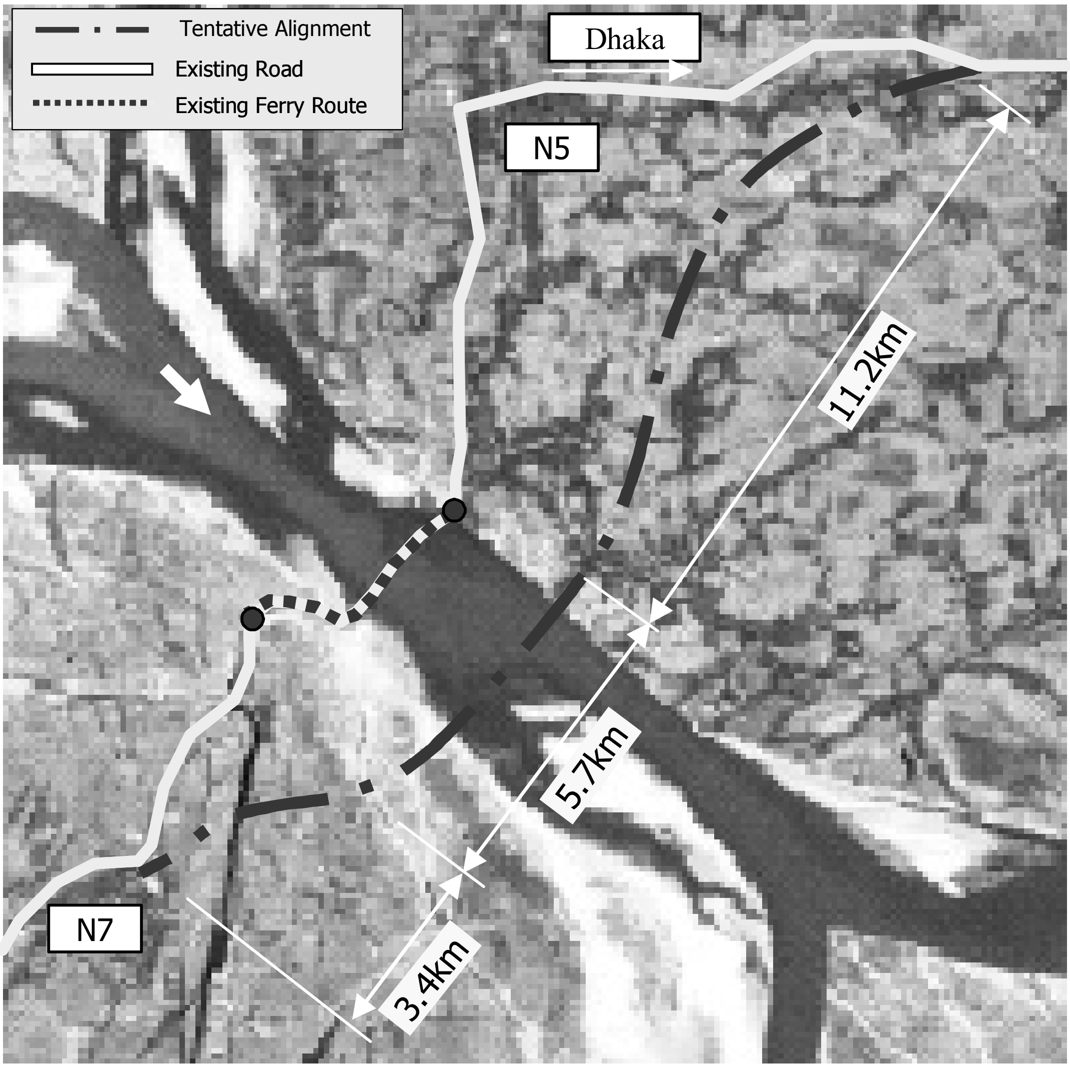
Description	Quantity (m3)	Remarks
Embankment	940,000	Assumed Embankment Height=4.0m
Replacement	290,000	Assumed unsuitable material =20% total embankment area with 1.5m depth.
Total	1,230,000	63,000m3/km

2.3.8 Potential Highway Route for Alternatives Project Locations

Based on the site conditions and the results of the initial inventory study, the features of the alignment of the two alternative project sites are summarized. Generally, no big difference is observed between the two alternative sites in terms of highway design.

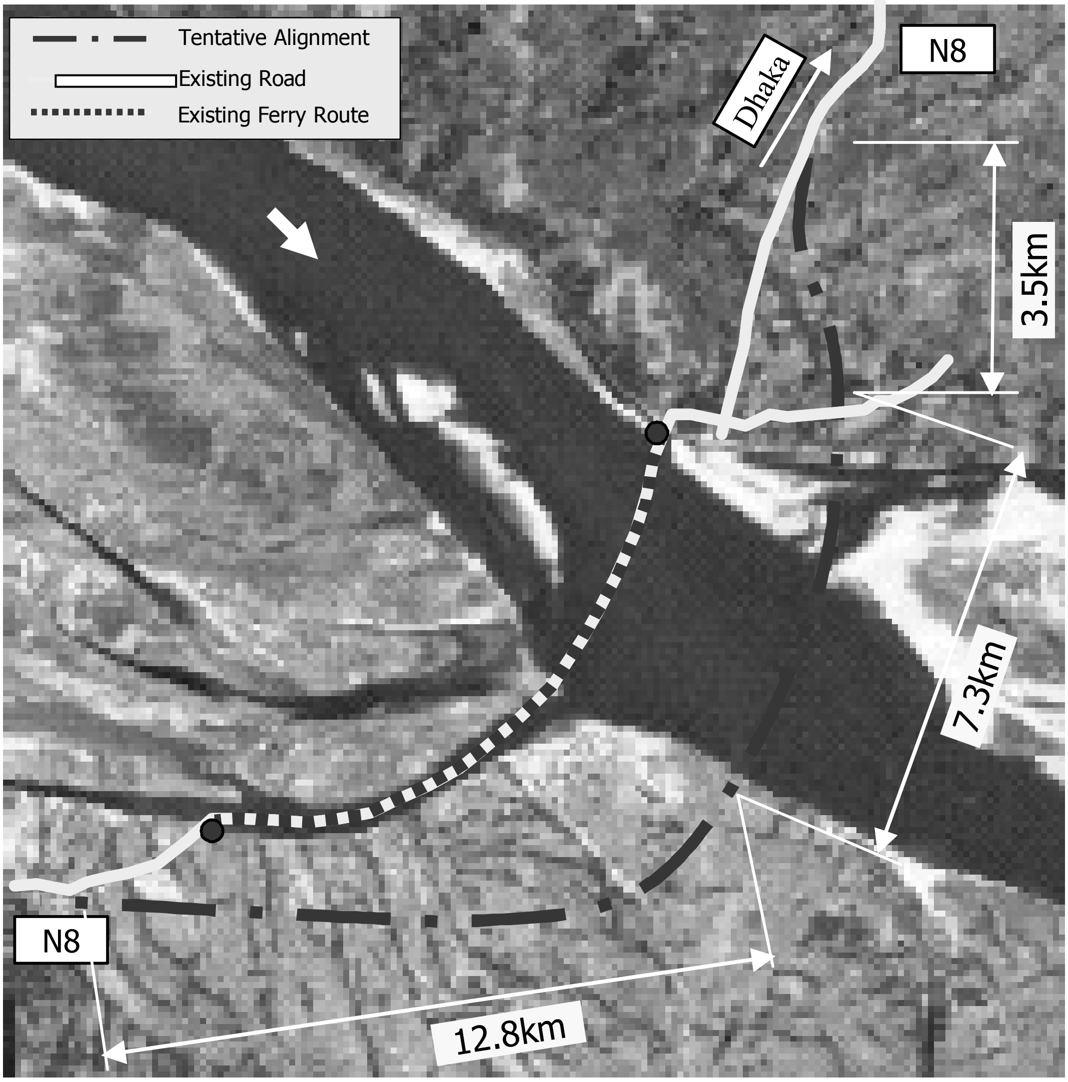
(1) Paturia-Goalundo alternative alignment plan (PG-2)

Description			Quantity	Remarks
1. Length	Bridge		5.7km	
	Approach Roads	Left bank	11.2km	Paturia side
		Right bank	3.4km	Goalundo side
		Subtotal	14.6km	
	Total		20.3km	
2. Number of Lane			4	
3. Slope gradient			1:3	RHD Standard
4. Land acquisition area (expected)			128ha	Road 85ha＋Borrow pit 43ha
5. Major Structures		Box culvert	9 Nos.	Grade crossing with minor road
		Minor bridge	3 Nos.	Length 460m（Inland river, Railway）
		Toll facility	2 Nos.	



(2) Mawa-Janjira alternative alignment plan (MJ-2)

Description			Quantity	Remarks
1.Length	Bridge		7.3km	
	Approach Roads	Left bank	3.5km	Mawa side
		Right bank	12.8km	Janjira side
		Subtotal	16.3km	
	Total		23.6km	
2.Number of Lane			4	
3.Slope gradient			1:3	RHD Standard
4. Land acquisition area (expected)			144ha	Road 96ha＋Borrow pit 48ha
5.Major Structures		Box culvert	22 Nos.	Grade crossing with minor road
		Minor bridge	7 Nos.	Length 371m（Inland river）
		Toll facility	2 Nos.	



2.3.9 Initial Pavement Design

The initial pavement design is carried out as initial stage examination for the Bridge Approach Road (Table 18-4). In the examination, the design traffic load per lane is calculated based on the traffic volumes estimated in this Study, the design pavement thickness is obtained by the AASHTO method.

Table 2.3.4 Initial Pavement Design

Description	Specification	
(1) Design Period	10 Years（2015-2025）	
(2) Design Traffic Volume	Paturia = 58.65Mill. Vehicle/10Years、Mawa = 122.03Mill. Vehicle/10Years	
(3) Traffic composition	Paturia Truck28.0%,, Bus48.7%、Mawa Truck18.9%,, Bus63.3%	
(4) Design Axle Load	Truck(2Axle) 8.42ton、Bus (2Axle) 6.85ton	
(5) Lane destribution	120%	
(6) Design Load（ESAL）	Paturia = 19.34Mill. ESAL/Lane、Mawa = 38.02Mill. ESAL/Lane	
(7) Pavement Material	* Asphalt Concrete: 5cm Wearing Course + 10cm Binder Course * Base: Crushed stone base (CBR 80 equivalent) * Sub Base: Glandular material (CBR 30 equivalent) * Sub Grade: Selected material (CBR 8, assumed)	
(8) Design Pavement Thickness	Site-1 (Paturia)	Surface: 15cm Base Course: 21cm Sub Base Course: 26cm
	Site-3 (Mawa)	Surface: 15cm Base Course: 23cm Sub Base Course: 34cm

Note: 1. (1), (2), (3) above = Total traffic volume will is calculated based on the estimated traffic volume (Traffic volume and composition %) of 2015 and 2025 on the Progress Report-1, and this result can cover between 2015 to 2025 (10 Year design period).

2. The data of 1990 Road Master Plan, ADB was used for the value of (4) above.

Chapter 3 Preliminary Design of Padma Bridge

3.1 INTRODUCTION

Preliminary design of Padma Bridge was conducted in such a manner as described below.

1) Based on the data collected so far, design criteria and standards were adopted for the preliminary design of Padma Bridge.
2) Prior to the preliminary design, it was reconfirmed that steel bridges were more costly than those of concrete.
3) Furthermore an extradosed girder bridge of 180 meters long span is estimated to be of the least cost.
4) Consequently the preliminary design of Padma Bridge was carried with cross-sections of the base case (without railway provision), the railway provision case, and additionally the minimum investment case with the narrowest cross-section.
5) Driven steel tubular piles were selected among other alternatives like in-situ concrete piles, tube caisson and large diameter caisson.
6) Five alternatives of longitudinal configuration to cross the Padma River were considered basically arranging 180 meters extradosed bridges and partially cable stayed girder bridge.
7) Finally quantities of the materials were worked out for the cost estimates.

3.2 DESIGN CRITERIA AND STANDARDS ADOPTED IN PRELIMINARY DESIGN

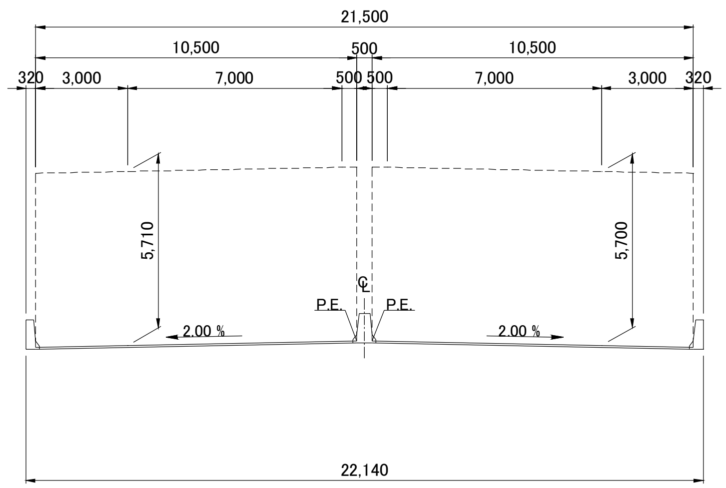
3.2.1 Pre-condition of Preliminary Design

(1) Railway Provision

The feasibility of the Padma Bridge was confirmed as a highway bridge through the economic evaluation in the Interim Report. In this regard, the Study would have to examine a possibility to include the provision of a broad gauge railway as an alternative design according to the agreed scope of works and minutes of meeting signed by the Government of Bangladesh and JICA on December 4, 2001.

(2) Bridge Width

The Padma Bridge would be built on the Asian Highway (AH) Route No. A-1 that is planned under UNESCAP. The Study Team has examined the standard bridge widths with that stipulated in AH standard by UNESCAP and with Bangladesh highway standard by RHD. As the width requirement of AH standard is larger than RHD, the preliminary design would be made based on AH standard as shown below.



Note: The effective width of the RHD Standard is 21.6m, which is very similar to that of the above Figure 3.2.1.

Figure 3.2.1 AH Standard without Railway Provision

Initial Stage

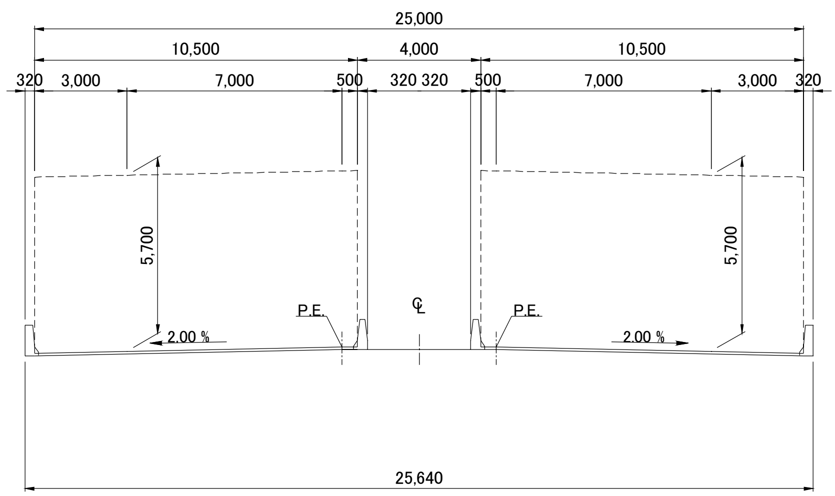


Figure 3.2.2.a AH Standard (Initial Stage with Railway Provision)

Final Stage

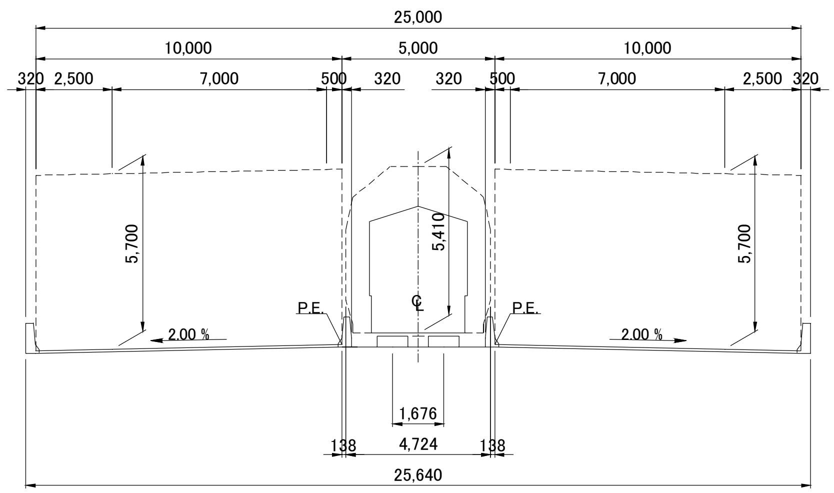


Figure 3.2.2.b AH Standard (Final Stage with Railway Provision)

In addition to the above bridge widths on the basis of AH and RHD standards, one more option will be examined to implicate the minimum investment case as shown Figure 3.2.3 for further economic and financial evaluations. This option has carriageway width of 7.3 m specified in RHD Standard and minimum side belt of 0.5 m on each edge side.

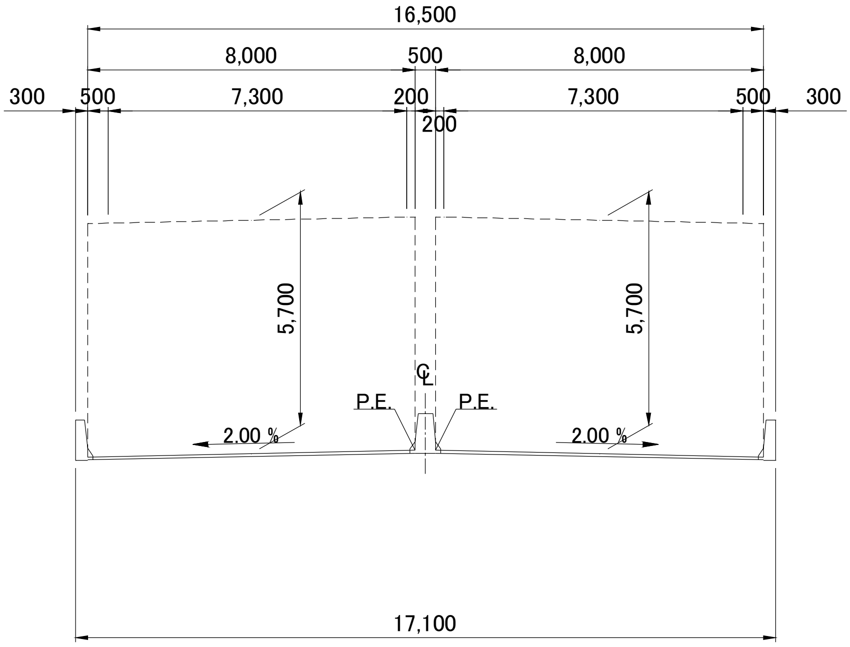


Figure 3.2.3 Minimum Investment Case

3.2.2 Design Criteria and Standards

The preliminary design in the Study would be conducted on the basis of AH standard and RHD standards, the latter is similar to AASHTO standards. Standards adopted by Japan Road Association (JRA), BSI and Indian Road Congress (IRC) would be used as supplementary ones.

(1) Design Loads

(a) Dead Load

The following unit weights are to be used for the preliminary design.

Table 3.2.1 Unit Weight by Structural Items

Items	Unit Weight
Steel, cast steel	77.0 kN/m^3
Aluminum	27.5 kN/m^3
Reinforced concrete	24.5 kN/m^3
Plain concrete	23.0 kN/m^3
Asphalt pavement	22.5 kN/m^3
Electric power transmission line and supports	13.0 kN/m^3
Gas pipeline and supports	7.0 kN/m^3
Telecom fiber optics	Not significant

(b) Live Load

According to AH standard and RHD standards, HS 20-44 stipulated by AASHTO standards is to be used.

(c) Impact

Impact effect is calculated based on the following:

$$\text{Impact fraction: } I = \frac{15}{L + 38}$$

In which,

I = impact fraction (Max. 30 %)
L = Length in meter of the portion of the span that is loaded to produce the maximum stress in the member.

(d) Longitudinal Forces

Not significant.

(e) Centrifugal Forces

Not significant.

(f) Thermal Effect

Referring to AASHTO sixteen edition, 1996, the following temperatures are considered.

Temperature Rise: 17 ℃
Temperature Fall: 22 ℃

(g) Earthquake Effect

The following statically equivalent seismic force will be considered based on the report "Preliminary Study of Seismic Design Parameters for Padma Bridge Corridor (Aricha-Goalundo, Mawa)" by Department of Civil Engineering, Bureau of Research, Testing and Consultation, BUET as per Appendix B of the Prefeasibility Report, Padma Bridge Study Phase I, February 2000, RPT, Nedeco & BCL.

Horizontally Accelleration Coefficient = 0.125 g

Where,

g = gravity acceleration (m/sec2)

Elastic seismic response coefficient for single mode analysis Cs is given by the following dimensionless formula according to AASHTO Sixteen Eddition:

$$Cs = \frac{1.2AS}{T^{2/3}}$$

Where,

A = Acceleration coefficient : 0.125
S = Soil profile characteristics at site: 1.5 (Soil Profile Type III)
T = Period of bridge

The value of Cs need not exceed 2.5 A (= 0.3125).

(h) Wind Loads

The Study Team collected the wind records of 1964 to 2003 from Bangladesh Meteorological Survey Department and produced Figure 3.2.4 for Faridpur observatory and Figure 3.2.5 for Dhaka observatory. Wind velocity of 95 knots (48.9 m/sec) was recorded in October 1964 and May 1972 at Faridpur observatory, which is bigger than AASHTO base wind velocity of 100 miles/hour (44.7 m/sec).

Power Grid Company of Bangladesh has been adopting the design wind velocity of 44.4 m/sec to 70 m/sec. In case of the design of electric power transmission line of the Jamuna Bridge, wind velocity of 70 m/sec was used for electric pole design while 62.6 m/sec was for overhead type wire.

So far taking into actually recorded maximum wind velocity in Faridpur, the following wind velocity and increased ratio of wind load are to be used for the preliminary design of the Padma Bridge.

Wind velocity = 95 knot = 50 m/sec

$$\text{Increased ratio} = \frac{95\text{knot}^2}{100\text{mile}^2} = 1.2$$

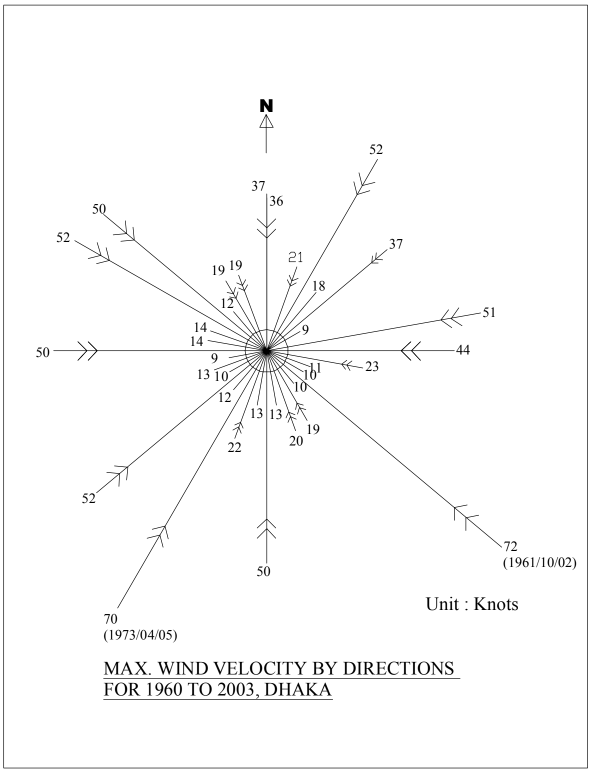


Figure 3.2.4 Maximum Wind Velocity by Directions for the Period of 1960 to 2003, Dhaka

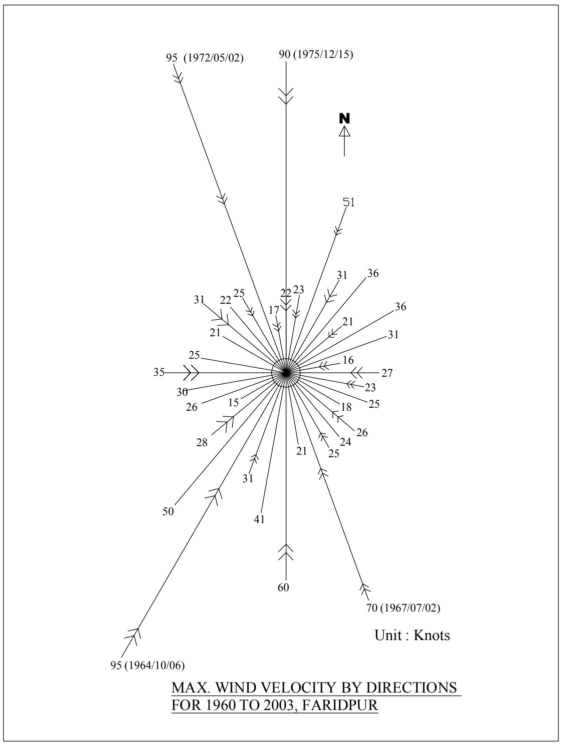


Figure 3.2.5 Maximum Wind Velocity by Directions for the Period of 1960 to 2003, Faridpur

(i) Significant Wave Height

From S-M-B monogram, the significant wave height is obtained in the following when the wind velocity is 95 knots and fetch is 25 km.

Significant Wave Height: $H_{1/3}$ = 5.5 m

(j) Ship Collision Force

As a result of discussions with BIWTA, the following tonnage and speed are to be considered for the calculation of ship collision force against piers located in the navigable course.

Dead Weight Tonnage = maximum 1,500 ton

Vessel Speed = maximum 10.0 knots = 16.9 fps

Impact Level of Ship collision Force = SHWL + 9.0 m

The following formula is to be applied based on Guide Specification and Commentary for Vessel Collision Design of Highway Bridges Volume I: Final Report, February 1991 by AASHTO.

$$Ps = 220(DWT)^{1/2} \left[\frac{V}{27}\right]$$
$$= 5{,}333 \text{ kips} = 23.7 \text{ MN}$$

Where,

Ps = equivalent static ship impact force (kips)

DWT = deadweight tonnage of ship (tones)
V = ship impact speed (fps)

(k) Parameters related to River Engineering

Water Levels

Design High Water Level (100 year return period):DHWL = +7.35 m PWD
Standard High Water Level: SHWL = +5.81 m PWD
Mean Water Level: MWL = +3.02 m PWD
Standard Low Water Level: SLWL = +1.43 m PWD

Water Discharge and Velocity

Design Discharge (100 year return period): Qd = 134,400 m^3/sec
Design Water Velocity (100 year return period): Vd = 4.6 m/sec

Scour Levels

Adjacent to Riverbank (100 year return period)
300m from Riverbank: Zs = -37.56 m PWD
In Middle of River (100 year return period): Zs = -23.63 m PWD

3.2.3 Review on Bridge Types, Span-Cost Relation of Interim Report

(1) Reasons for Review

- Bridge width was changed from 17.2m in the Interim Report to 21.8m in this report, so the span-cost relation of the Interim Report would have to be refined.
- In the Interim Report, foundation stability was examined based on the ship collision force 23.3 MN, which is slightly refined 23.7 MN in this report. Foundation stability in this report would have to be calculated on both the ship collision and earthquake cases as per subsection 3.2.2 of this report.
- In the Interim Report, current market prices in Japan were used for estimating the indicative cost of steel girder bridge (US$ 5,000 /ton including materials, fabrication, transport, erection). As the steel girder bridge is in a wide rage of prices, it would have to be re-estimated taking into consideration the possible lower case of internationally prevailing market prices (US$ 3,800 /ton including materials, fabrication, transport, erection).

(2) Revised Bridge Types, Span-Cost Relation

The revised span-cost diagram is shown in the following.

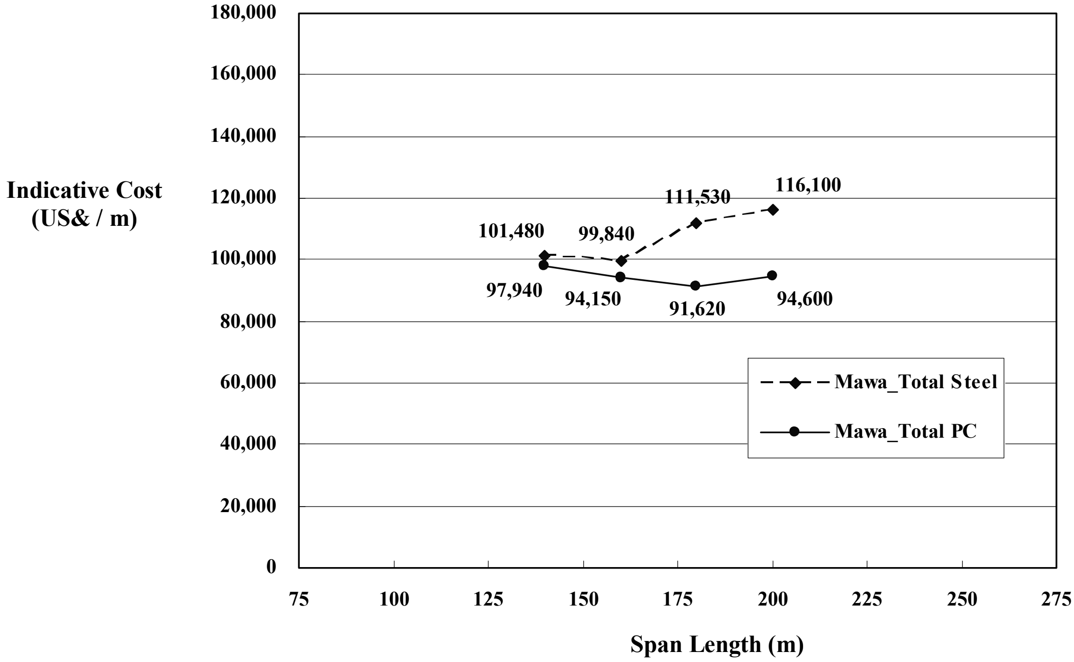


Figure 3.2.6 Revised Span - Cost Relation

From the above figure, the following are concluded.

- In case of prestressed concrete girder bridge, the minimum cost will be about US$ 9,200 /m at the span length of 150 m. to 200 m.
- In case of steel girder bridge, the minimum cost will be about US$ 100,000 /m at the span length from 140 m to 170 m.
- Comparing the indicative minimum cost of steel and PC bridges, cost of steel bridge is higher than PC bridge.
  Consequently, PC bridge would have to be studied in the preliminary design. The details of types, span lengths and costs of the Padma Bridge is discussed in the subsequent section 3.3 in this report.

3.2.4 Bridge Type Alternatives

(1) Specific Bridge Types for Preliminary Design

Bridge types being considered for the subsequent preliminary design are mainly based on the following aspects.

- Less construction cost in structural type and span length
- Better aesthetic view
- Less constriction for river section by bridge piers

(a) Less Construction Cost

In the subsequent Section 3.3 of this chapter, studies are discussed not only on superstructure and foundation types but also on favorable span length. As for superstructure types, continuous PC box girder, PC extradosed girder and PC cable stayed girder bridges are examined. According to the result of Section 3.3, the following are concluded.

i) If PC box girder is employed, favorable span length is about 160 m and approx. cost is US$ 99,010 per meter without railway case.
ii) If PC extradosed girder is employed, favorable span length is about 180 m and approx. cost is US$ 91,620 per meter without railway case.
iii) If PC cable stayed girder is employed, favorable span length is about 200 m and approx. cost is US$ 109,640 per meter without railway case.

In addition, total bridge lengths largely affect the bridge construction cost. The extradosed girder and cable stayed girder bridges are advantageous than continuous PC box girder bridge since the girder depth of the continuous PC box girder bridge is larger than others and requires longer bridge length. The girder depth affects longitudinal alignment, and then larger girder depth requires longer bridge length due to navigational requirements. For example, if span length of each bridge type is assumed at optimum span length, girder depths and total bridge lengths (Padma Bridge + viaducts) will be in Table 3.2.2.

Table 3.2.2 Girder Depths and Bridge Lengths

Bridge Type (Favorable Span Length)	Girder Depth at mid-span	Approx. Bridge Length
PC Box Girder (Span=160 m)	4.8 m	5,680 m
PC Extradosed (Span=180 m)	3.3 m	5,580 m
PC Cable Stayed (Span= 200 m)	1.8 m	5,480 m

From the above, the indicative bridge costs are roughly estimated in the following table.

Table 3.2.3 Indicative Cost by Bridge Type without Railway

Bridge Type (Span)	Unit Cost /m	Bridge Length	Indicative Cost
PC Box Girder (Span=160 m)	US$ 99,010	5,680 m	562 million US$
PC Extradosed (Span=180 m)	US$ 91,620	5,580 m	511 million US$
PC Cable Stayed (Span= 200 m)	US$ 109,640	5,480 m	601 million US$

In terms of construction cost, the PC extradosed girder bridge is regarded as a favorable bridge type.

(b) Aesthetic Aspect

From the aesthetic view points, PC extradosed girder and PC cable stayed girder are advantageous than PC box girder.

(c) Constriction for River Section by Bridge Piers

The width of the Padma River is about 5,300 m. The river section will be reduced by constructing a number of piers. The reduced areas of the river section by piers of the respective bridge type are summarized as follows.

Table 3.2.4 Reduced River Widths by Bridge Types

Bridge Type (Span)	Number of Piers in the River	Pier Width per Each	Reduced River Width	% Reduced River Width
PC Box Girder (Span=160 m)	34	15.0 m	510 m	9.6 %
PC Extradosed (Span=180 m)	31	15.0 m	465 m	8.8 %
PC Cable Stayed (Span= 200 m)	28	16.2 m	454 m	8.6 %

Remarks: Pier width per each = pile cap width of a pier
Present river width in 2004 = approx. 5,300 m

From the above table, PC extradosed girder and PC cable stayed girders are advantageous.

(d) Specific Bridge Types and Alternatives for Preliminary Design

The PC extradosed girder type bridge has been selected for the main bridge structure of the Padma Bridge from the view points of less construction cost, aesthetics and river aspect as discussed above.

In addition, the Bangladesh side requested the Study Team to include an alternative option that has Cable Stayed girder at the center portion of approximately 800 m in length.

As a result of the above, the following five alternatives will be examined in the preliminary design of the Study.

- Alternative - H1 (Base Case): PC extradosed girder bridge without railway provision
- Alternative - H2: PC extradosed girder bridge with PC Cable Stayed Girder at bridge center without railway provision
- Alternative - HR: PC extradosed girder bridge with railway provision
- Alternative - HR2 PC extradosed girder bridge with PC Cable Stayed Girder at bridge center portion with railway provision
- Alternative-H3: (Min. Invest.): PC extradosed girder bridge with Minimum Investment Case

(2) Total Bridge Length

Total bridge length consists of the lengths of the Padma main bridge and viaducts on left (north) bank and right (south) bank sides. Design scour depths will be considered in the Padma main bridge design but not considered in the viaduct design. Major factors to govern these lengths are:

- Navigational requirements such as location and range of navigable course and navigation clearance
- Location of river facilities
- Longitudinal geometry of approach roads and maximum embankment height

(a) Navigational Requirements

After discussions among BIWTA, JMBA and Study Team, navigable course would have to be provided in approximately 4.8 km width taking into consideration the present river characteristics. As for navigation clearance, horizontal and vertical requirements are as follows:

Horizontal: Min. 240 ft (73.1 m)
Vertical: Min. 60 ft (18.3 m) at least 1 span (3 spans preferable) at the river center, and for the rest Min. 40 ft (12.2 m)

(b) Location of river facilities

River facilities, of which details are discussed in Chapter 7 of this report, will be planned on both bank sides. Basically, effects of deep scour would have to be considered for designing the Padma main bridge which crosses over the river section between river facilities on both banks. Distance between both banks is measured at approximately 5.3 km by topographic survey. Subsequently, the distance between river facilities on both banks will be 5.3 km or more.

(c) Longitudinal Geometry of Approach Roads and Maximum Embankment Height

Longitudinal grade of approach is determined at 3.0 % for approach roads as discussed in the subsequent Chapter 6 of this report. From the technical examination in the subsequent Chapter 6, maximum embankment height is 7.0 m. Bridge abutments height will be located at the location of maximum embankment height of the approach roads.

Viaduct will be planned from the end point of the Padma main bridge to the abutments.

As a result of the above, the total bridge length and lengths of Padma main bridge and viaducts were decided as follows:

Padma Main Bridge	5,400 m
Left (North) Bank Viaduct	60 m
Right (South) Bank Viaduct	360 m
Total Bridge Length	5,580 m

3.3 STUDY ON SUPERSTRUCTURE AND FOUNDATION TYPES AND PREFERABLE SPAN LENGTH

During the previous stage, several candidate types for superstructure and foundation were studied, and the construction costs were estimated and compared for the main bridge over the Padma River for spans between 100 and 250 meters in increments of 25 meters.

The followings were concluded in the Interim Report;

1) Concrete superstructure is less costly than steel ones,
2) Insitu Concrete Piles are not suitable for the bridge due to the lack of strength and long operation period for the installation, and.
3) The target span length for the main bridge might be ranged between 175 and 200 meters in terms of construction cost.

3.3.1 Superstructure Study

(1) General F~~eatures of Applicable Superstructure Ty~~pes

Spans in the mid-river must be navigable and thus be longer than 100 meters in order to clear 76.2 meters (250ft.) wide for navigation, taking into account of width of pile caps.

Superstructure types, which can satisfy the condition, are PC Box Girder Bridge, PC Extradosed Girder Bridge and PC Cable Stayed Girder Bridge. Some descriptions are given below on each type.

(a) PC Box Girder Bridge

Most of major bridges in Bangladesh are of this type. All the vertical loads are borne solely by the box girder.

Generally this type is applicable to spans approximately up to 160 meters. The girder depth at intermediate piers is approximately 1/18 of the span length, and at mid-span 1/40 as shown in the Figure 3.3.1. Therefore this type may appear bulky and monotonous compared with other types.

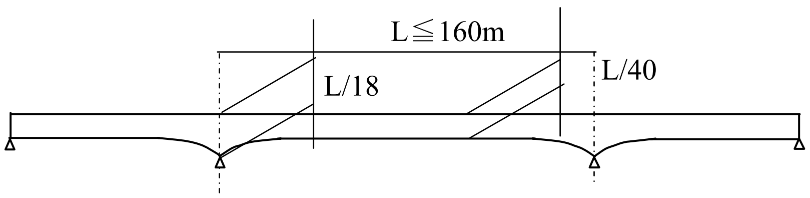


Figure 3.3.1 PC Box Girder Bridge

(b) PC Extradosed Girder Bridge

The type is of rather a new concept, whose applicable span lengths lie in between Box Girder and Cable Stayed Girder, usually having spans between 100 and 200 meters.

It has a cable system connecting the deck and comparatively low pylons. The primary purpose of the cable system is to provide more effective pretension to the deck girder by placing the cables outside of the girder section. The cable system usually bears 10 to 20 % of vertical loads working on the girder, and the rest is supported by the girder itself.

As shown in the Figure 3.3.2, usually tower height is approximately 1/8 to 1/12 of the span length. Girder height is 1/30 to 1/35 of the span at piers, and 1/50 to 1/55 at mid-span.

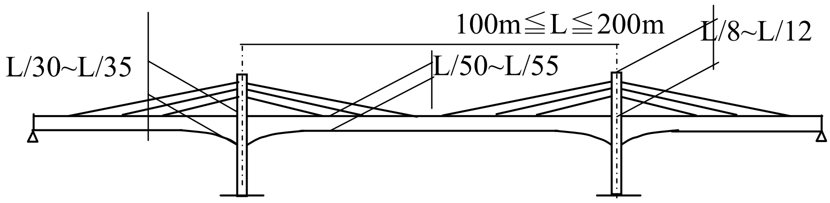


Figure 3.3.2 PC Extradosed Girder Bridge

(c) PC Cable Stayed Girder Bridge

More than two hundred PC Cable Stayed Girder Bridges have been constructed in Japan for the last twenty years. It has a cable system with higher pylons, which bears 85 ~90% of the vertical loads. Therefore the girder can be more slender than the other two options, giving an impression of the smart shape of the bridge to viewers, although an aerodynamic stability of this type has to be examined more in detail.

Commonly it can span between 100 and 250 meters, having the pylon height of 1/5 of the span length.

The Figure 3.3.3 shows that the girder depth can be lowered 1/100 to 1/200 depending on number of cables and the arrangement, and is usually uniform longitudinally.

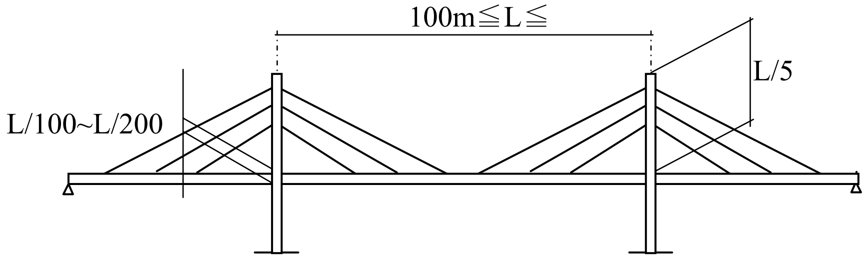


Figure 3.3.3 PC Cable Stayed Girder Bridge

(d) Composite Cable Stayed Girder Bridge

When more than 250 meters have to be cleared, PC cable stay bridges cannot span the gap due to the heavy self weight. A composite structure may be adopted in order to reduce the weight, where a portion of the deck girder, usually the middle portion of the deck at the main span, is constructed of steel comparatively light weighted. The Figure 3.3.4 shows a composite cable stayed girder bridge.

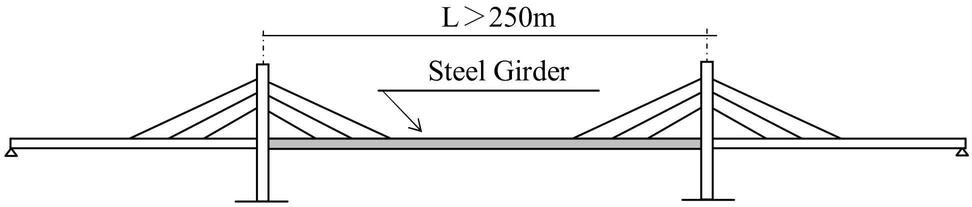


Figure 3.3.4 Composite Cable Stayed Girder Bridge

(2) Alternatives of Structural Skeleton for Analysis

The concrete structure of a cable stay bridge and an extradosed bridge is composed of pylon, girder and pier.

There are several alternatives of combinations of connecting with each other. Typical alternatives of these are illustrated in the Table 3.3.1.

(a) Rigid Connection Type

Pylon, pier and girder are rigidly connected with each other, where costly bearings are not installed to support the girder. Furthermore the longitudinal displacement of the girder is minimized during earthquake. Such connection systems have an advantage of requiring no temporary fixing or support during balanced cantilever erection.

(b) Continuous Girder Type

The girder rests on bearing at pier, which will allow rotating and/or longitudinal displacement, though cost of bearings affects much.

This connection is classified into two types.

One is that the pylon and the girder are connected rigidly and the whole is supported on a massive bearing at pier, which enables reaction force to be distributed among piers. It requires a temporary fixing during a cantilever erection.

The other is that the pylon is connected to the pier, but the girder is supported on a comparatively compact bearing without any connection with the pylon. This type also needs temporary fixing for cantilever erection.

(c) Floating Type

Pylon and pier are directly connected with each other, but the girder is suspended by cables without any bearing support at pier. Seismic forces can be reduced due to longer natural period, though horizontal displacement of the girder increases.

This type is adopted for a long span cable stay bridge.

(d) Option Adopted for the Preliminary

The rigid connection type is adopted for the preliminary design of the extradosed option taking into account the advantages during erection. This type is most commonly used for modern extradosed.

The floating type is to be recommended for the cable stay bridge option.

(3) Alternatives of Girder Cross-section

With the cable stayed and extradosed bridges the greater participation of the girder in the overall structural behavior gives the opportunity to consider alternative girder forms particularly with concrete being an efficient material when used for compression members.

When a span is long enough, properties of the girder section regarding the aerodynamic behavior also have to be taken into account.

There are three major options of girder sections as mentioned below and also shown in the Table 3.3.2.

(a) Box type

Usually this section is rectangular or trapezoidal. One or more boxes arranged in parallel are used, which may be separated by longitudinal walls into one or more cells. The box type has a great torsional resistance and can be adopted either for a single or twin plane stay system.

Trapezoidal sections are stable in an aerodynamic behavior thus they are often used when the spans are long where aerodynamic problems have to be considered. This type is also suitable for a wide deck.

Most modern cable stay bridges and extradosed bridges have these types of girder section.

(b) Wing Type

This is light weighted, and also stable against wind, though it can be used only for a twin plane stay system and difficult to apply to a girder with varying depth.

(c) Twin Girder Type

This is light weighted and has a good constructivity especially for uniform girder depth, but on the other hand is less resistant against torsional forces and can be used only for a twin plane stay system.

(d) Option Adopted for the Preliminary Design

As the objective bridge for the preliminary design has a span as long as 180 meters, therefore an aerodynamic consideration must be given. And also eccentric loadings may occur which needs considerable torsional resistance.

Taking into account all of these conditions the preliminary design adopts a trapezoidal box section with three cells.

Table 3.3.1 Alternatives of Connection among Girder, Pylon and Pier

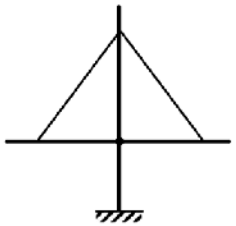
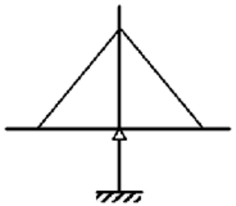
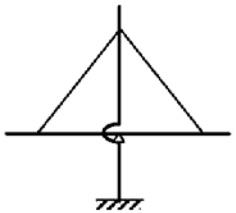
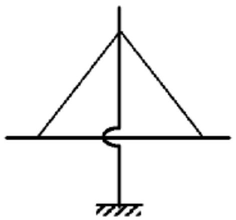
Type	Connection among pylon, pier and girder	Geometry	Features
Rigid Connection Type	Pylon, pier and girder connected rigidly with each other		-Bearing unnecessary, suitable for cantilever erection. -Affected considerably by temperature, creep and shrinkage in case of multi-span continuous -Less horizontal displacement of girder by seismic force, though large forces working on girder adjacent to pier.
Continuous Girder Type	Pylon and girder connected rigidly, supported by bearing at pier		-Efficient distribution of reaction force among piers. -Larger section required for joint porion between pylon and girder. -Massive bearing required to support the structure due to self weight of both pylon and girder. -Necessary to be tempolarily supported during cantilever erection. -Less forces working on supersturcture by earthquake. -First mode dominating during earthquake, thus natural period being shorter.
	Pylon and pier connected rigidly, girder supported by bearing at pier		-Efficient distribution of reaction force among piers. -Less massive bearing is sufficient. -Suitable to continuous girder. -Necessary to be tempolarily supported during cantilever erection. -Large forces working on pylon and pier due to earthquake.
Floating Type	Pylon and pier connected rigidly, girder suspended by cables		-Seismic forces reduced due to longer natural period, though horizontal displacement of girder increased. -No bearing support required, though transverse restraint needed. -Necessary to be tempolarily supported during cantilever erection. -Limited only to multi-cable type. -Large forces working on pylon and pier due to earthquake.

Table 3.3.2 Alternatives of Girder Cross-section

Section Type	Girder Form	Features
Box Type		-Having a large torsional rigidity. -Suitable for a wide deck. -Stable against aerodynamic behavior. -Easy to encase public utisities. -Possible to be utilised to either single or twin stay system.
Wing Type		-Light weight. -Stable against aerodynamic behavior. -Used only to twin plane system of cables. -Difficult in varying girder depth.
Twin Girder Type		-Light weight. -Less torsional rigidity. -Used only to twin plane system of cables. -Good constructivity especially for uniform girder depth.

(4) Alternatives of Stay and Pylon

As stated in 3.3.3 (2), an Extradosed Girder Bridge with spans of 180 meters was recommended as the most favorable superstructure, and a Cable Stayed Girder Bridge was also requested to be included as an option by the Bangladesh side.

As both of the superstructure types have pylons and stay cable system, alternatives of stay cable and pylon have to be considered and favorable combinations of them must be selected. The types and features are described hereunder.

(a) Stay Cable Arrangement

Three basic arrangements have been developed for the layout of the stay cables:

- Fan Stay System
- Modified Fan Stay System
- Harp Stay System

These alternative stay cable arrangements are illustrated in the Table 3.3.3.

i) Fan Cable System

The fan system was adopted for several of the early designs for the modern cable stay bridge; the method of supporting the stays on top of the pylon was taken from suspension bridge technology where the cable is laid within a tower top saddle. Each cable stay can pass over the pylon and be anchored directly within

The back span. This arrangement is structurally efficient with all the stays being located at their maximum eccentricity from the deck and applying minimum moment to the pylon.

The fan arrangement proved suitable for the moderate spans of the early cable stay designs, with a small number of stay cables supporting the deck. As longer spans become necessary the size of the limited number of cables increased, becoming uneconomically large and difficult to accommodate within the fan configuration. The anchorages are heavy and more complicated and the deck needed to be heavily strengthened at the termination point. There are also obvious difficulties with the corrosion protection of cables at the pylon head and with the replacement of individual stays in the event of any damage.

ii) Modified Fan Cable System

A greater number of stays could be provided when the modified fan layout was introduced so that the stays are individually anchored near the top of the pylon. This is now the more commonly adopted system. In order to give sufficient room for anchoring the cable anchor points are spaced vertically. Providing the anchor zone is maintained close to the pylon top there is little loss of structural efficiency as the behavior of the cable system will be dominated by the outer most cable attached to the top of the pylon and anchored end of the back span.

Table 3.3.3 Alternatives of Stay Cable Arrangement

Types	Arrangements	Features
Fan Stay System	Stay cables are arranged radially from pylon top. 	-Anchoring details more complicated at pylon top. -Pylon to be completed before girder being erected. -Greater forces working on the pylon than other alternatives. -More possibility of pylon being buckled. -Stay being more efficient, thus less cable materials required. -Suitable to moderate spans. -Difficulties in maintaining and replacing the stays in case of the damage.
Modified Fan Stay System	Stay cables are arranged almost radially from separate anchorages near pylon. 	-Intermediate between "Fan" and "Parallel" -Suitable to long spans. -Possible to accommodate a greater number of stays. -Less possibility of pylon being buckled. -Easy to maintain and replace the stays. -Greater uniformity through the deck section providing a continuous elastic support.
Harp Stay System	Stay cables are arranged parallelly each other. 	-Convenient to cable installation due to wide spacing of anchorages. -Possible to costruct pylon keeping step with progress of girder erection. -Stay being less efficient, thus more cable materials required. -Less possibility of pylon being buckled. -The least efficiencies of stays. -Providing a visual emphasis of the flow of the forces and being aesthetically pleasing.

iii) Harp Cable System

With the harp system the individual stays are anchored at equal spacing over the height of the pylon and are placed parallel to each other. This arrangement provides a visual emphasis of the flow of forces from the back span to the main span and is aesthetically pleasing. However, the arrangement is not as structurally efficient as the fan layout and relies on the bending stiffness of the pylon and/or deck for equilibrium under non-symmetrical live loading.

iv) Option Adopted for the Preliminary Design

The span length is as long as 180 meters; hence the number of cables required is many. Thus the fan system is not suitable taking into consideration of the complicated details of the connection at the pylons.

The harp system has a visual advantage, but on the other hand has a fatal disadvantage of inefficiency of cable tension, which will lead to uneconomical construction.

Then the modified fan stay system is recommended for the preliminary design of both Extradosed and Cable Stayed Girders.

(b) Number of Cable Planes

The cable layout may be arranged as either a single plane system or as twin plane system, as shown in the Table 3.3.4.

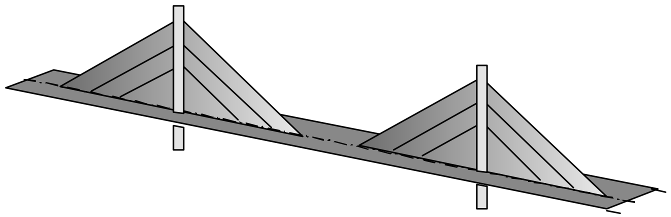
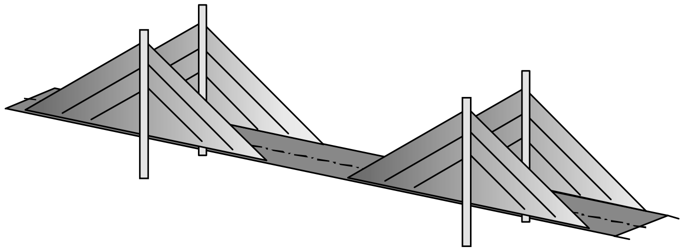
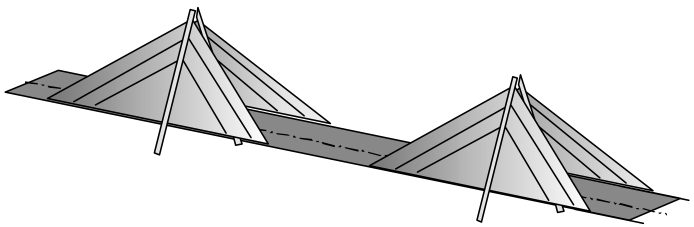
i) Single Plane System

The single plane creates a classic structural form avoiding the visual interference often associated with twin cable planes. However the single plane is not able to resist torsion loading from eccentric live loading and therefore this configuration requires the deck to be in the form of a strong torsion box. A deck section of this form is then likely to have excess resistance to the longitudinal bending of the deck, particularly when a multi-stay arrangement is used. The single pylon has to be located within the central median of the carriageway and as such an additional width of deck is required for the necessary clearances of traffic.

ii) Twin Plane System

The twin plane system may either be formed as two vertical planes connected from the edge of the deck
to two pylon legs located outside the deck cross-section or as twin inclined planes connected from the edge of the deck to either an A frame or inverted Y frame pylon. Inclined stays are of particular benefit when adopted for the longer spans as they improve the torsion response of the structure to both eccentric live load and aerodynamic effects. When comparing the two alternative stay systems, one with two vertical stay planes and one with two inclined stay planes, supporting a deck which has low torsional stiffness the inclined stay system connected to an A shaped pylon will have approximately half the twist under eccentric loading. The inclined stay system is also aerodynamically superior reducing the magnitude of vortex shedding oscillations and increasing the critical wind speed of the structure. However, the geometry of the inclined stay planes must be carefully checked in relation to the traffic envelope and the clearance required may result in an increase in the overall width of the deck.

Table 3.3.4 Number of Cable Planes

Types	Arrangements	Features
Single Plane System		-Wider median required. -An additional width of deck required for the pylons and cables. -More spacious view and avoiding visual interference for drivers. -More consideration required against torsional forces. -Easthetically nice and simple. -Pier width in transverse direction possible to be reduced.
Twin Plane System (Vertical)		-Torsional resistance of girder increased due to twin system. -Not necessary to widen the median for pylon. -Wider pier in transverse direction required. -Many crossing cables giving not pleasant view.
Twin Plane System (Inclined)		-Torsional resistance of girder increased greatly. -Deck to be widened to secure the traffic clearances. -Wider pier in transverse direction required. -Many crossing cables giving unpleasant view.

iii) Option Adopted for the Preliminary Design

The railway is to be located at the center of the cross-section, and then Single Plane System cannot be taken as an alternative. Even with the exclusively highway purpose unfavorable torsion will be created due to eccentricity caused by the four lane vehicle loadings.

The inclined twin system will interfere with the traffic clearance or the cross-section must be widened to avoid the problem unreasonably.

Thus an option with Vertical Twin Plane System is adopted for the preliminary design.

(c) Pylon

The pylon is the main feature that expresses the visual form of any bridges with cable system giving an opportunity to impart a distinctive style to the design. The design of the pylon must also adapt to the various stay cable layouts, accommodate the topography and geology of the bridge site and carry the forces economically.

The primary function of the pylon is to transmit the forces arising from anchoring the stays and these forces will dominate the design of the pylon. The pylon should ideally carry these forces by axial compression where possible such that any eccentricity of loading is minimized.

Concrete is very efficient when supporting loads in axial compression. Advances in concrete construction and modern formwork technology has made the use of concrete increasingly competitive for pylon construction, despite the much greater self weight, when compared with a steel alternative. Concrete has proved particularly adaptable to the more complex forms of pylon. Many varied types of pylon have been developed to support both the vertical and inclined stay layouts. These include H frame, A frame and inverted Y frame pylons as illustrates in the Table 3.3.5.

i) Single Pylon

This can be applied only to single plane system, which is not to be adopted for the preliminary design.

ii) Twin Pylon

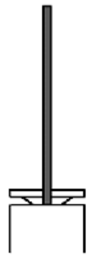
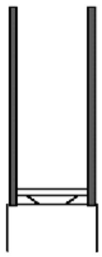
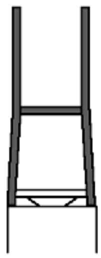
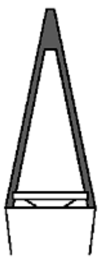
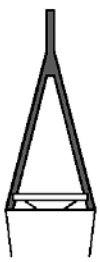
Twin pylons are constructed separately for each plane of twin stay system. They are completely independent each other.

iii) H Frame Pylon

A modification of the twin pylon, where the two pylons are connected each other to work as a single structure of a rigid frame.

The stay anchors are normally located above the level of a crossbeam. With the modified fan arrangement of stays this crossbeam location would be between mid height and two-thirds of the pylon height above the deck. When the harp arrangement of stays is adopted the anchors are distributed over the full height of the pylon above the deck.

Table 3.3.5 Types of Pylon

Pylon Type	Pylon Geometry	Features
Single Pylon		-Median needed to be widened. -Low transeverse rigidity. -Adopted only for single plane system. -Spacious view provided for drivers.
Twin Pylon		-Low transeverse rigidity. -Adopted only for twin plane system.
H Frame Pylon		-Moderate transverse rigidity. -Adopted only for twin plane system.
A Frame Pylon		-High transverse rigidity. -Wider pier required. -More complex work required for construction. -Deck covered with inclined cables above. -Suitable for inclined stay system
Inverted Y Frame Pylon		-High transverse rigidity. -Pier required to be widened. -More complex work required for construction. -Suitable for single plane stay system.

iv) A Frame Pylon

The A frame pylon is suitable for inclined stay arrangements.

v) Inverted Y Frame Pylon

This is a variation of the A frame where the vertical leg, containing the stay anchors, extends above the bifurcation point.

vi) Option Adopted for the Preliminary Design

As a vertical twin plane system is to be adopted, “Single Pylon”, ”A Frame Pylon”, “Inverted Y Frame” cannot be adopted.

An H Fame Pylon may be suitable for a tall pylon such as cable stay bridges, but when pylon are as low as those for extradosed bridges the crossbeam attached below cables might interrupt drivers’ view.

Twin pylons are used for the preliminary design of the extradosed bridge, and H-frame for the cable stay bridge.

(d) Stay Anchorage

In early designs the connection between the stays and the pylon was formed in the same manner as for suspension bridges where the cables are laid in a saddle and carried through the pylon (Referred as “Penetrating Type” in the Table 3.3.6). The evolution of the modified fan and harp arrangements with stays anchored over a portion of the pylon leg has led to the use of separate stays for the main span and back span (Referred as “Independent Type”). The Independent Type has three modifications; they are Separate Anchor, Cross Anchor and Steel Frame Anchor.

i) Separate Anchor

This is the most direct form of anchoring by attaching the stay socket or anchorage plate to the wall of the pylon. In this layout the hollow pylon shaft gives access to the stay anchors for stressing during erection and inspection or replacement in service.

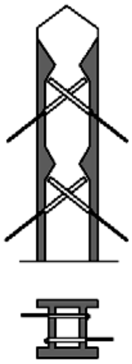
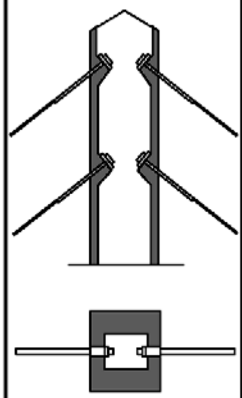
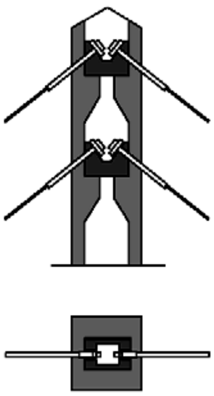
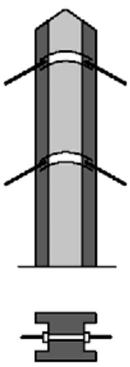
ii) Cross Anchor

An alternative arrangement, producing a more slender pylon, allows the main span and back span stays to cross so that they are anchored in rebates on the reverse sides of the pylon. The horizontal component of the cable forces will place the pylon into compression. However, the two stays cannot be in the same plane and the pylon must be designed for the resulting torsion arising from this eccentric loading.

iii) Steel Framing Anchor

All the above methods of connecting the stay to the pylon rely on the accurate placement of the steel formers and any anchor prestress and reinforcement within the concrete walls if the stay geometry and the strength of the connection intended in the design is to be realized. The complexity of the required
details will often slow the progress of the erection throughout this critical zone of the pylon construction. In order to mitigate these problems steel fabricated anchorage modules have been manufactured such that the required stay geometry is completely defined.

Table 3.3.6 Layout for Stay Anchor at Pylon

	Type	Pylon Shape	Features
Independent Type	Cross Anchor		-Stays anchored on the cross in a solid pylon. -Many such pylons performed in the past. -Some considerations required against torsional force.
	Separate Anchor		-Stays anchored without crossing in a hollow pylon cross-section. -Pylon required to be reinforced against lateral tension force. -Less intervals between stays. -Easy inspection of anchorage allowed.
	Steel Framing Anchor		-Stays anchored without crossing in hollow pylon cross-section. -Pylon to be reinforced against lateral tension force by steel beam. -Greater cross-section of pylon required. -Easy inspection of anchorage allowed.
Penetrating Type	Saddle Anchor		-Stays penetrating solid pylon over saddle. -Stays fixed at inlet and outlet of pylon. -Less intervals achieved between stays. -Width of pylon limited by the minimum curveture of stay.

iv) Saddle Anchor (Penetrating Type)

The cables are laid in a saddle and carried through the pylon, where the both ends of the cables are anchored at the main span and the back span. This method enables the spacing of the stay anchors at pylon to minimize so that the eccentricity of each cable can be kept the greatest. The pylon is solid and can be reduced in cross-section size. The cables are fixed at each side of the pylon not to slides.

v) Option Adopted for the Preliminary Design

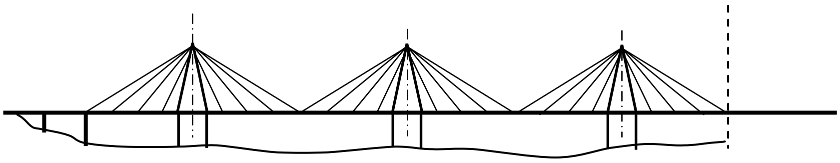
With the extradosed bridge the Saddle Anchor has the advantage of the great eccentricity, the minimum cross-section of the pylon and easy handling of stays during construction.

As to the cable stay the each cable will be bulky and heavy, and hard to handle, thus the Saddle Anchor is not suitable for the anchorage. The steel Framing Anchor is recommended for this the cable stay alternative.

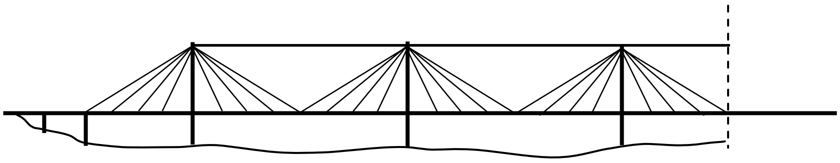
(5) Multiple Span Bridges

The main concern with multiple span cable stayed bridges is the lack of longitudinal restraint to the inner pylons, which cannot be directly anchored to an approach pier. Without providing additional longitudinal restraint a multiple span structure would be subject to large deformations under the action of live load. Increasing the stiffness of either the pylons or deck can provide this additional restraint. Any increase in the deck stiffness will be accompanied by an unacceptable increase in the dead load and thus, the more practical approach is to stiffen the pylon. A typical example of the stiffened pylon is the A-frame braced pylon shown in the Figure 3.3.5 (a). However, such an arrangement will require a substantial increase in the pylon materials and require a much larger foundation. An alternative to relying upon the bending stiffness of the pylon is by the introduction of an auxiliary cable system to provide the required stability. Two cable systems are illustrated, the first system, in Figure 3.3.5 (b), connects the tops of the pylons and thus directly transfers any out of balance forces to the anchor stays in the end span. The second system in Figure 3.3.5 (c), connects the top of the internal pylons to the adjacent pylon at deck level so that any out of balance forces are resisted by the stiffness of the pylon below deck level.

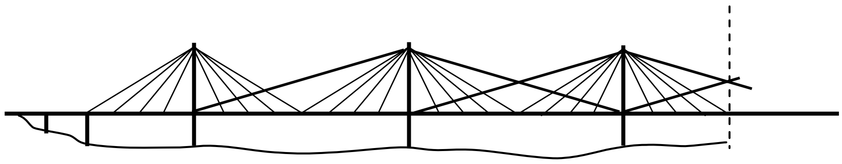
Extradosed bridge has low and rigid pylons therefore the disadvantages mentioned above are not serious, but with the long span cable stay bridge such a multiple span system should be avoided for economical and aesthetic reasons.



(a) A-frame braced pylon



(b) Additional Cable System



(c) Additional Cable System

Figure 3.3.5 Multiple Span Bridges

(6) Aerodynamic Design Consideration

Although the overall configuration of the bridge is determined from functional and structural requirements, consideration of wind loading and aerodynamic stability is an essential part of the design process for long span bridges with slender deck such as cable stay or extradosed bridge.

The behavior of a long span structure in response to the wind is a key part of the design process. In order to determine the aerodynamic behavior of the structure it is necessary to have an understanding of the wind regime at the bridge site. The aerodynamic response of a long span structure may be sensitive to inclined wind and this effect is dependent upon the ground topography at the bridge site.

(a) Aerodynamic Motion

The flow of the air over the slender deck of a bridge, such as a cable stay bridge or an extradosed bridge, induces vertical bending and torsional oscillations of the deck section. There are several forms of such aerodynamic motion.

i) Vortex Shedding Response

The periodic shedding of vortices alternately from the upper and lower surfaces of the deck causes a periodic fluctuation of the aerodynamic forces on the structure. Over one or more limited ranges of wind speed the shedding of the vortices can be resonant with the natural frequency of the structure in the bending or torsional modes. For the most pronounced effect it is generally necessary for the wind flow to be perpendicular to the bridge cross section. The magnitude of the response will increase with increasing wind speed at which the resonant behavior occurs. The response will decrease with increasing structural damping and with increasing turbulence intensity.

ii) Turbulent or Buffeting Response

The forces developed by the wind fluctuate over a bridge deck within a wide range of frequencies. Only the lowest modes of vibration of the largest structures are affected at the high frequency end of the spectrum and turbulence arises from these fluctuating components of the wind acting about the three axes of the structure. If there is sufficient energy within the turbulent frequency bands that are resonant with the structure there may be a significant forced oscillation of the structure. Turbulence has the greatest impact with bluff deck cross sections and on the more flexible suspension structures. For such structures the total equivalent wind effect including the dynamic enhancement can be twice the static force. The greatest response to turbulence for cable stayed structures is likely to be during cantilever erection when the deck can respond at the lower frequencies.

Usually with bridges this response is taken into account as the static wind load, and the dynamic analysis is not made as is negligible.

iii) Divergent Amplitude Response (Galloping and Flutter)

At the critical wind speed of the section there is a rapid increase in the response of the structure. This occurs when the sum of the equivalent aerodynamic damping, which can have either negative or positive sign, and the structural damping becomes negative resulting in zero total stiffness. At this point the amplitude can grow without limit exhibiting divergent behavior. Thus the value of structural damping will increase the critical wind speed but will not decrease the magnitude of the oscillations. A divergent response can be either vertical or torsional, but for most practical deck sections the torsional behavior will dominate. Usually the presence of turbulence will increase the critical wind speed.

A very strong response of the divergent kind can arise in the event of a coupling of the vertical and torsional motion. In order that this form of response cannot occur it is necessary to ensure that the torsional and bending frequencies of the structure are separated.

When considering the effect of aerodynamic motion upon the design of the cable stayed bridge both divergent amplitude response and limited amplitude response must be considered. Divergent amplitude response will result in the failure of the structure and therefore an adequate margin of safety must be provided between the characteristic mean hourly wind speed and the critical wind speed. Limited amplitude response, from either vortex shedding or turbulent wind must be considered in respect of both fatigue damage and any disturbing physiological effect upon the bridge user.

(b) Oscillation of Stays

Another phenomenon that must be considered with cable stay and extradosed bridge construction is the effect of stay oscillation. During cantilever erection a slender deck may be prone to wind induced movement. This can in turn excite the stays producing violent oscillations which have to be restrained with temporary straps. In service the stays may also be subject to oscillation, which can reach amplitudes of more than a meter. This behavior is not solely due to wind but can be accentuated when wind and rain are combined. The rain forms rivulets which run down the stay pipe such that the cross-section of the circular stay pipe is altered, sufficiently to induce lift and giving rise to an unstable divergent amplitude response. Rain-wind induced oscillation of the stays usually occurs within the wind speed range 7-15 m/s. Below that range, the top rivulet does not form because the wind is insufficient to prevent it from running down the side of the stay.

(c) Aerodynamic Stability during Construction

During erection, wind on the part completed suspended deck girder can present problems with respect to its stability and other wind load effects, as each stage of structural completion can have significantly reduced stiffness, and a different mass distribution from the final stage, and consequently different natural frequencies.

(d) Wind Tunnel Testing

One of the most useful and economic tools for determining the aerodynamic behavior of the bridge structure is through the testing of the sectional model in a wind tunnel. The sectional model consists of a representative section of the deck, geometrically and aerodynamically similar to the prototype. It is mounted in the wind tunnel in such a way as to measure the static and dynamic lift, drag and torsion produced by the wind. The model is usually located between the parallel walls of the wind tunnel so as to channel a two-dimensional wind over the model.

When mounted on springs, with scaled mass per unit length, mass moment of inertia per unit length, structural damping and natural frequencies the sectional model can be used to investigate:

- the dynamic response to vortex shedding
- to determine the response to turbulence
- to ensure that the section is aerodynamically stable and has an acceptable factor of safety with respect to the design wind speed.

(e) Examinations on Necessity of Aerodynamic Design

"The Manual of Aerodynamic Design for Highway Bridges, Japan Road Association, 1991" provides with useful guidelines of the procedure by which the necessity for the detailed aerodynamic design for bridges is examined.

Thereby usually aerodynamic design is preceded in such a manner as described below:

A. Wind characteristics at the site are estimated.
B. The necessity to design the bridge aerodynamically is examined.
C. When the examination is resulted that it is necessary, the aerodynamic parameters are calculated.
D. The aerodynamic stability is evaluated by using several formulas.
E. Further studies, including the wind tunnel testing, may be required resulting from the evaluation.

i) Wind Characteristics

Basic design wind velocity (U_d) is calculated below:

$U_d = E_1 \times U_{10} = 1.36 \times 50$ m/s $= 68.0$ m/s
Where the elevation of the deck of the Padma Main Bridge is (**z = 26m**), the terrain classification is taken as **I**, wind velocity (U_{10}) is 50m/s.

The wind turbulence is moderate for the bridge site. Therefore it is estimated as

I_u = 0.13,

based on that the river is as wide enough and the elevation of the deck is approximately 26 meters above the standard low water level.

ii) Factors for the Options

Factors for the PC extradosed bridge option and the composite cable stay bridge options are:

Factors		Extradosed Bridge	Cable Stay Bridge
Span Length	L (m)	180.0	360.0
Girder Depth	d (m)	3.5	2.5
Deck Width			
Without Railway	B (m)	24.5	24.5
With Railway	B (m)	28.0	28.0

iii) Examination on Instability in Torsion (Flutter)

<u>Formula for the judgment</u>

Where L x U_d / B ＞ 520, then the aerodynamic design is required.

	L x U_d / B	
Railway Provision	Extradosed Bridge	Cable Stay Bridge
Without Railway	500	**999**
With Railway	437	**874**

It was resulted that the cable stay bridge options with and without railway have to be further studied on the aerodynamic behavior.

iv) Examination on Instability in Bending (Galloping)

Usually concrete bridges will never have this kind of problem, thus the examination is of no need.

With the composite cable stayed bridge, the deck girder is made of steel, light weighted and slender, it has to be examined.

L x U_d / B=999 : Without Railway
L x U_d / B=874 : With Railway

Both are greater than 330

B/d = 24.5/2.5 = 9.8 : Without Railway
B/d = 28.0/2.5 = 11.2 : With Railway

Both are greater than 5, then further consideration on the instability of galloping is unnecessary.

v) Examination on Instability by Vortex Induced Oscillation (Vortex Shedding)

Formula for the judgment

Where L x U_d / B ＞ 200 , and also **I_u ＜ 0.2**
then the aerodynamic design is required.

	$L \times U_d / B$	
Railway Provision	Extradosed Bridge	Cable Stay Bridge
Without Railway	**500**	**999**
With Railway	**437**	**874**

All the resulted values are over 200.

The formula for the judgment is:

$$I_u = 0.13 < 0.20$$

The conclusion is that all the options of the cable stay bridge and the extradosed bridge with and without railway have to be further studied on the aerodynamic behavior.

vi) Conclusions on the Necessity of Aerodynamic Design

It is concluded as shown below, in accordance with the examinations above:

Necessity of Aerodynamic Design

Subject of Aerodynamic Design	Extradosed Bridge		Cable Stay Bridge	
	Without Railway	With Railway	Without Railway	With Railway
Flutter	Not Necessary	Not Necessary	**Necessary**	**Necessary**
Galloping	Not Necessary	Not Necessary	Not Necessary	Not Necessary
Vortex Shedding	**Necessary**	**Necessary**	**Necessary**	**Necessary**

(7) Construction Method

After completion of the foundation and pier, the superstructure is erected.

The erection of the superstructure will generally start simultaneously from the both banks, and proceed toward the mid-river.

The common method of deck erection is by the successive balanced cantilever method using movable bridge-builders, especially when temporary supports are not available on the riverbed.

Usually the deck is erected starting from several neighboring piers at the same time, depending on the availability of erection time and number of bridge-builders.

The construction process of the superstructure is described below taking an example of the piers P2 and P3 illustrated in the Figure 3.3.6.

a) The erection of the pier P2 and P3 starts simultaneously, when P1 deck is under work or has been completed.
b) The deck on P2 and P3 are extended with keeping balance on the right and the left of each pier using the pair of the bridge-builders. When the deck reaches to some extent, the pylons start being constructed.
c) When the pylons are completed and the cantilevers require supports, the lowermost stay cable is installed from a span to the other side of the span over the saddle of the pylon. The cantilevers will be extended again until the following anchorage module is completed, and the next stay is laid in the same manner as before. The same procedure follows until all the stays are installed.
d) The cantilevers are extended until the both ends meet at the span center, and finally joined together by the closing module.

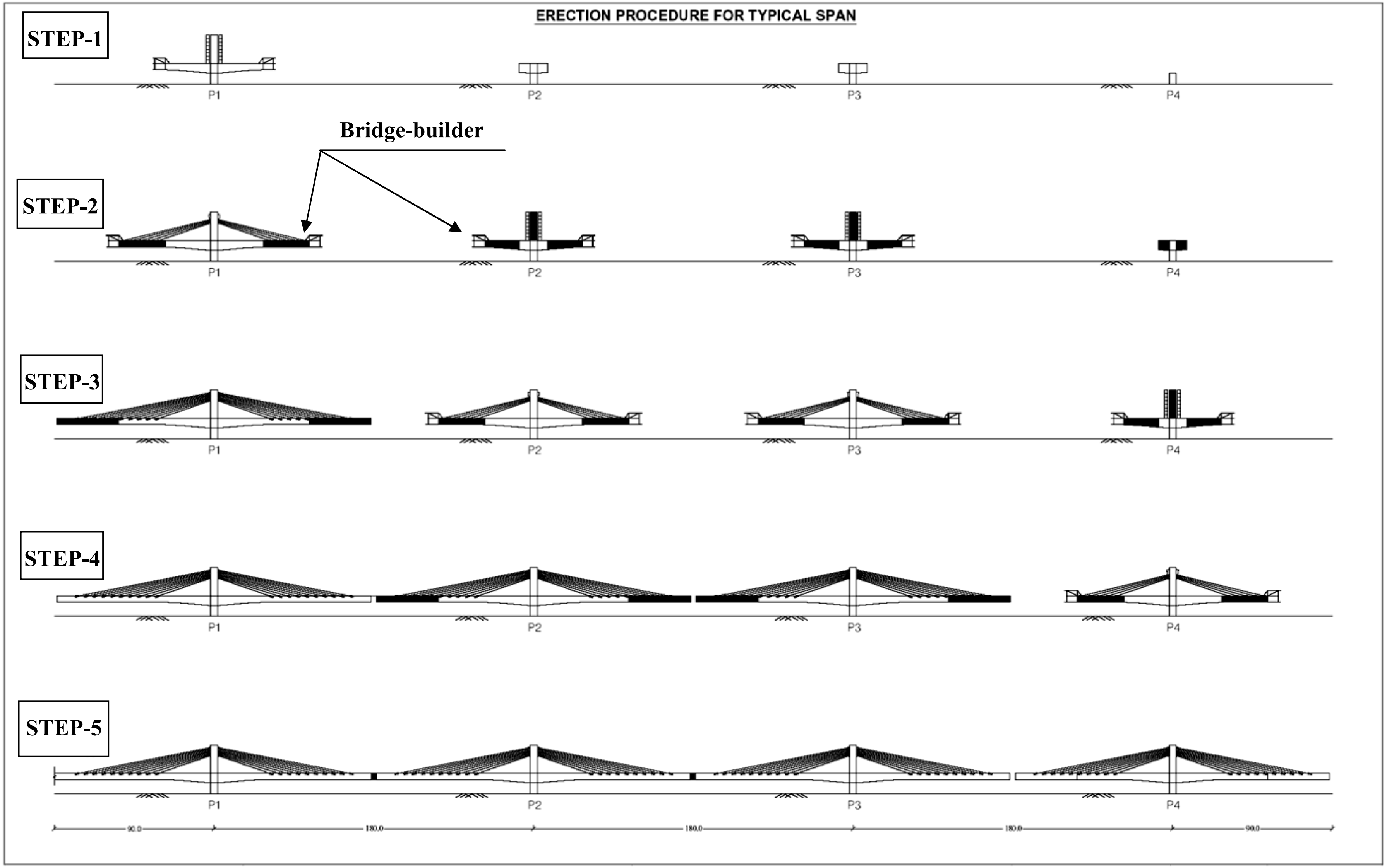


Figure 3.3.6 Erection Procedure for Typical Span

3.3.2 Substructure Study

(1) Introduction

The Padma Bridge will provide a fixed link for traffic across the Padma River. The bridge foundations will support the loads from the bridge structure and traffic and carry loading arising from the environment of the bridge.

Using the information gathered and reviewed for the study it is possible to define the environment in which the Padma Bridge will function. This environment includes the ground conditions of the bridge site; an impressive and powerful river that shifts its channel and scours to considerable depths during flood seasons; there is transport activity by large vessels and ferries on the river and the proposed site is in a seismically active area.

The ground conditions along the river are fairly uniform and the soils may be characterized as upper strata of silts, clays and silty fine sands resting on fine to medium sand with silt and clay inclusions. The soils are generally soft and the sands when exposed to the river are easily eroded.

The river study has furnished information about scour, both physical measurements made on the river and estimates of scour that could occur during high flow conditions. The hydro-graphic information shows that the Padma River has an average depth 16.5 m and that in some years during high flows the main channel has been over 30 m deep in some places. In the past the deep channel of the river has altered its position by up to 500 m in a year as it shifts from bank to bank.

River traffic is made up of numerous large and small vessels: the various data available indicate that vessels of up to 4000 DWT use the river. That such vessels ply past the Padma site raises the possibility of severe loading from ship impact.

Bangladesh is situated in a seismically active zone of the world and has experienced several large earthquakes in the last one hundred years. Earthquakes may be expected to affect the Padma Crossing and cause significant loads on the bridge as a result of ground motions.

The ground and river are inherent and continuous constituents of the bridge environment and will cause lasting or general loading on the structure. Alternatively the river traffic and seismic activity would only occasionally affect the bridge. The loading these would cause would be transient and may be considered unusual or abnormal.

The main conditions that will govern the foundation design are the scour depths coupled with the large horizontal loads imposed on the foundations by the governing load cases of earthquake or ship impact. The type and size of the foundation will depend on the capacity of the ground to resist the vertical and horizontal loads and will govern the practicalities of constructing the foundations

(2) Bridge Supports

(a) Intermediate Support - Piers

The piers above the pile caps will need to be of robust construction in the zone at risk from ship impact. They may be in the form of reinforced concrete discrete columns, leaf pier, hollow box in-filled with concrete (Jamuna bridge) or solid walls used in a pierced box form to provide a stiff frame (Bhairab Bridge). Above the level of impact the design would need to encompass the requirements and details for seismic conditions.

The geometry of the main bridge pier is largely dictated by the monolithic connection to the superstructure and cable pylons above. A portal frame arrangement has been selected comprising hollow box columns to support the pylons, and cross beam to support the superstructure. This monolithic cross beam currently shown beneath the superstructure may be integrated within the superstructure during the detailed design when further consideration should be given to bridge aesthetics.

(b) End Support – Abutments

The abutments for the bridge could be constructed using a bank seat, hollow box or retaining wall structure. Bank seats are usually provided at the top of either a cut or filled embankment in order to provide an open appearance to the bridge structure and reduce the cost of wing wall construction. The former is not likely to be critical for the long open aspect of the Padma River Bridge. A 'T' shaped cantilever abutment wall is the most common form of construction for heights up to approximately 9.0m. For each of these forms of construction piled foundations would be necessary to distribute the load and reduce settlement. A wall solution would generally be simpler to construct, however the advantage of a box structure is that the vertical loading on the foundations would be reduced and horizontal load could be shared over a larger number of piles. For a bank seat the piles must be designed for horizontal loads applied on the pile shaft above general ground level at which lateral ground resistance will start to restrain the piles.

The upper levels of the ground have been described as silty or clayey sands, such soils can be prone to consolidation. The abutment design must allow for the effects that consolidation settlement may have on the foundations such as increased active soil loading and negative skin friction on the piles.

The height of the structure should be kept to the minimum possible to reduce the effects of retained fill as the soil load is related to the square of the height. Under seismic loading active soils pressures may be expected to increase by 60% and the acceleration effects on the mass of the structure will also load the foundation.

(3) Foundation Design Parameters

(a) Design Philosophy

There are three requirements to be satisfied for a satisfactory foundation design:

- The structures of the foundation must be strong enough to carry the loads applied.
- The foundations distribute the loading such that the strength of the soil strata is not exceeded.
- Deflections and settlements to be within designated limits for adequate function of the bridge.

(b) Foundation Loads

The foundations must be designed to resist all applied loads that may be reasonably foreseen during the design life of the structure. These loads may generally be divided into the following categories:

- Permanent loads including self-weight of all structural components, road and rail furniture, surfacing, services and structural behavior.
- Transient live loads due to highway and railway vehicle loads.
- Environmental loads due to wind, wave, river flow and temperature variation.
- Abnormal loads due to boat impact or earthquake.

- Construction loads

The above loads will be combined appropriately with respect to their frequency of occurrence to provide the design load combinations applied to the substructure.

The preliminary design has shown that there are three onerous load combinations that can be generalized as:-

- Permanent, transient live loads and maximum wind load during maximum scour
- Boat impact and permanent loads with maximum scour
- Seismic and permanent loads with maximum scour and liquefaction effects.

It should be noted that the magnitude of effect from the first two combinations are directly proportional to the applied loads. However the magnitude of effect from the seismic combination will also vary with structure stiffness or change in stiffness throughout the length of the bridge.

From the initial design it is evident that both the vertical and horizontal loads on the foundations will govern the design. These loads coupled with the scour depths estimated at the bridge site causes overturning effects in addition to high shear loads on the foundation. For shorter or lightly loaded spans the boat impact force will determine the critical combination, whilst for longer spans the seismic load combination will generally be critical. The initial design of extradosed girder bridge with railway provision has shown that the seismic combination will dominate for the optimized span length discussed in Section 3.3.3 of this Chapter.

Seismic events in addition to causing horizontal loading can induce liquefaction in the top of the supporting ground. The layer of soil becomes a fluid that will not support load, in effect lowering the ground level. Consequently foundations must be installed below the layer affected. The liquefaction and local scour are taken as additive effects; this is an adverse effect that must be included when determining seismic loads on the foundations. Although the current geotechnical investigation does not anticipate any potential for liquefaction effects at the bridge site this is discussed further in Appendix 4 of this report.

(c) River Scour and Flow

River scour influences the following aspects of substructure and foundation design:-

- The design height of the substructure.
- Overturning effects on the foundation
- Loads due to water flow and wave.
- The slenderness of the substructure affecting vibration and buckling.

The design scour depth is the sum of the general and local scour depths related to the flow in the river channel and the size of the bridge support obstruction respectively. At this stage of the study the local scour, considered to be of the order of twice the obstruction width, is a large portion of the design scour depth. River scour is discussed in greater detail in Appendix 5 of this report.

The drag force produced on the substructure by river flow is related to the square of the velocity of the water passing the substructure. This force is very low in comparison to seismic and vessel impact loads. However it is important in regard of pile oscillation caused by vortex shedding and the structural capacity of the piles to withstand these oscillations, the onset of oscillation is related to the flow velocity. It will be necessary to check piles for construction (free standing) and maximum scour (braced) cases to ensure the pile sections adopted are sufficiently stiff to resist this phenomenon.

Scour occurs during high flow conditions, which reach their peak during the months of July through to October. It is not clear at which stage of the flow cycle re-deposition of bed material will begin but it is assumed that scour action will continue to keep the river channel open until a marked fall in river levels begins. Wind loading will be a maximum during cyclone conditions for which there are two seasons each year mid April through May and October to November. Thus through October there is a period when the conditions for maximum scour and cyclones to occur overlap. The maximum wind load design case should thus adopt maximum scour as a design condition.

Navigation on the Padma River may take place at any time of the year as ferry services are a continuous and important function of the transport system in Bangladesh. Although large ferries are used for the crossings at Mawa it is understood that this service will discontinue one the bridge is in use. Ship impacts on bridges are random events. The size of the impact force is governed by the size of the vessel that can reach the bridge together with its velocity. Which in turn depend on vessel draft, water depth and river flow velocity. The channel at the bridge site will be deep enough for vessels to transit at all times of the year. As noted above the conditions for maximum scour are present for 4 months in each year there is a high probability that ship impact and maximum scour would coincide.

Seismic events though rare are random events, and of short duration. As maximum scour will occur over sustained periods there is a high probability that a seismic event could coincide with maximum scour conditions.

The river flow and state of scour depth will also influence the construction method adopted. This is discussed later in this Chapter.

(d) Soil Conditions

The ground conditions along the river are described in Appendix 4.

Below the scour depths at the bridge site the soil is generally very silty fine sand with traces of mica. It is the engineering properties of these strata that will govern the foundation design for vertical and particularly horizontal load capacity. The resistance of the ground to lateral loads and movement will be the most important design consideration. The presence of mica can have a marked effect on the soil properties.

(e) Trial and Working Pile Tests

Whilst the soils properties can be derived from Site Investigations and laboratory tests these are obtained by sampling in a limited context. The piles will subject a wide zone and depth of soil to load. In order to avoid the effects of scale it will be important to conduct some large scale pile tests to prove that the soil properties used for design are satisfactory. Also during construction at each foundation a system of testing some of the piles and the ground to ensure the basic assumptions of the design will be met will be needed.

(4) Foundation Types

(a) General Requirements

Three types of foundation have previously been identified as potentially suited for use to support the Padma Bridge.

- Steel tubular piles – Raking / Vertical

- Reinforced in situ concrete piles - Vertical
- Reinforced concrete open caissons

There are some characteristics that will be common to each foundation type:

- The structure must be robust in the zone of ship impact to resist the impact force.
- The design height of the free standing substructure will be governed by the general and local scour depths combined with liquefaction depth.
- The need for the foundation to extend to a sufficient depth to mobilize enough of the soils load capacity to ensure the foundation loads are safely transmitted to the supporting soils.
- The foundation should be of a form which allows methods of construction compatible with the seasonal environment changes to the river state.

(b) Piled Foundations

i) Vertical Piles

Pile foundations will present slender members projecting above the river bed. Vertical piles offer some advantages over raking piles, as they may be installed by either driving pre-cast concrete or steel sections (this method may be augmented by pre-boring) or by using bored cast in situ concrete piles.

Horizontal loads on the piles are resisted by the flexural and shear strength of the piles together with the lateral resistance of the supporting soil. Axial loads are considerably less than those caused on equivalent raking piles therefore increasing uplift and tension in the outer piles. Lateral deflections will be greater than those from equivalent raking piles due to the reduced horizontal stiffness.

Interaction between closely spaced piles and their supporting soil reduces the capacity of the pile group. This effect may be reduced or negated by increasing the pile spacing to a point where negligible interaction occurs. This directly increases the size of the pile cap by the same proportions. The close spacing of piles would also increase local scour.

Vertical piles are not considered suitable for the Padma Bridge main spans due to the large lateral design loads and the large design height of the structure under maximum scour condition.

ii) Raking Piles

Raking piles for the bridge foundations is considered to be the most effective use of piles to resist horizontal loads. The piles provide a propping action that reduces the shear load and bending moments on the piles. This has the advantage of reducing the lateral load on the ground and consequently the ground displacements. The rake layout also reduces the rotations and deflections at the top of the pile group.

Conversely, there will be a large increase in pile axial load in both compression and tension when subject to horizontal load cases. Therefore some piles may require anchoring against pile uplift.

A further advantage of raking piles is that the pile spacing at river bed and founding level may be increased relative to that at pile cap level. This will reduce the pile group capacity effect and minimize the interaction of local scour.

Raking piles are usually installed by driving prefabricated units connected in-situ. Boring equipment for long large diameter piles can usually only operate vertically due to the high risks of damage and jamming of the drilling equipment). The piles must be positioned using a support frame resulting in some loss of pile hammer efficiency. This frame would be mounted on a barge or the river bed. For the Padma Bridge a rake of 1:6 gives significant benefits to the foundation whilst change to the vertical load capacity of the pile will be minimal.

The feasibility of large 3150mm diameter steel tube raking piles has been successfully demonstrated for similar ground conditions on the Jamuna Bridge Project.

iii) Steel Tubular Piles

Using steel piles would be the quickest way to construct the foundations with the piles installed in a raked or vertical position. A further advantage of steel piling is that the high strength of the pile section will work equally in tension and compression. Although piles supporting the Padma Bridge will be subjected to tension loading, this will not be critical to the design for steel piles.

The end bearing capacity of the piles may be significantly increased by providing a mass concrete plug to the toe. Furthermore the steel section may be in filled with concrete to provide an enhanced strength composite section. This may require provision of shear keys throughout the length of the pile to ensure composite action between the steel tube and in-situ concrete.

Durability of the steel pile would be accorded by provision of a sacrificial steel thickness to the pile wall. The thickness would vary according to the corrosive environment.

iv) In-situ Concrete Piles

Large diameter bored reinforced concrete piles could be adopted for the Padma Bridge foundations. This option would limit the foundation to a vertical pile arrangement only. Bored pile construction uses a larger sequence of operations and is slower than driven steel pile installation. Concrete piles will be weaker under the tensile loading expected on the piles of this bridge.

The reinforcement to these piles will be cumbersome due to the minimum requirements for seismic design specified in the AASHTO Standard Specification for Highway Bridges.

(c) Reinforced Concrete Caisson

Two types of caisson have been considered for initial comparative design purposes, a tube (well) caisson and a large diameter caisson.

i) Tube Caissons

A tube or well caisson could be used to provide the foundation beneath each bridge pier, this type of foundation would have adequate capacity to carry the loading from the structure. Horizontal loading on the caisson would be resisted by lateral reactions from the ground acting on the pile shaft. Resistance to vertical loading would be taken by a combination of shaft friction and end bearing.

To carry the loads of ship impact it is estimated the caisson would need to be approximately 14 m in diameter with walls 1.8 m thick. The caisson height from standard high water level to local scour bed level would be 72 m. Using the methods developed for determining lengths of laterally loaded piles for offshore structures the caisson would have to penetrate at least 43 m below scour bed level. The caisson length would be at least 115 m and its shell weight over 8400 tonnes.

Checks on the probable deflections of a caisson of this size during ship impact loading indicates that sway movements at superstructure level of over 400 mm would occur. This would impose large forces on the superstructure. Sway and rotation can be reduced by 50% by extending the caisson down by a further 17 m, beyond this it is not possible to reduce the movements further because of the flexibility of the structure. (The caisson length would increase to 132 m.)

Two methods of installing the tube caissons have been identified. One method is to sink the caisson from the riverbed level after providing a support system, working platform and leveling the riverbed. A reaction system is required to provide the force to overcome shaft friction. The second method is to create a temporary island with a guide trench to enable bentonite to be injected beside the caisson shell to overcome the soil friction. In each case major temporary works are required and a robust logistics system to continually deliver materials and equipment out into the river.

During the works on Aricha Power Conveyor where caissons were installed one caisson suffered a blow out during sinking operations, this would be a risk if this type of foundation were to be adopted for the Padma Bridge.

Tube caissons are suitable to carry the loading and distribute into the ground. However the large deflections at bearing level under ship impact load and the works required to install them make them a costly form of construction.

ii) Large Diameter Caissons

Use of large diameter caissons was considered to check if a shorter caisson height could be used to provide the bridge foundations. For this case it was assumed bearing capacity under the base would resist the vertical loads and horizontal loads would be resisted by base friction. Outline calculations show it is not a suitable foundation unless taken to a considerable depth due to the nature of the supporting ground.

It is important that overturning and eccentric loads do not cause uplift and the consequent failure of the adjacent soils. To avoid this, the resultant of the forces must fall in the mid third of a rectangular base or mid half of a circular base. For a caisson supporting a 180 m span outline calculations indicate that a 50 m diameter structure would be required if base uplift is not to occur during a seismic event. To provide the bearing capacity required to support the self weight of the caisson alone it would have to be installed at least 30 m below the design scoured bed level to compensate for lost bearing capacity as a result of the horizontal load effects. The overall length for the caisson would then exceed 100 m.

It is considered that any gain in reducing the caisson depth is more than offset by the increase in diameter required to compensate for the loss of lateral support. Furthermore the structure must be predominantly designed for the effects of its own mass rather than the weight of the deck structure.

It is considered there are no advantages from this type of caisson as a foundation for the bridge

(d) Conclusion

It is proposed to use 3150mm diameter steel tubular raking piles to support the Padma Bridge main span piers. These will be driven at a maximum rake of 1 in 6 to the vertical and provided with a mass concrete plug or concrete fill as necessary to resist the applied loading.

(5) Methods of Construction

(a) General

The environment in which the bridge piers and foundations are to be constructed will be harsh and vary both with time and pier location. As the contractor is likely to have a preferred method of construction based on his own experience it is not the intention of this section to make recommendations. However, it is prudent to identify a number of construction methods that will allow the contractor to maintain a construction programme during changing conditions.

The following potential methods of construction have been identified for the construction of the pile cap:

- Precast shell off-site and transfer onto pile group by barge and crane.
- Precast shell and transfer onto pile group by floating / sinking.
- Precast shell above pile group and lower by jacks or cable
- Discrete precast shells craned onto single pile / small group
- In-situ construction within sheet pile cofferdam
- In-situ construction within falsework platform suspended between piles.

(b) Pile Cap

i) Precast Shell Transported by Barge and Crane.

The pile cap shell comprising base, with openings for the piles, outer walls and internal diaphragms would be constructed within a precasting yard(s) on the river bank close to the bridge site. After achieving sufficient strength the shell would be transported to the river by combination of crane and / or purpose built transporter. The pile cap shell would then be transported to the pier site by pontoon and lifted into position over the driven piles by barge mounted crane.

After sealing the openings around the piles by inflatable grout bags, or otherwise, the pile cap may be pumped dry for the continued construction.

The top of pile cap level should have sufficient freeboard above the river water level in order to maximize dry periods for working within the shell. Design of this pile cap is likely to be governed the temporary condition during transport and providing a watertight working area.

Alternatively, smaller pile cap shell sections appropriate to each pile or smaller pile group, could be prefabricated and lifted into position. The connecting slab to form continuity between the discrete shells would then be constructed above the tidal level or partly submerged within a prefabricated water tight chamber assembled during low tide.

This latter method of construction would require a greater overall depth of construction to achieve the required structural capacity between the discrete shells.

The preliminary design and detail does not assume this form of construction

ii) Precast Shell Transported by Floating and Sinking,

This option would allow the use of the pile cap as the pile guide and pile hammer support barge. The pile cap shells would be constructed in a temporary dry dock on the river bank(s). The shell would be similar to option (i) above, with additional sleeves at the pile positions extending from the soffit to deck level, together with working platforms for the construction plant. After floating the pile cap to the pier site it should be held in position using tension cables to counter tidal effects. Lateral position would also need to be assured by suitable anchors or temporary piled frames.

After completion of piling, the annulus around the piles should be sealed, the temporary sleeves cut down, and construction continued in the dry.

Design of this pile cap would be similarly governed by the temporary condition. It is probable that some channel dredging would also be required should this method be adopted for all piers.

iii) Precast Shell Construction above Pile Group

The pile cap shell could be precast above the piles, and the tidal range, supported by temporary falsework founded either on the piles or the river bed. The shell would then be lowered over the piles by either jacks under the pile cap or cables suspended above.

Construction would then proceed as in method (i) above.

iv) In-situ Pile Cap Construction within Sheet Piled Cofferdam

This standard form of construction would be more suitable for construction above water or in shallow water close to the river bank or char during the low flow season. A variation on this would be to cast a floor slab and walls allowing work to continue inside the shell. This would leave the sheet piles available for continued construction at another pier. A further variation to accelerate construction would be to construct this shell from precast segments.

v) In-situ Pile Cap Construction within Falsework Platform

A temporary platform possibly comprising a grid of ‘I’ beams, timber packing wedges and caulking would be installed after construction of the piles. A sealing system using inflatable grout bags would be required between the piles and the platform. After fixing of side panels and pumping out the river water the construction could proceed in the dry.

(c) Pier

The main options for the construction of the pier columns and cross beam are to either precast or construct in-situ. Precasting the columns and crossbeams would allow the elements to be mass produced in the controlled environment of a precast yard rather than the remote pier site. Their detailing should allow an in-situ concrete stitch to the superstructure, and socket connection to the pile cap. This method of construction would also allow construction work to proceed during times when access to the pier site may be difficult due to high wind or river flow.

In-situ construction would require the transport of all formwork, falsework, reinforcement, wet concrete, labour and support facilities to each pier site for a sustained period. However, facilities for transporting these resources would already be provided as it is anticipated that the superstructure will be constructed in-situ and the pile cap constructed part in-situ.

(d) Abutment

The same options are applicable to the construction of the abutments. However, as these sites are readily accessible onshore there is little advantage to precasting these elements.

3.3.3 Preferable Span Length

(1) Procedures for Selection of Preferable Span Length

Construction costs per one meter of span length were estimated and compared in order to find out the most preferable span length.

Such procedures as described below were followed.

a) As each superstructure type has its own suitable range of span length as mentioned above in 3.3.1, combinations of the types and the span length were made as shown in the table below.

SPAN LENGTH	Continuous Box	Extradosed	Cable Stayed
100m	○		
120m	○		
140m	○	○	
160m	○	○	
180m		○	○
200m		○	○
220m			○
240m			○

b) Those combinations with "○"mark in the table were examined.
c) Quantities of each combination required for a span of the superstructure were estimated based on the past data regarding the bridges of similar structure.
d) Quantities required for one substructure including foundation piles were calculated.
e) Cost for the superstructure and the substructure were estimated and then divided by the span length and summed. Thus the total costs per meter were obtained.
f) The same procedures were repeated for the both cross-sections "without railway provision" and "with railway provision" described in 3.2.1.

(2) Considerations on the Results

(a) Cross-section without Railway Provision

The Span-Cost Graph in the Figure 3.3.7 shows the span lengths and the total unit costs per meter of super- and substructure of the cross-section without railway provision, resulted from the procedures described above.

The longer the span is, then the higher the unit cost for superstructure becomes almost linearly for each superstructure type. It happens because a longer span requires heavier cross section to resist a bending moment that is proportional to the square of the span length.

On the other hand, unit substructure costs decrease as the span becomes longer but the decreasing grade becomes dull, or becomes even higher in some case. This may be understood in such a manner that a number of substructures decreases and thus the unit cost may also be reduced as the span is longer to some extent, but at the same time the quantities required to compose one set of substructure becomes more. Consequently the unit cost for the substructure tends to drops but not linearly.

The minimum of the total unit cost, which is a summation of the super- and substructure unit cost, come out at 180 meters of the span lengths for an extradosed girder bridge type. This tendency of the result looks very similar to what was reported in the Interim Report.

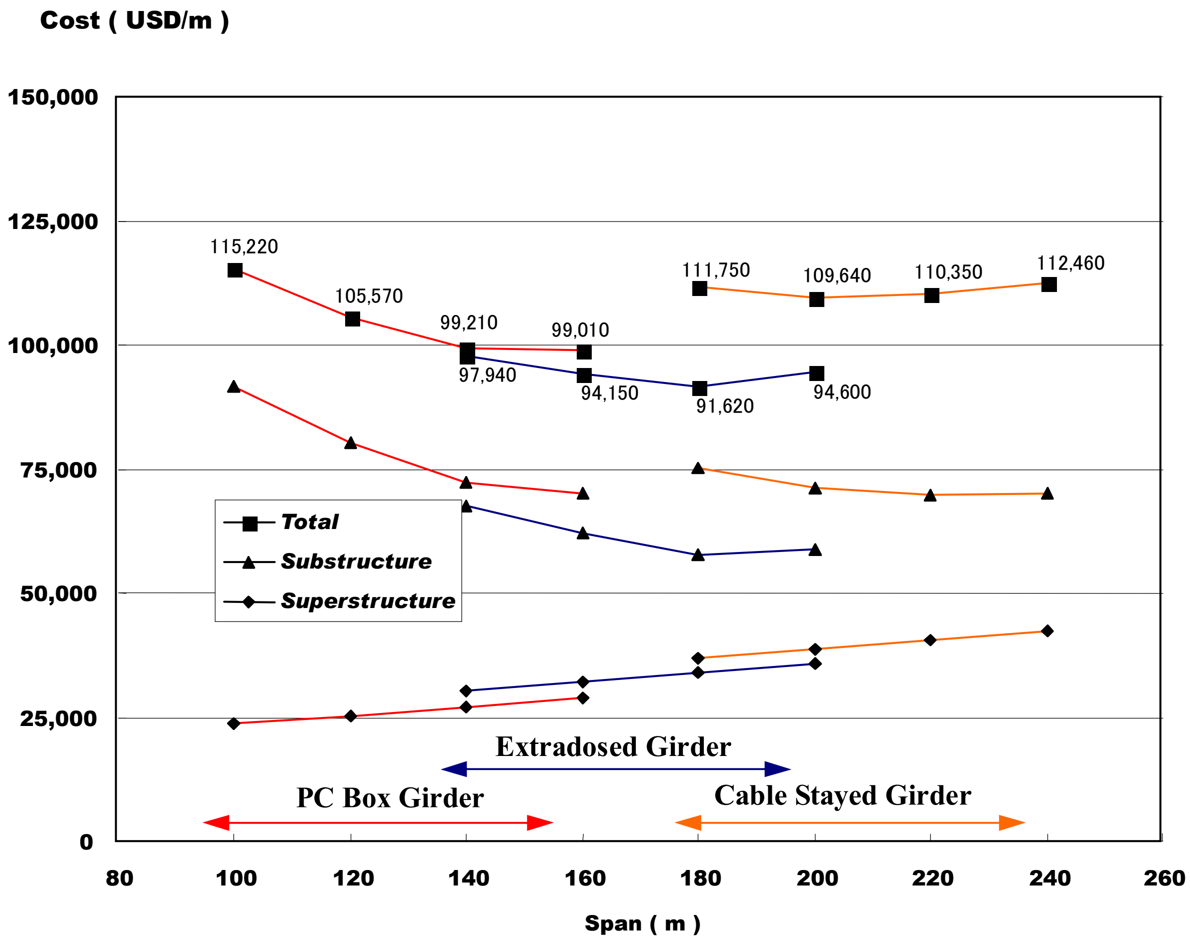
(b) Cross-section with Railway Provision

As is seen in the Figure 3.3.8, the one with Railway Provision has exactly similar tendency as that without Railway Provision, that is, 180 meters span length of extradosed girder costs the minimum among others.

The unit cost with railway is 13% higher than that without Railway.

(c) Conclusion from the comparison

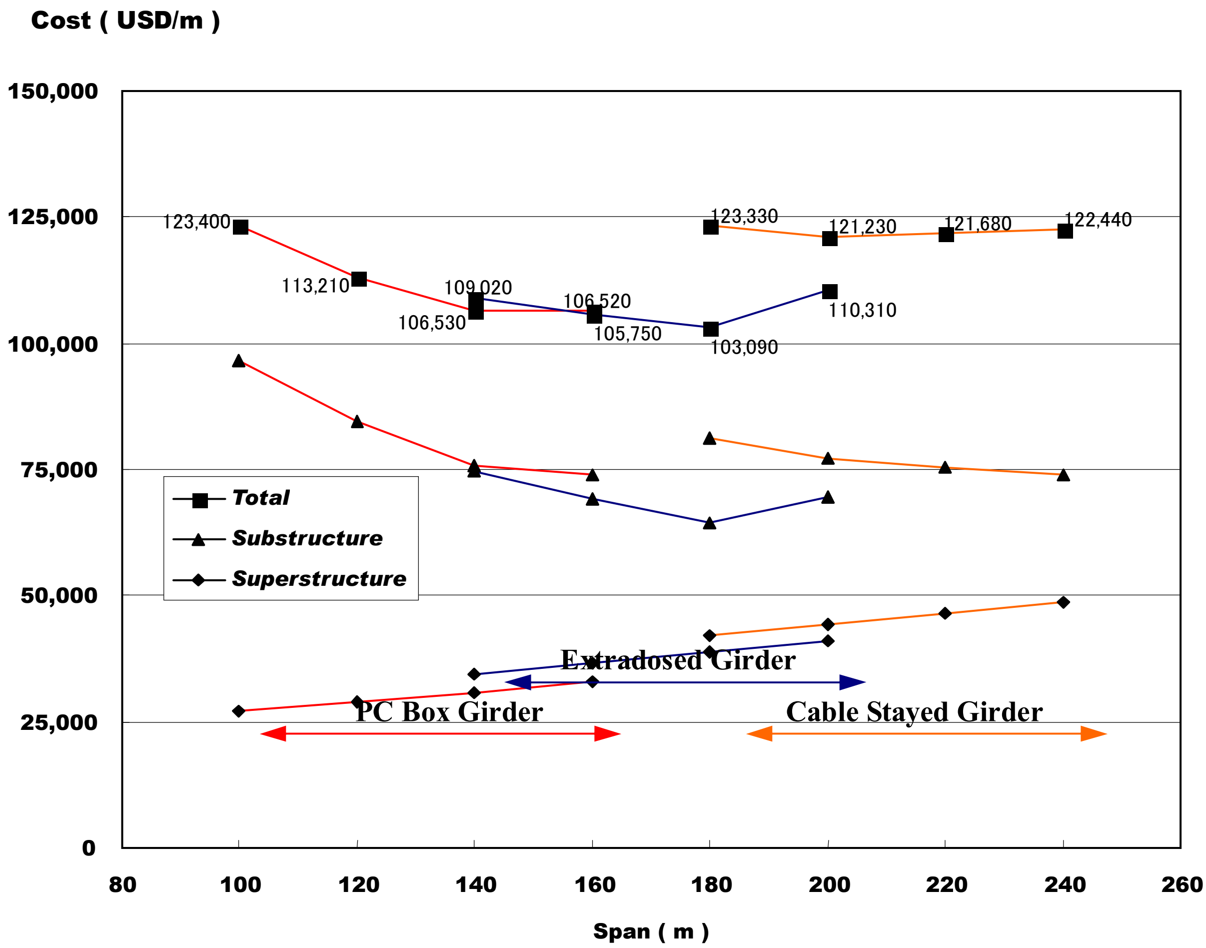
The preliminary design will be conducted for the span of 180 meters, and an extradosed girder bridge is to be regarded as the standard superstructure type for the main bridge over the Padma.



(US$/m)

Span (m)	Bridge Type	Super-St.	Sub-St.	Total
100	Box Girder	23,740	91,480	115,220
120		25,350	80,220	105,570
140		27,030	72,180	99,210
160		28,780	70,230	99,010
140	Ex. Girder	30,240	67,700	97,940
160		32,030	62,120	94,150
180		33,830	57,790	91,620
200		35,760	58,840	94,600
180	CS. Girder	36,700	75,050	111,750
200		38,590	71,050	109,640
220		40,500	69,850	110,350
240		42,450	70,010	112,460

Figure 3.3.7 Cost-Span Graph for Cross-section without Railway



(US$/m)

Span (m)	Bridge Type	Super-St.	Sub-St.	Total
100	Box Girder	26,930	96,470	123,400
120		28,820	84,390	113,210
140		30,770	75,760	106,530
160		32,790	73,730	106,520
140	Ex. Girder	34,440	74,580	109,020
160		36,520	69,230	105,750
180		38,660	64,430	103,090
200		40,870	69,440	110,310
180	CS. Girder	41,930	81,400	123,330
200		44,150	77,080	121,230
220		46,360	75,320	121,680
240		48,580	73,860	122,440

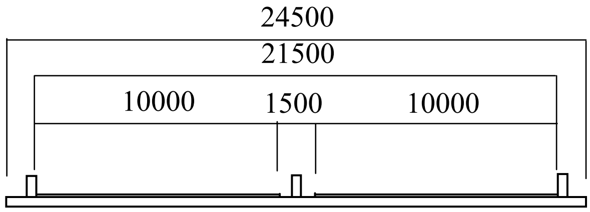
Figure 3.3.8 Cost-Span Graph for Cross-section with Railway

3.3.4 Preliminary Design of Extradosed Bridges

Based on the results from the preferable span length and superstructure type, and also details of the stay cables and pylon studied above, preliminary design of the extradosed bridges with and without railway have been drawn.

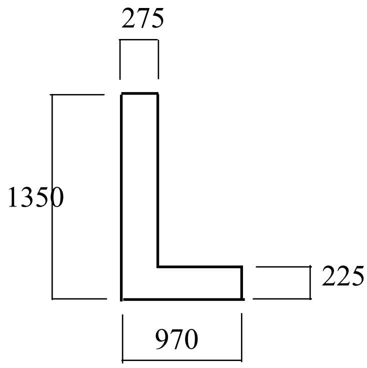
(1) Major loads considered for preliminary design of superstructure

(a) Loads for Base Case (Without railway)



i) Dead loads

a) Asphalt pavement
w =2x10.0x0.075x22.5=33.8 kN/m
b) Curbs
A =0.275x1.35+0.225x(0.97-0.275)=0.528m^2
w =2x0.528x24.5=25.9 kN/m

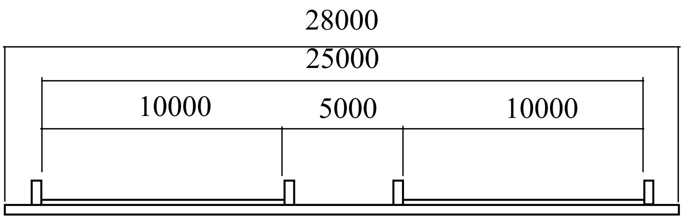


c) Separator
w = 0.528 x 24.5 =12.9 kN/m
d) Self weight
Assumed self weight for 180m span is
w = 22.5 x (0.0014 x 180 + 0.8268) x 24.5 = 594.7 kN/m
e) Utilities and tray
400kV power cable, natural gas pipeline and communication cables are assumed to weigh 20.0kN/m, tray for the power cables attached beside the deck is 22.8 kN/m

ii) Live loads

i) Vehicle Loads (AASHTO HS20-44)
UDL =4x0.75x9.4 =28.2 kN/m
KEL =4x0.75x80=240kN
Impact = 15.24/(L+38) L: m

(b) Loads for Railway Provision Case

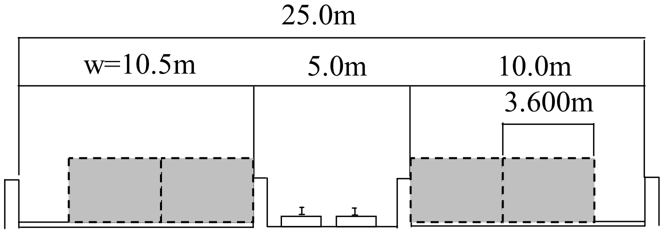


i) Dead loads

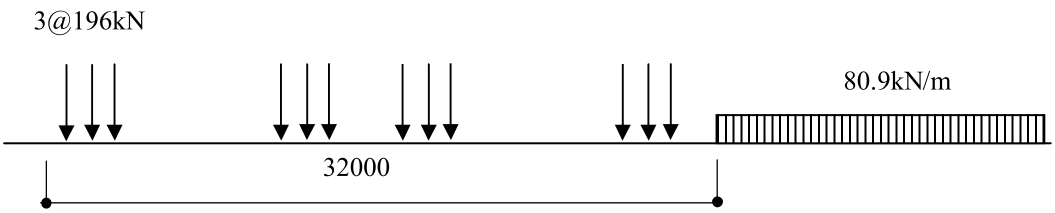
a) Asphalt pavement
w =2x10.0x0.075x22.5=33.8 kN/m
b) Curbs
A =0.275x1.35+0.225x(0.97-0.275)=0.528 m^2
w =2x0.528x24.5=25.9 kN/m
c) Separator
w =2x0.528x24.5=25.9 kN/m
d) Self weight
Assumed self weight for 180m span is
w =26.0x(0.0014x180+0.8268)x24.5=687.2 kN/m
e) Utilities and tray
400kV power cable, natural gas pipeline and communication cables are assumed to weigh 20.0kN/m, tray for the power cables attached beside the deck is 22.8 kN/m
f) Railway facilities
w =18.0 kN/m

ii) Live loads

1) Vehicle Loads (AASHTO HS20-44)
UDL =4x0.75x9.4 =28.2 kN/m
KEL =4x0.75x80=240kN
Impact = 15.24/(L+38) L: m



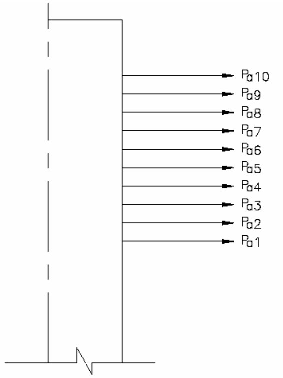
2) Railway Loads (Indian Railway Standards)



Impact = 19.8/(L+13.7) L: m

(c) Cable Prestressing

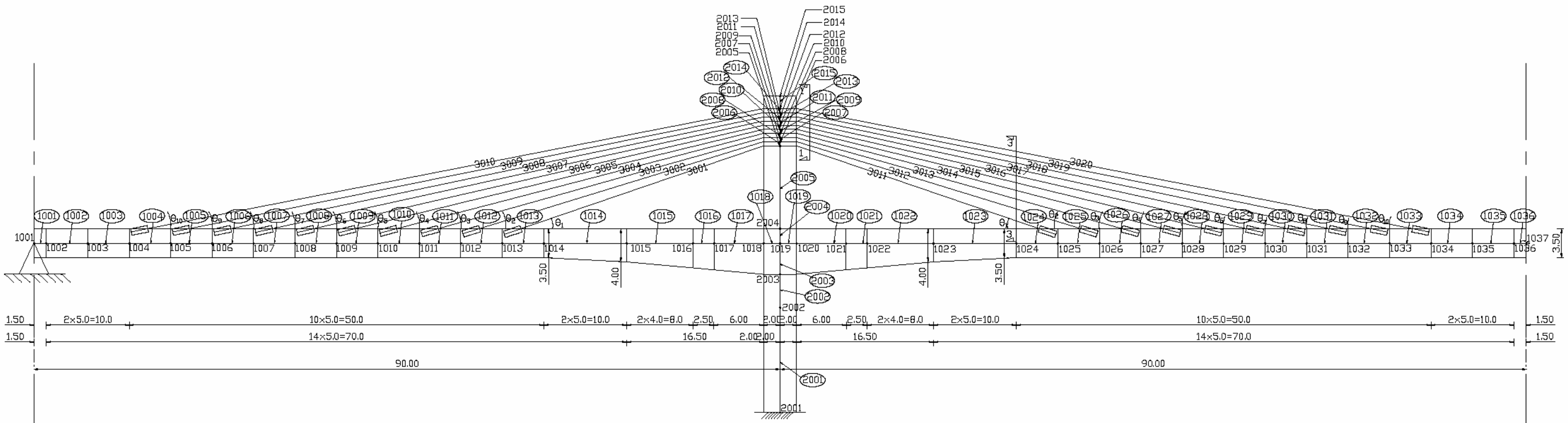
Cable tensioning forces introduced into each stay cable are shown in the Figure 3.3.9 and 3.3.10 for the Base Case and the Railway Provision Case respectively.



EXTRADOZED CABLE PRESTRESSING FORCE

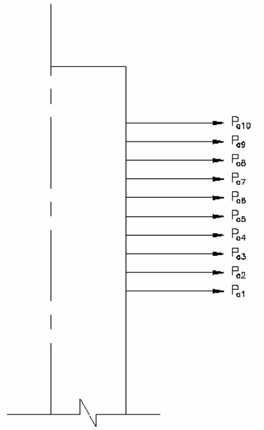
CABLE SLOPE (θ in degree)		Pa(Horizontal) in kN			CABLE ECENTRICITY SEC:1−1 (e in m)	MOMENT (PahXe in kN.m)	CABLE ECENTRICITY SEC:3−3 (e in m)	MOMENT (PahXe in kN.m)
θ_1	18.87°	Pa1	16913	16004	12.82	205171	2.013	32216
θ_2	17.04°	Pa2		16171	13.32	215397	3.493	56485
θ_3	15.56°	Pa3		16293	13.82	225169	4.723	76952
θ_4	14.56°	Pa4		16370	14.32	234418	5.788	94750
θ_5	13.69°	Pa5		16433	14.82	243537	6.738	110726
θ_6	12.97°	Pa6		16482	15.32	252504	7.605	125346
θ_7	12.38°	Pa7		16520	15.82	261346	8.41	138933
θ_8	11.87°	Pa8		16551	16.32	270112	9.168	151740
θ_9	11.43°	Pa9		16578	16.82	278842	9.889	163940
θ_{10}	11.05°	Pa10		16599	17.32	287495	10.58	175617
Total=				164001		2473993		1126704

NB. 1) Cable 1~10 2 x 27T15.2 SWPR7BL
Pa=0.6 X 2 X 7047 = 8456.4 kN
2) For both side Pa= 8456 x 2= 16913 kN



FRAME MODEL FOR THE EXTRADOZED BRIDGE

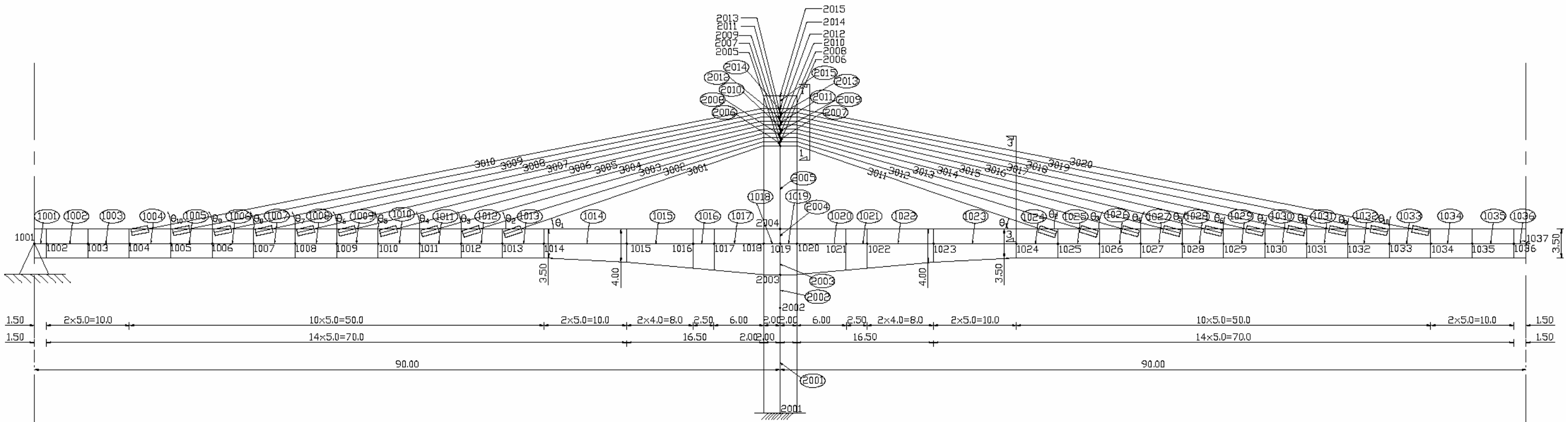
Figure 3.3.9 Cable Prestressing (Base Case)



EXTRADOZED CABLE PRESTRESSING FORCE

CABLE SLOPE (θ in degree)		Pa(Horizontal) in kN			CABLE ECENTRICITY SEC:1−1 (e in m)	MOMENT (Pah x e in kN.m)	CABLE ECENTRICITY SEC:3−3 (e in m)	MOMENT (PahXe in kN.m)
θ_1	18.87°	P_{a1}	23177	21931	12.82	281155	2.013	44147
θ_2	17.04°	P_{a2}		22160	13.32	295171	3.493	77405
θ_3	15.56°	P_{a3}		22328	13.82	308573	4.723	105455
θ_4	14.56°	P_{a4}		22433	14.32	321241	5.788	129842
θ_5	13.69°	P_{a5}		22519	14.82	333732	6.738	151733
θ_6	12.97°	P_{a6}		22586	15.32	346018	7.605	171767
θ_7	12.38°	P_{a7}		22638	15.82	358133	8.41	190386
θ_8	11.87°	P_{a8}		22681	16.32	370154	9.168	207939
θ_9	11.43°	P_{a9}		22717	16.82	382100	9.889	224648
θ_{10}	11.05°	P_{a10}		22747	17.32	393978	10.58	240663
Total=				224740		3390254		1543986

NB. 1) Cable 1~10 2 x 37T15.2 SWPR7BL
Pa=0.6 X 2 X 9657 = 11588.4 kN
2) For both side Pa= 11588.4 x 2= 23177 kN



FRAME MODEL FOR THE EXTRADOZED BRIDGE

Figure 3.3.10 Cable Prestressing (Railway Provision Case)

(2) Forces acting in each element

(a) Base Case

a) Cable tensions
Cable tension forces due to the dead load, flooring load, live load and cable prestressing are shown in the Table 3.3.7.
b) Bending moments and normal forces are shown in the Figure 3.3.11 ~ Figure 3.3.15
c) Reaction forces
Reaction forces working at the pile cap are shown in the Table 3.3.8 and Table 3.3.9

The live load acting in the Table 3.3.8 is fully distributed over the length of the deck, on the other hand the Table 3.3.9 is due to the partially distributed live load.

(b) Railway Provision Case

a) Cable tensions
Cable tension forces due to the dead load, flooring load, live load and cable prestressing are shown in the Table 3.3.10.
b) Bending moments and normal forces are shown in the Figure 3.3.16~3.3.20
c) Reaction forces
Reaction forces working at the pile cap are shown in the Table 3.3.11 and Table 3.3.12

The live load acting in the Table 3.3.11 is fully distributed over the length of the deck, on the other hand the Table 3.3.12 is due to the partially distributed live load.

Table 3.3.7 Cable Tension Forces

Cable No.	D.L (kN)	Flooring (kN)	L.L (kN)	CablePrestressing (Induced) (kN)	Total (kN)
3001	3977	829	204	10177	15187
3002	4387	921	235	9821	15364
3003	4752	1003	262	9513	15530
3004	5082	1077	288	9258	15705
3005	5371	1142	310	9056	15879
3006	5617	1198	329	8903	16047
3007	5824	1245	346	8792	16207
3008	5994	1283	360	8716	16353
3009	6133	1315	372	8669	16489
3010	6242	1340	381	8645	16608
3011	4367	917	154	9893	15331
3012	4526	954	161	9686	15327
3013	4637	981	170	9534	15322
3014	4704	999	180	9443	15326
3015	4725	1006	191	9411	15333
3016	4702	1003	202	9431	15338
3017	4638	991	213	9496	15338
3018	4537	971	223	9598	15329
3019	4404	944	234	9732	15314
3020	4242	910	244	9891	15287
3101	3639	756	65	10620	15080
3102	3801	793	63	10469	15126
3103	3907	819	64	10376	15166
3104	3962	834	66	10349	15211
3105	3967	837	68	10381	15253
3106	3926	830	72	10463	15291
3107	3842	814	75	10587	15318
3108	3722	790	79	10745	15336
3109	3570	758	84	10930	15342
3110	3390	720	150	11135	15395
3111	3641	759	163	10611	15174
3112	3816	800	171	10419	15206
3113	3935	828	175	10290	15228
3114	4000	845	174	10229	15248
3115	4013	851	171	10231	15266
3116	3977	845	163	10287	15272
3117	3897	830	153	10389	15269
3118	3779	806	141	10529	15255
3119	3627	775	126	10699	15227
3120	3446	737	117	10893	15193

<16913 kN

The Maximum Tensile Force can be Sustained by the current

Extradosed cables (2x27T15.2SWBR7BL x 2sides)

= 0.6 x Pu

= 0.6 x 2cables x 7047 kN x 2sides=16913kN

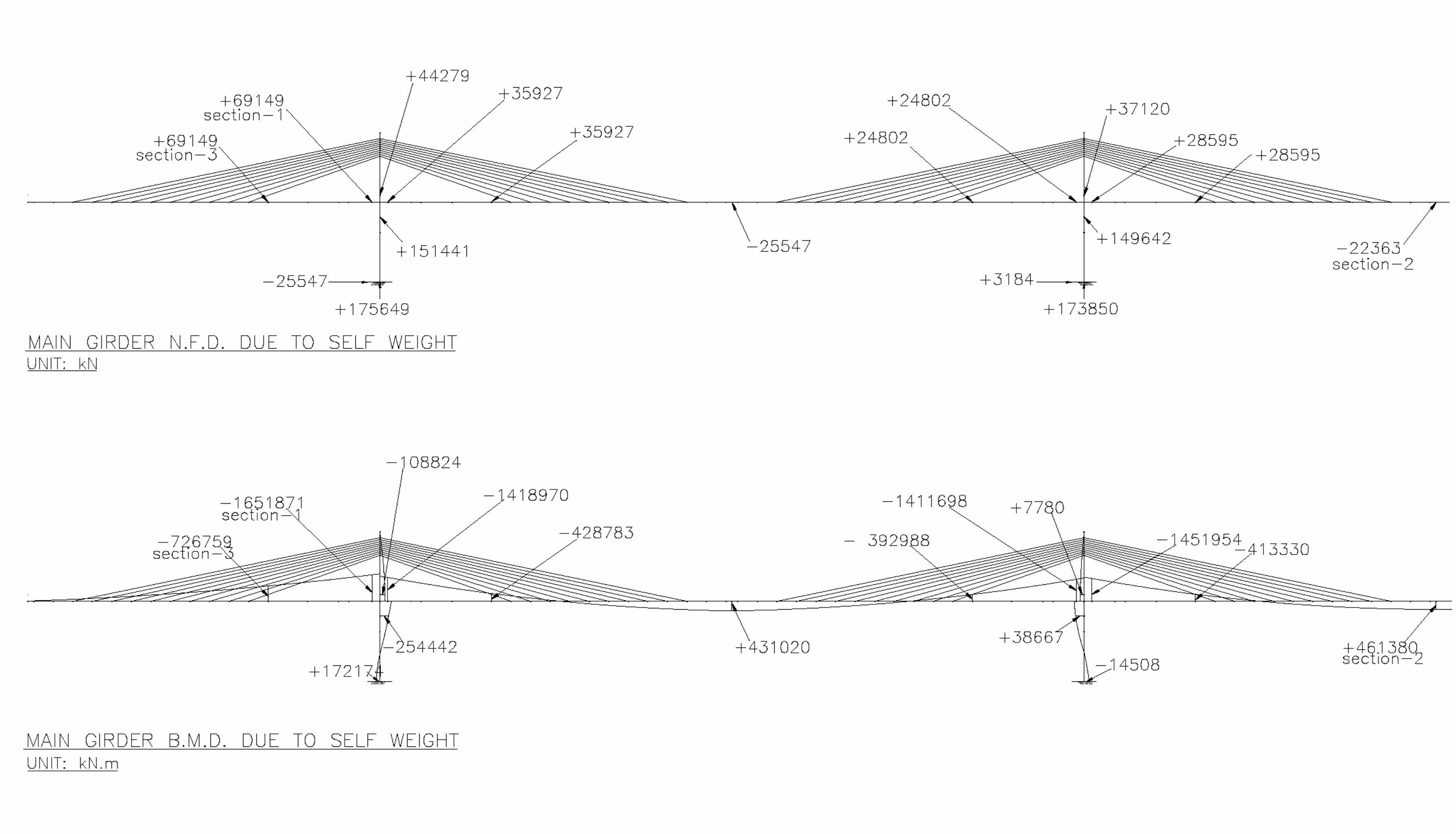


Figure 3.3.11 Bending Moment and Normal Force Due to Self Weight (Base Case)

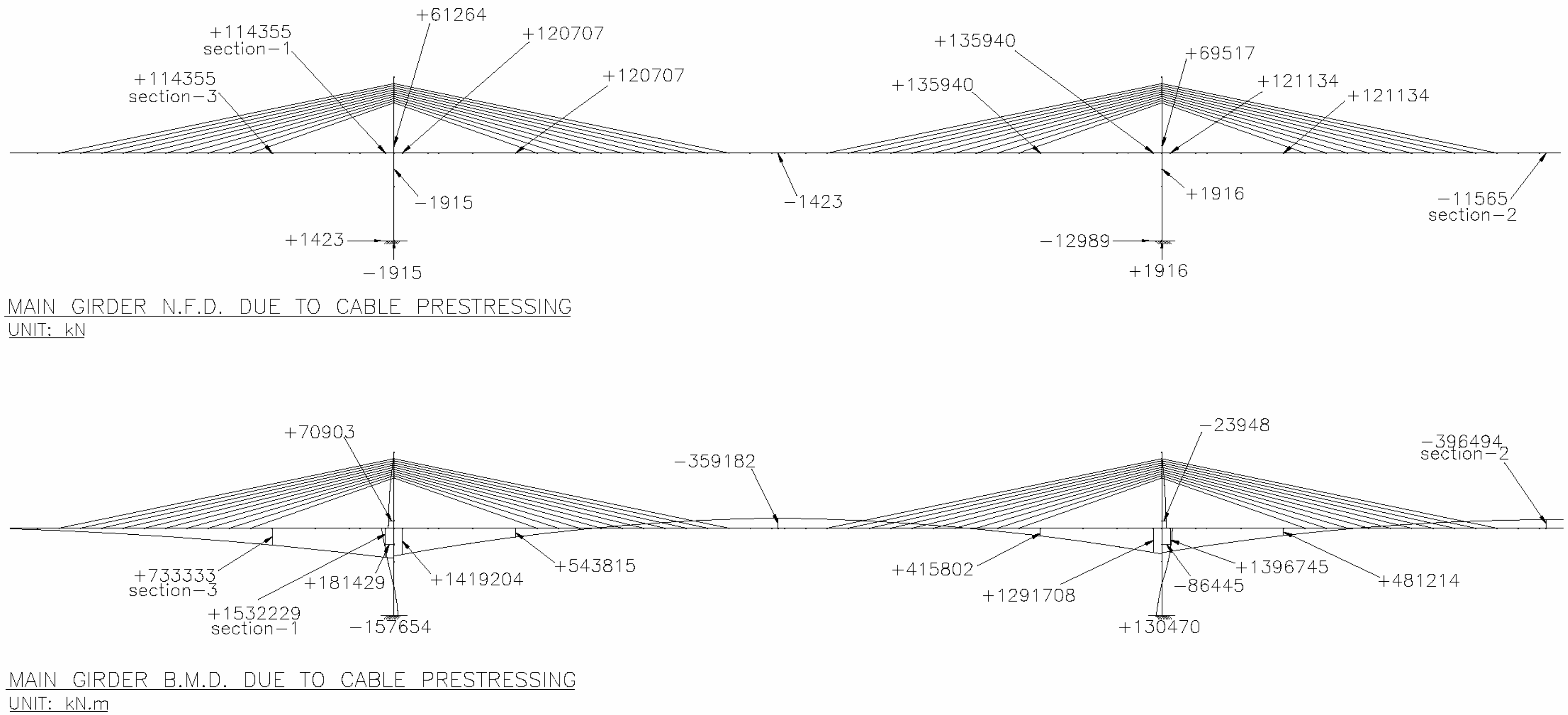


Figure 3.3.12 Bending Moment and Normal Force Due to Cable Prestressing (Base Case)

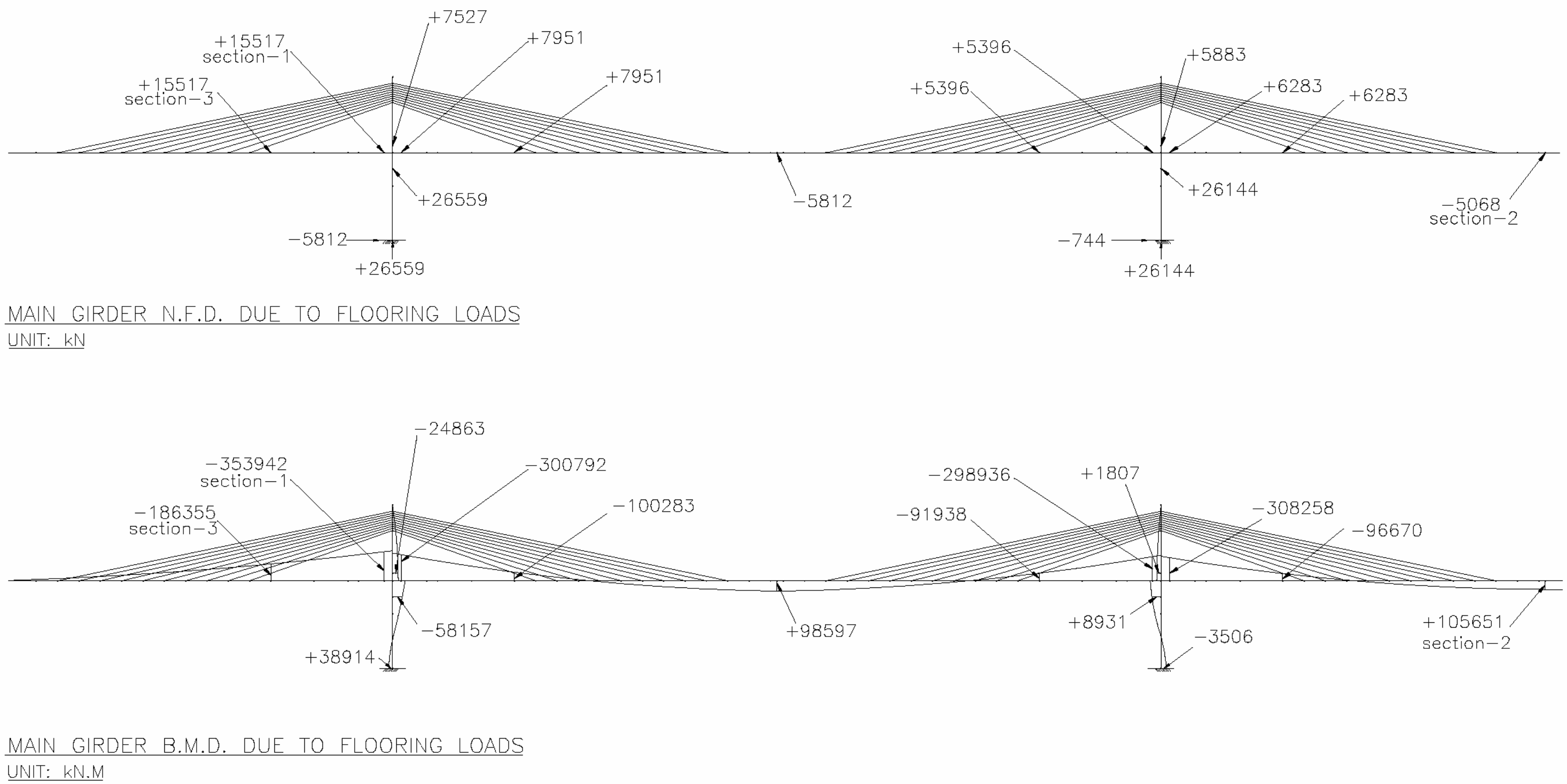
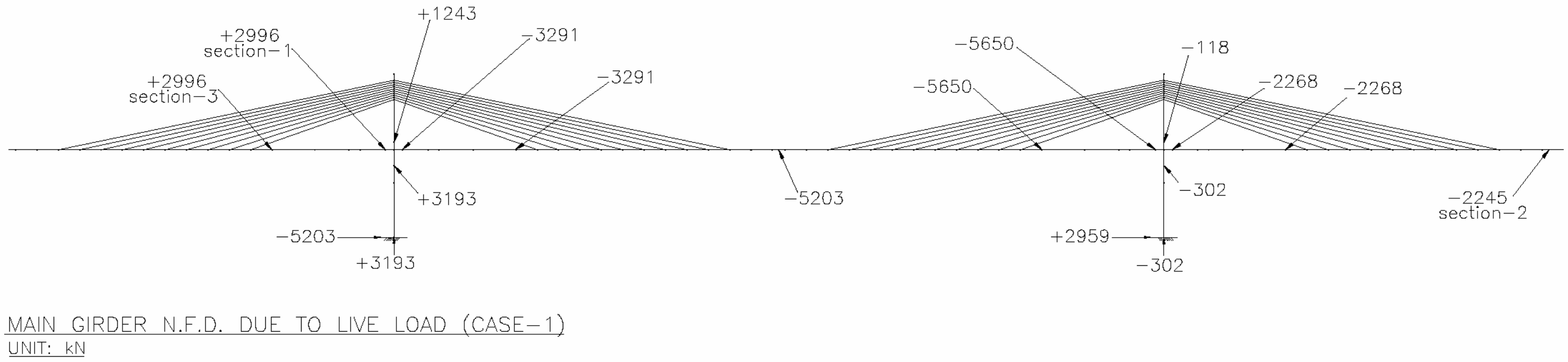


Figure 3.3.13 Bending Moment and Normal Force Due to Flooring Loads (Base Case)



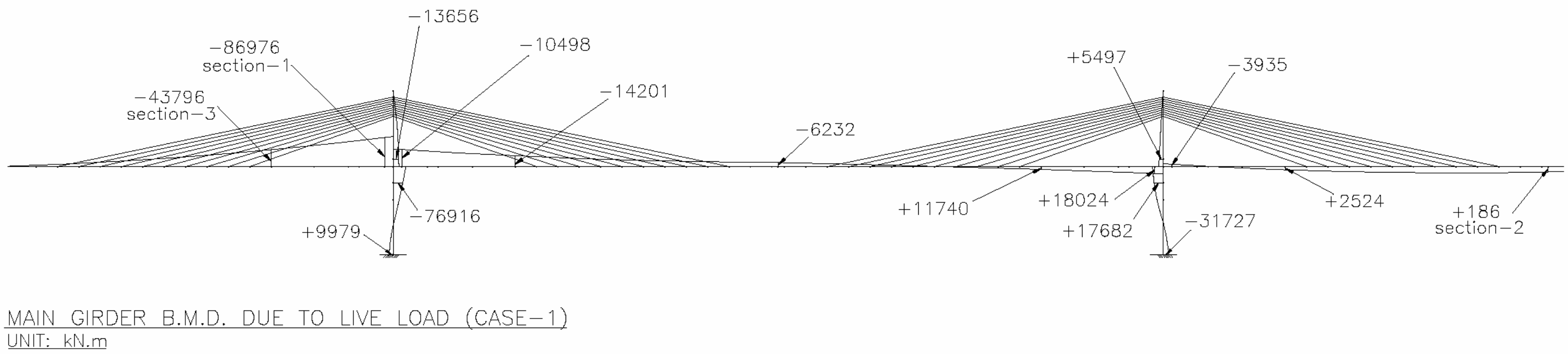


Figure 3.3.14 Bending Moment and Normal Force Due to Live Load-Case 1 (Base Case)

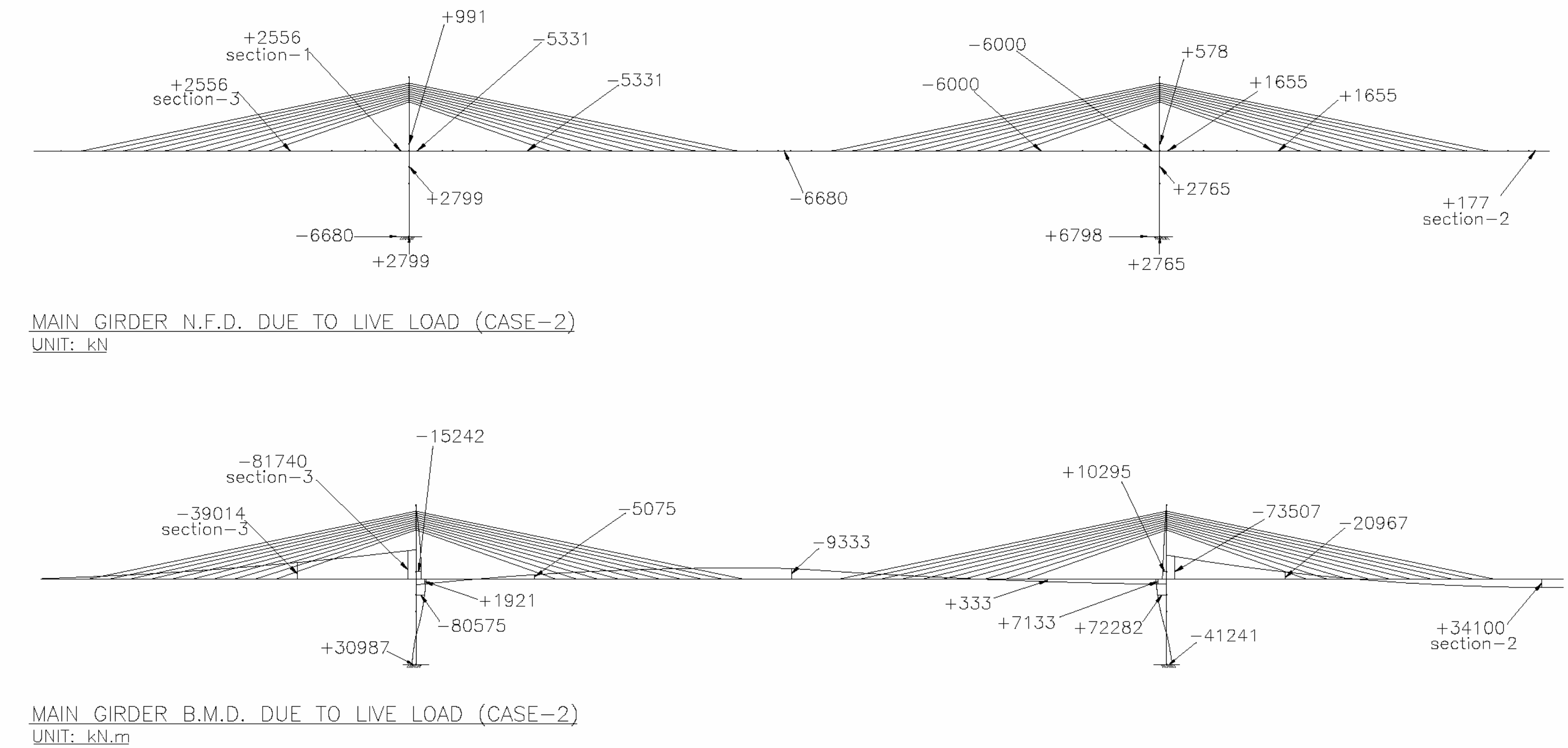
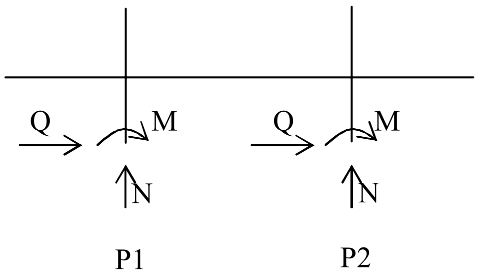


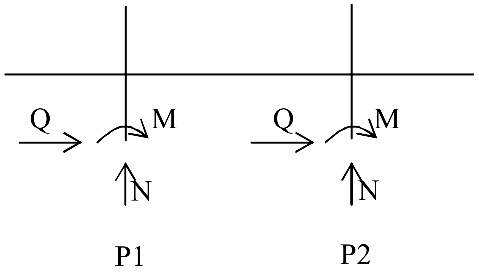
Figure 3.3.15 Bending Moment and Normal Force Due to Live Load-Case 2 (Base Case)

Table 3.3.8 Reaction Forces at Pile Cap Level (Live load Case 1)



Pile	Self Weight			Cable Prestressing			Flooring			Live Load			Total		
	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)
P1	151606	-	-	-1346	-	-	20925	-4814	30097	5463	-1337	8582	176648	-6151	38679
P2	150201	-	-	1346	-	-	20618	-629	-3684	5573	525	-2860	177738	-104	-6544

Table 3.3.9 Reaction Forces at Pile Cap Level (Live load Case 2)



Pile	Self Weight			Cable Prestressing			Flooring			Live Load			Total		
	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)
P1	151606	-	-	-1346	-	-	20925	-4814	30097	2799	-6680	30987	173984	-11494	61084
P2	150201	-	-	1346	-	-	20618	-629	-3684	2765	6798	-41241	174930	6169	-44925

Table 3.3.10 Cable Tension Forces

Cable No.	D.L (kN)	Flooring (kN)	L.L (kN)	Cable Prestressing (Induced) (kN)	Total (kN)	
3001	5259	1153	819	13428	20659	
3002	5823	1286	945	12874	20928	
3003	6326	1405	1058	12396	21185	
3004	6785	1514	1160	11998	21457	
3005	7189	1610	1250	11684	21733	
3006	7537	1693	1328	11445	22003	
3007	7830	1763	1394	11271	22258	
3008	8074	1821	1449	11153	22497	
3009	8274	1869	1495	11079	22717	
3010	8435	1908	1532	11041	22916	**< 23177**
3011	6091	1348	686	12794	20919	**kN**
3012	6304	1401	719	12470	20894	
3013	6455	1440	761	12229	20885	
3014	6548	1466	810	12082	20906	
3015	6581	1477	863	12027	20948	
3016	6555	1474	918	12053	21000	
3017	6473	1459	973	12150	21055	
3018	6341	1431	1028	12307	21107	
3019	6163	1393	1082	12513	21151	
3020	5946	1345	1133	12760	21184	
3101	5024	1099	301	13926	20350	
3102	5247	1154	291	13690	20382	
3103	5396	1192	290	13545	20423	
3104	5475	1214	295	13503	20487	
3105	5487	1220	304	13554	20565	
3106	5434	1211	318	13685	20648	
3107	5323	1188	335	13883	20729	
3108	5161	1154	355	14135	20805	
3109	4954	1109	378	14430	20871	
3110	4709	1054	403	14757	20923	
3111	5011	1101	611	13968	20691	
3112	5254	1160	664	13663	20741	
3113	5419	1202	697	13457	20775	
3114	5512	1228	710	13360	20810	
3115	5533	1236	705	13362	20836	
3116	5487	1229	684	13451	20851	
3117	5380	1208	648	13614	20850	
3118	5219	1174	599	13838	20830	
3119	5009	1128	538	14111	20786	
3120	4759	1073	469	14422	20723	

The Maximum Tensile Force can be Sustained by the current
Extradozed cables (2x37S15.2 x 2sides)

= 0.6 x Pu

= 0.6 x 2cables x 9657 kN x 2sides=23177 kN

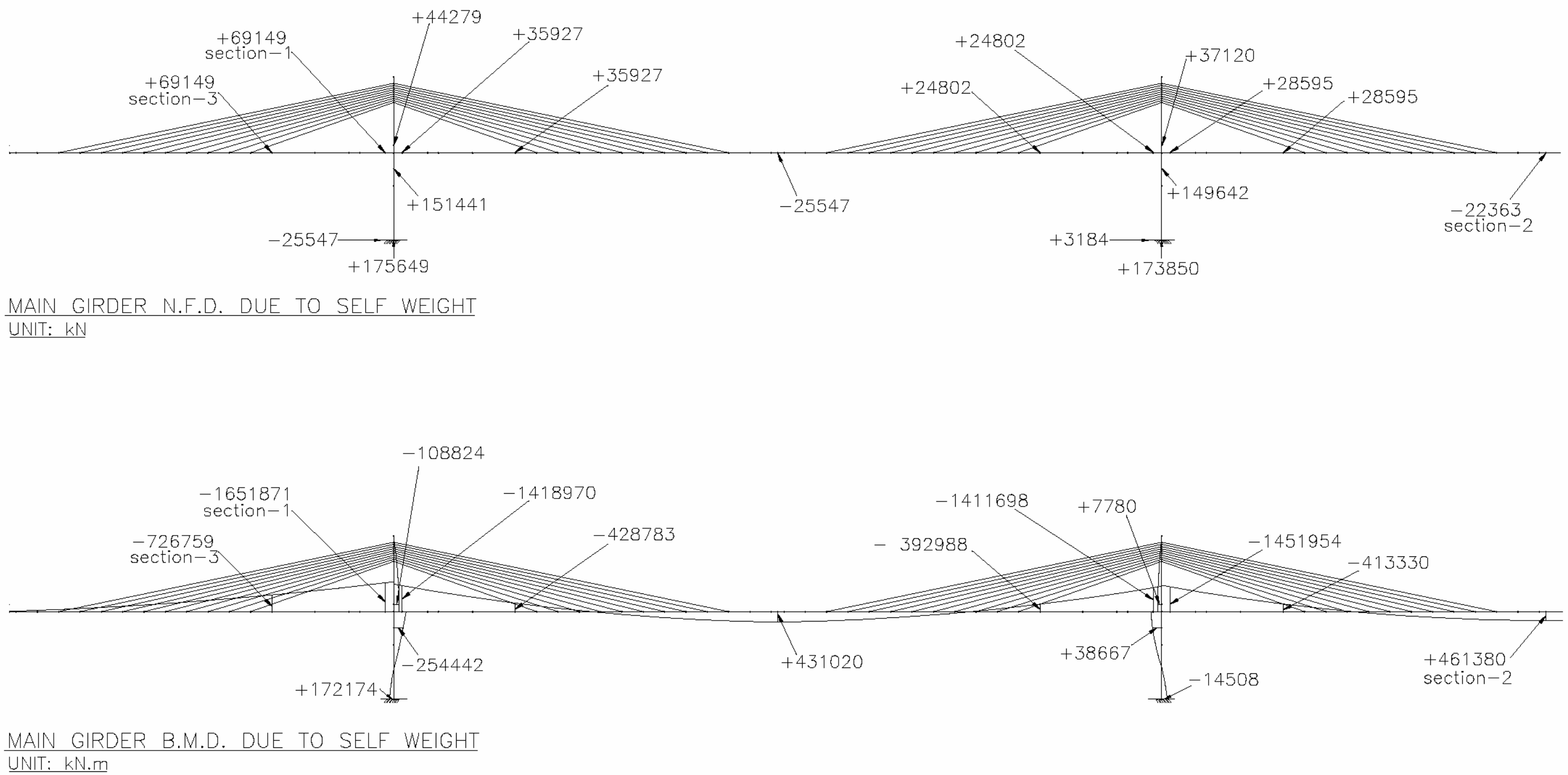


Figure 3.3.16 Bending Moment and Normal Force Due to Self Weight

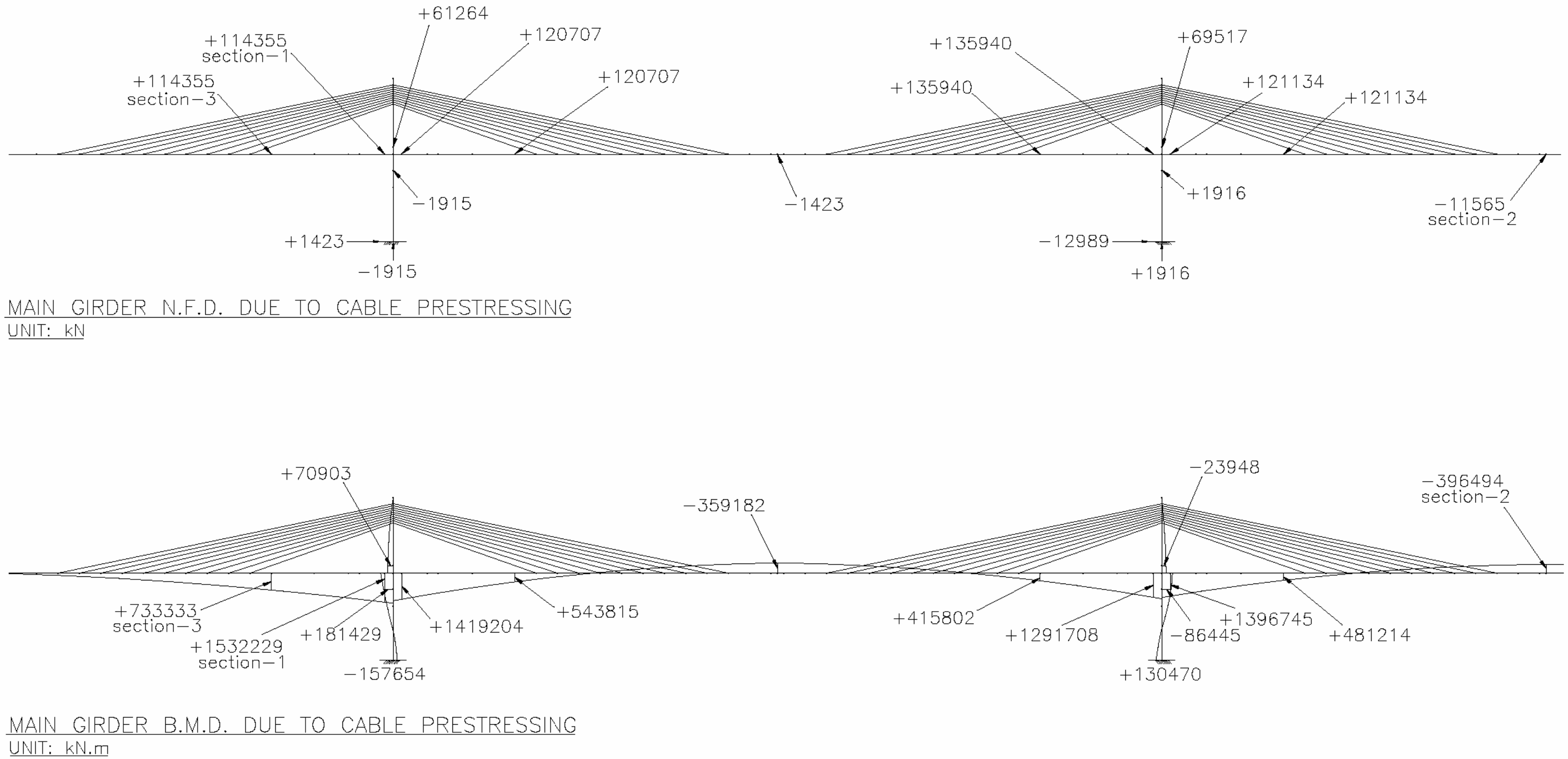


Figure 3.3.17 Bending Moment and Normal Force Due to Cable Prestressing

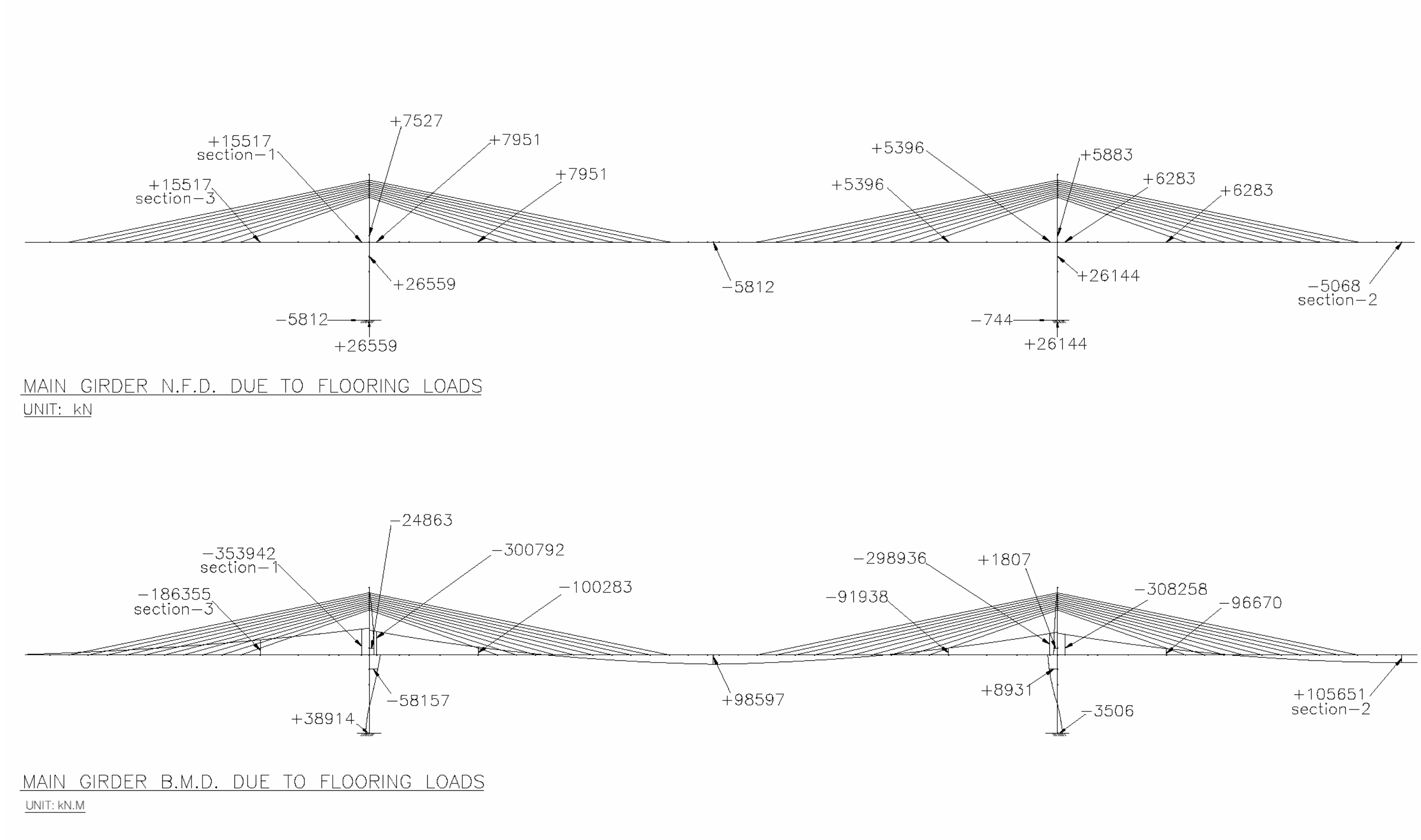
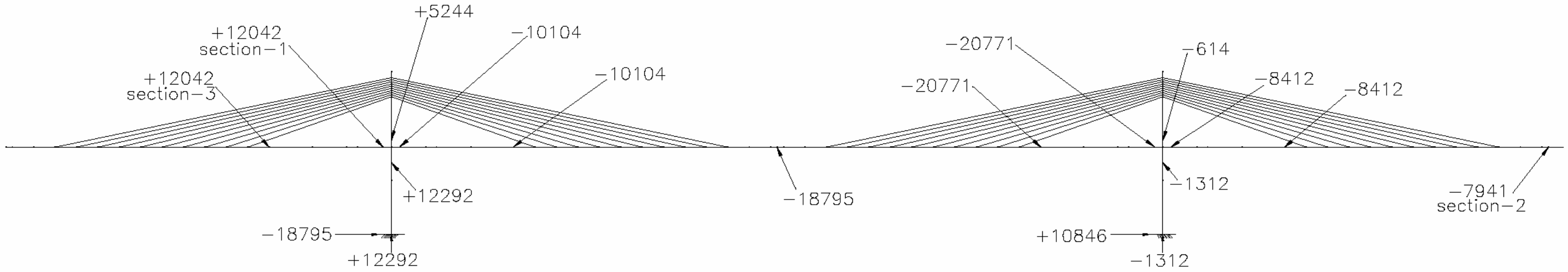
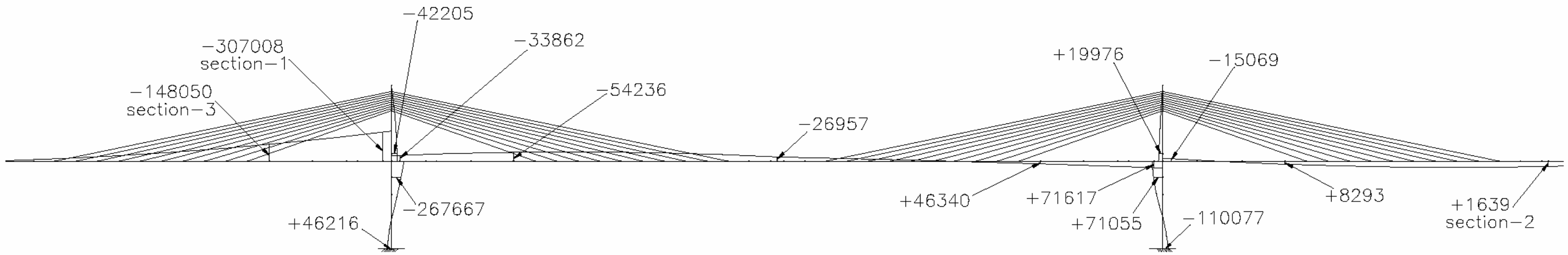


Figure 3.3.18 Bending Moment and Normal Force Due to Flooring Loads

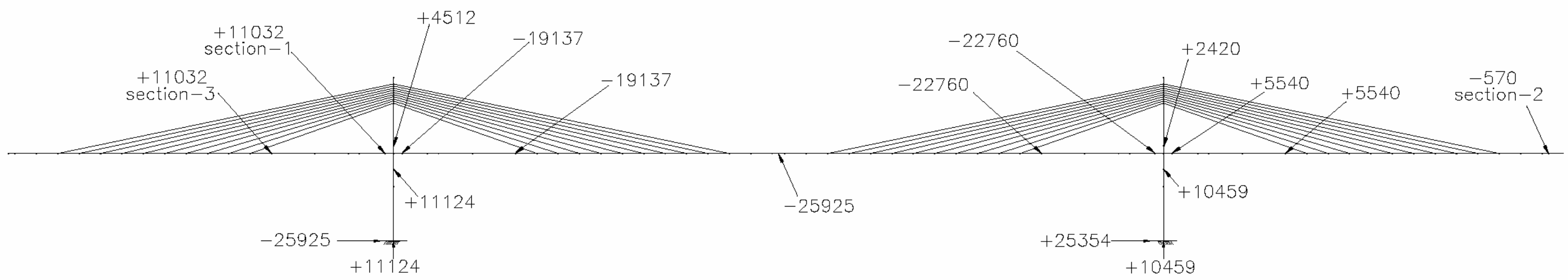


MAIN GIRDER N.F.D. DUE TO LIVE LOAD (CASE−1)
UNIT: kN

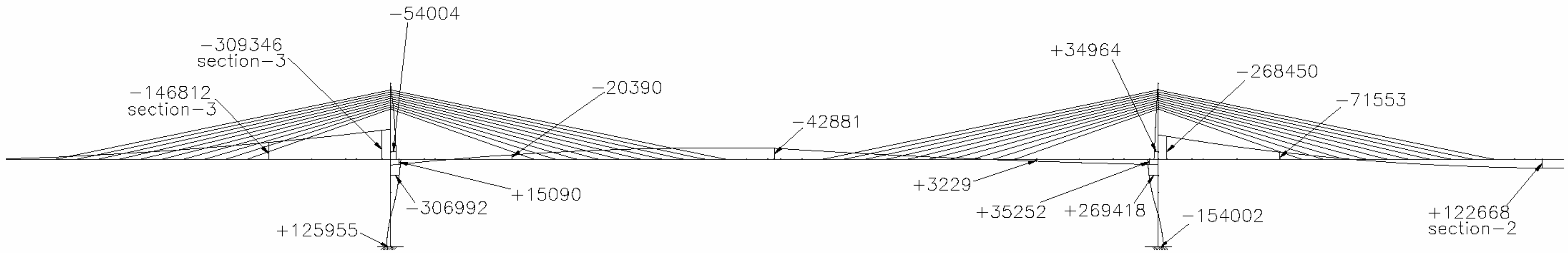


MAIN GIRDER B.M.D. DUE TO LIVE LOAD (CASE−1)
UNIT: kN.m

Figure 3.3.19 Bending Moment and Normal Force Due to Live Load (Case-1)



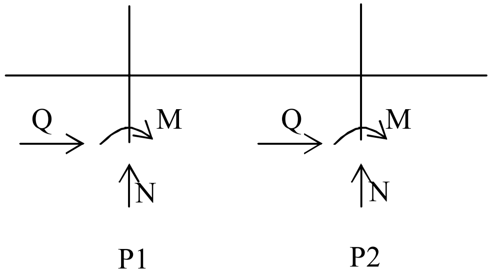
MAIN GIRDER N.F.D. DUE TO LIVE LOAD (CASE−2)
UNIT: kN



MAIN GIRDER B.M.D. DUE TO LIVE LOAD (CASE−2)
UNIT: kN.m

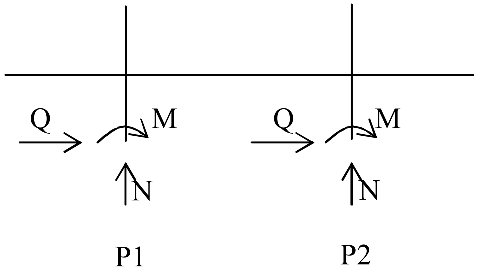
Figure 3.3.20 Bending Moment and Normal Force Due to Live Load (Case-2)

Table 3.3.11 Reaction Forces at Pile Cap Level (Live load Case - 1)



Pile	Self Weight			Cable Prestressing			Flooring			Live Load				Total	
	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)
P1	175649	-	-	-1915	-	-	26559	-5812	38914	21610	-4815	32377	221903	-10627	71291
P2	173850	-	-	1916	-	-	26144	-744	-3506	21430	954	-4723	223340	210	-8229

Table 3.3.12 Reaction Forces at Pile Cap Level (Live load Case 2)



Pile	Self Weight			Cable Prestressing			Flooring			Live Load			Total		
	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)	N (kN)	Q (kN)	M (kN.m)
P1	175649	-	-	-1915	-	-	26559	-5812	38914	11124	-25925	125955	211417	-31737	164869
P2	173850	-	-	1916	-	-	26144	-744	-3506	10459	25354	-154002	212369	24610	-157508

(3) Deflection angle check of the railway provision case

As the slope that a train can climb is limited to 1% the deflection angle due to the live loads were checked.

$\theta = \tan^{-1}[(0.387\text{-}0.378)/5.0]$
$= 0.34^0$
$= 0.6\ \% < 1.0\ \%$ OK

The deflection diagram is shown by the Figure 3.3.21

(4) Deck section and prestressing cable arrangements

The Figure 3.3.22 and Figure 3.3.23 Show designed typical cross-sections for the Base Case and the Railway Provision Case respectively.

(5) Drawings of Extradosed Bridge

The Figure 3.3.24 and Figure 3.3.25 give the extradosed structure for the Base Case and the Railway Provision Case.

(6) Quantities of major materials

The Figure 3.3.26 and Figure 3.3.27 give the quantities of stay cable and the deck materials for the Base Case, Figure 3.3.28 and the Figure 3.3.29 are for the Railway Provision Case.

The Table 3.3.13 sums up all the major materials required to construct one span of superstructure of extradosed bridge.

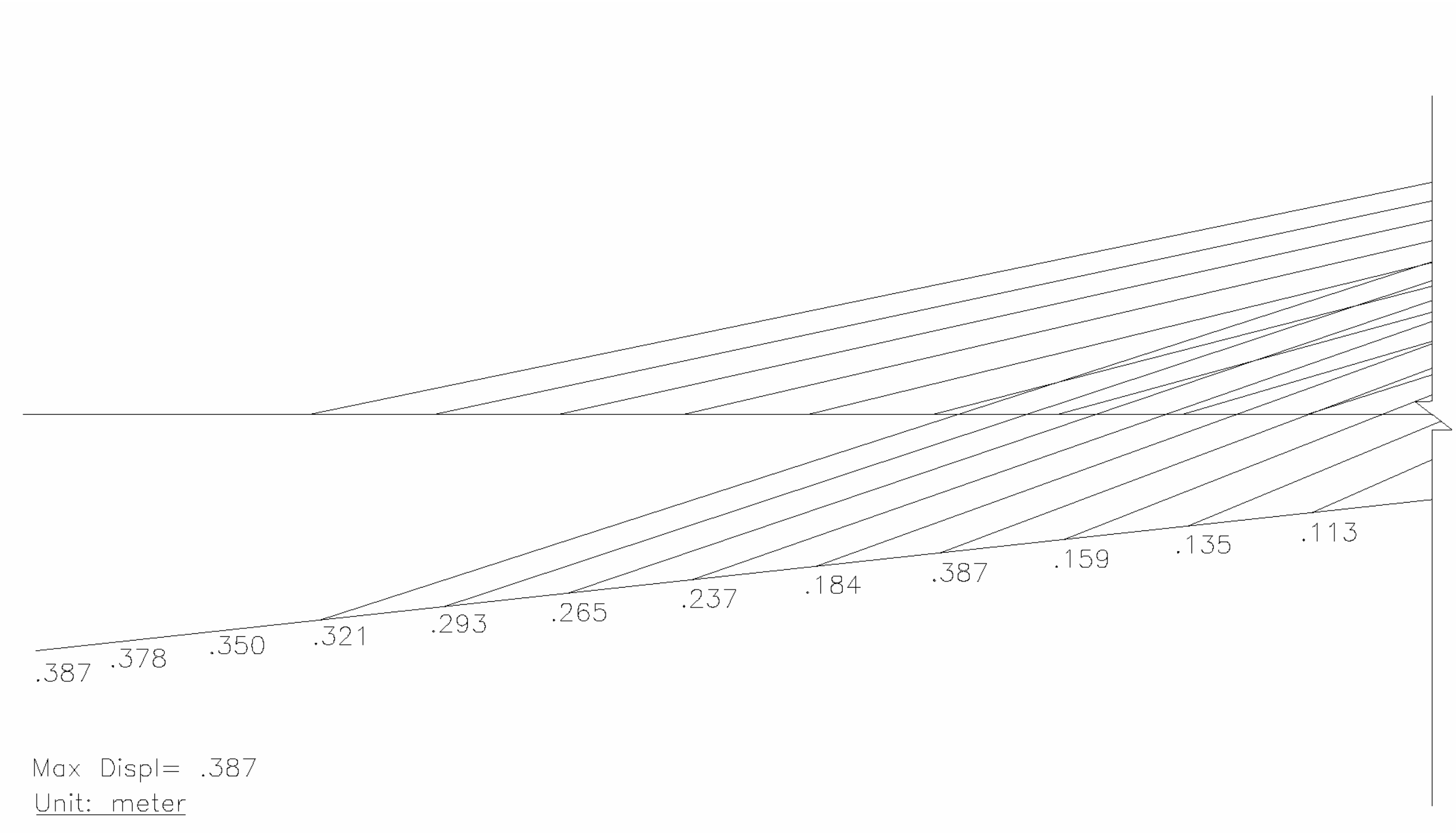


Figure 3.3.21 Deflection Diagram

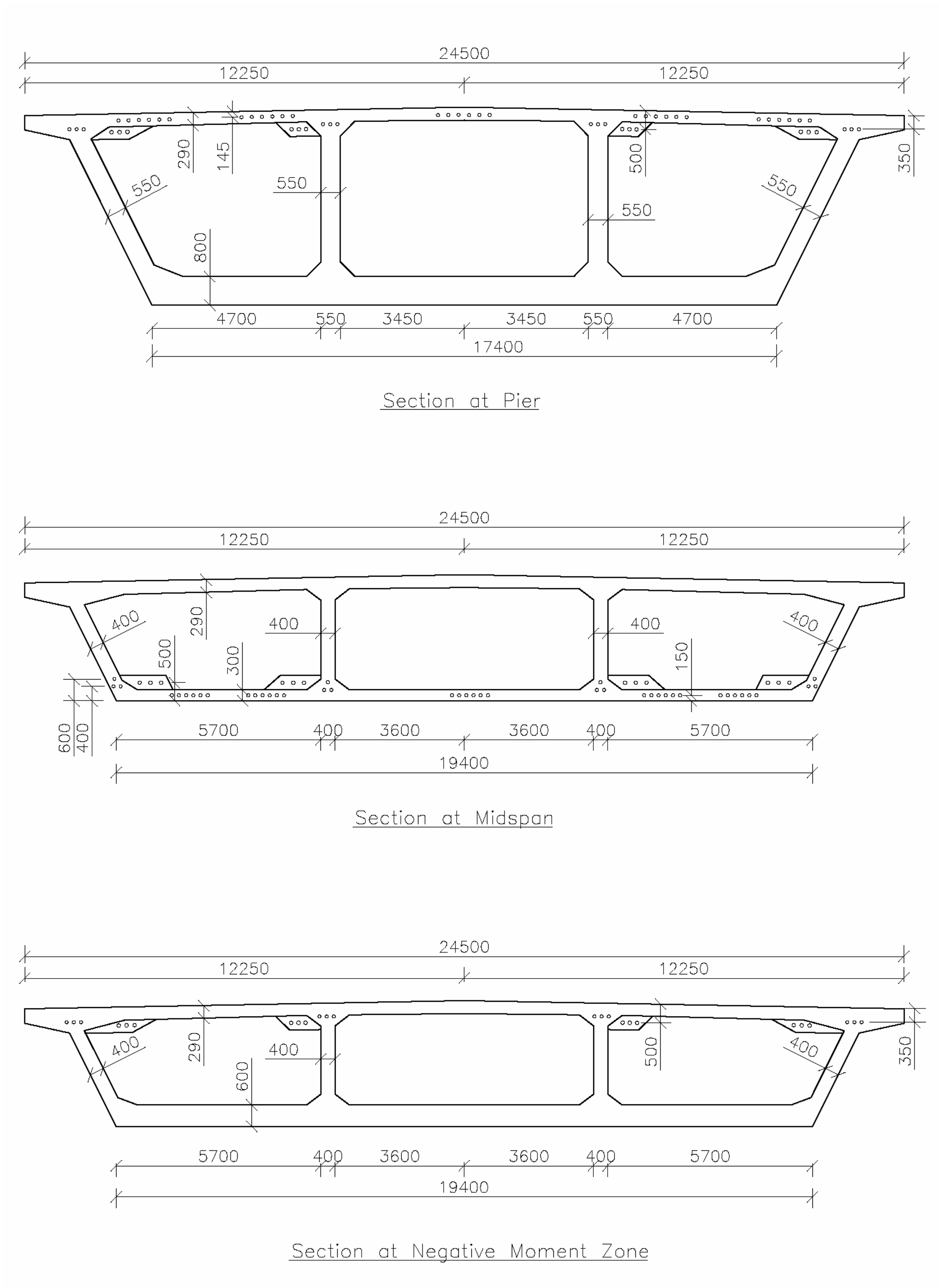
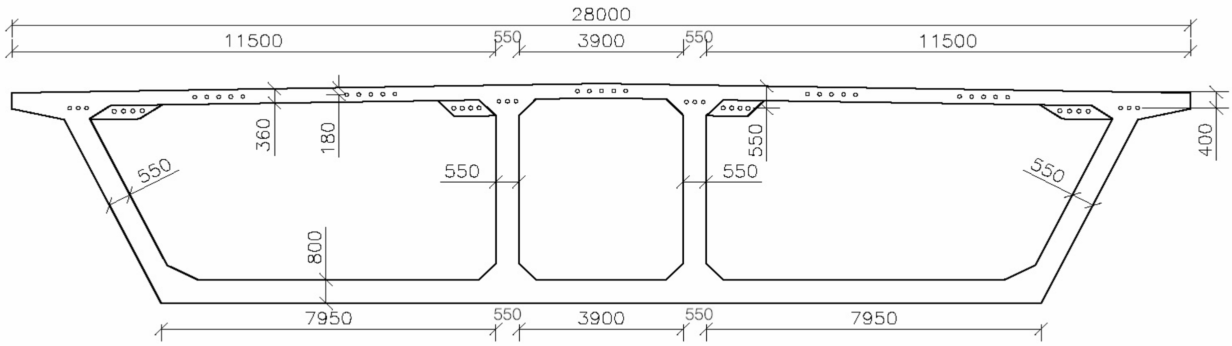
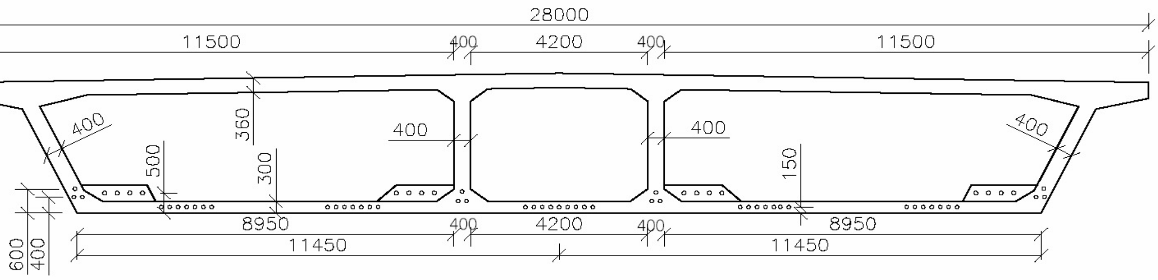


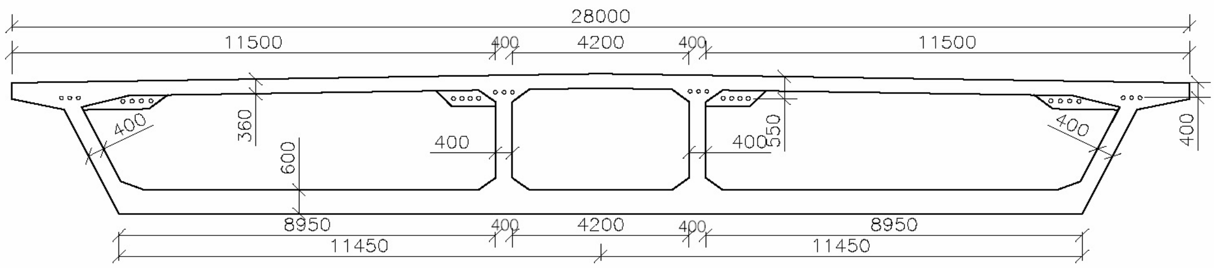
Figure 3.3.22 Prestress Cable Arrangement (Base Case)



Section at Pier



Section at Midspan



Section at Negative Moment Zone

Figure 3.3.23 Prestress Cable Arrangement (Railway Provision Case)

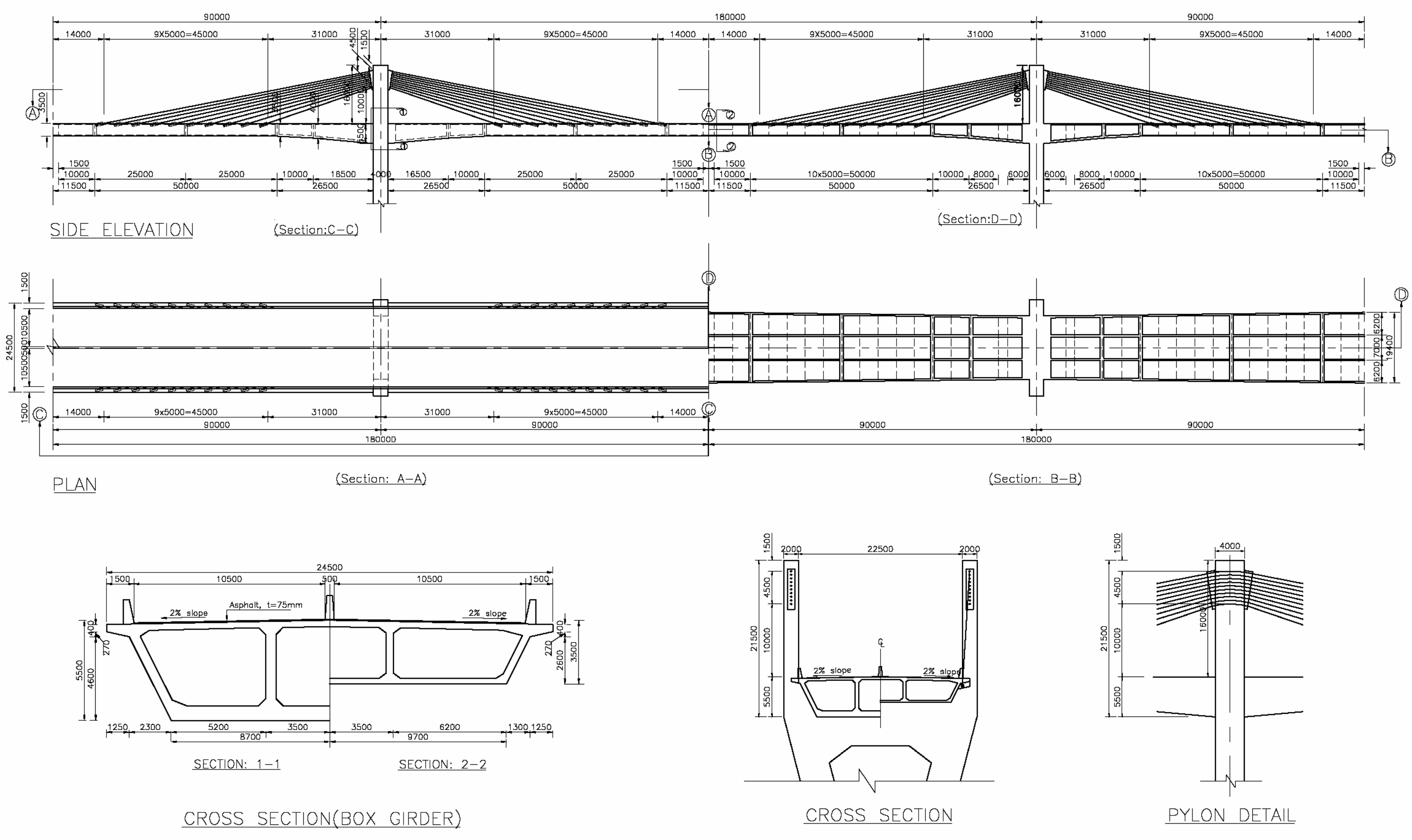


Figure 3.3.24 General View of Extradosed Bridge without Railway

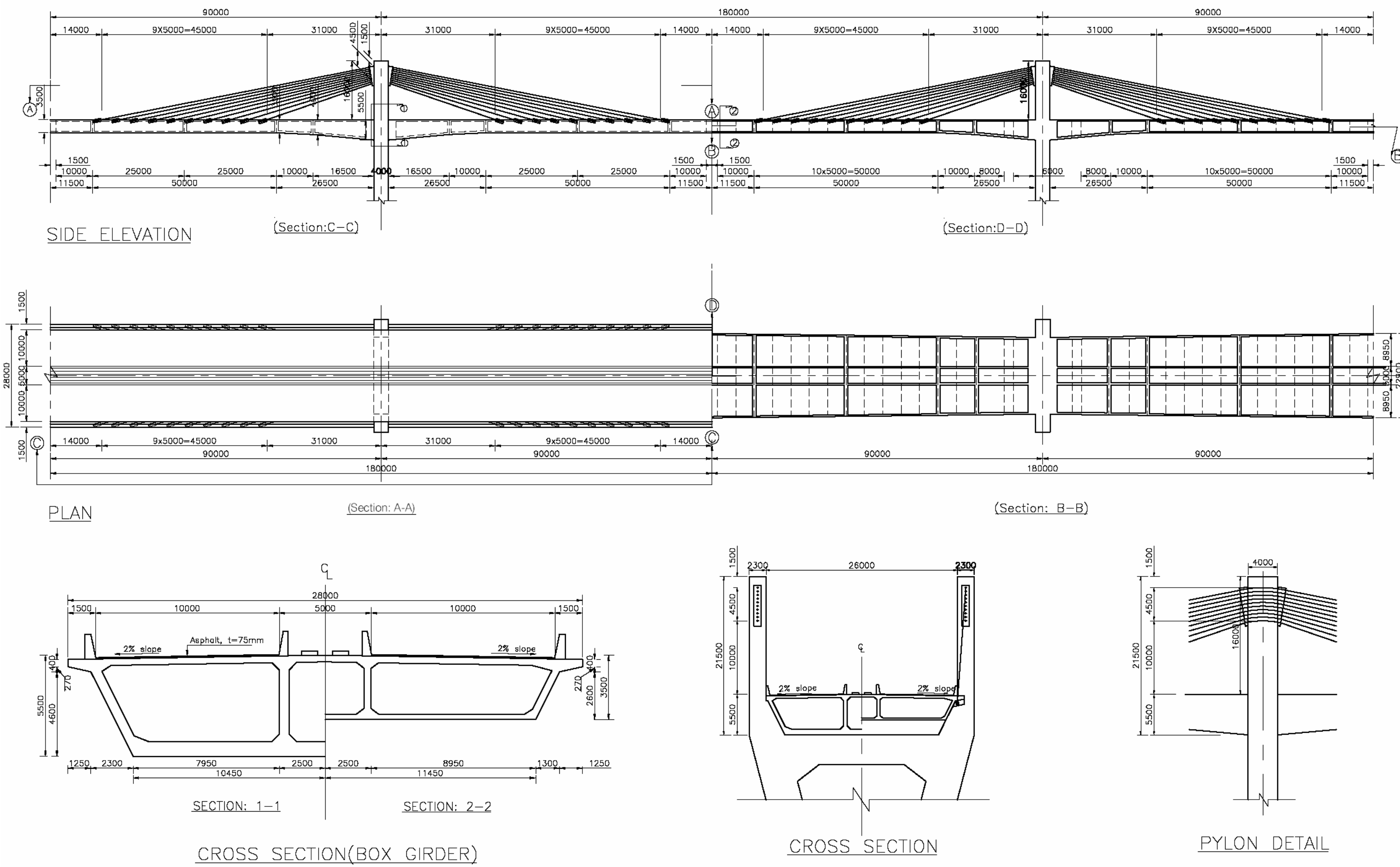
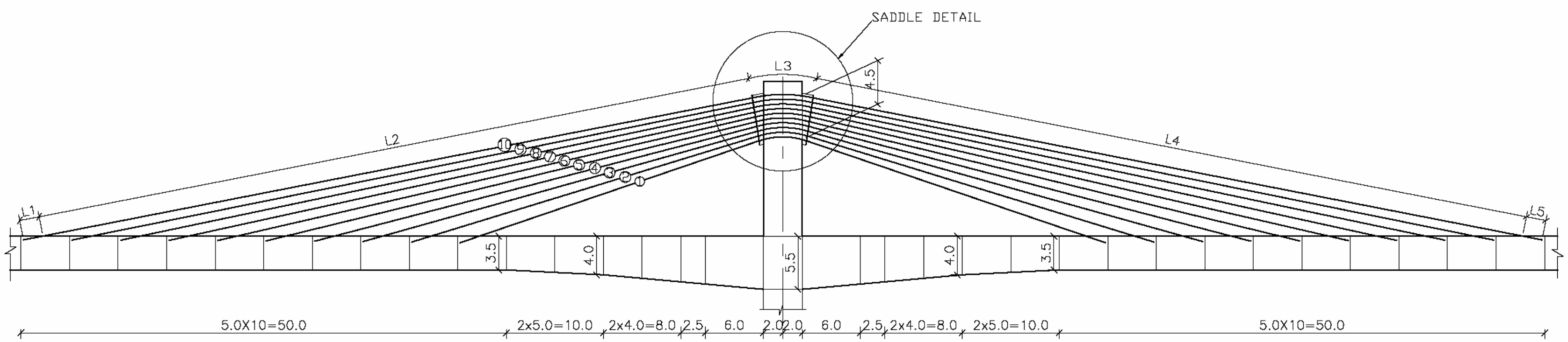
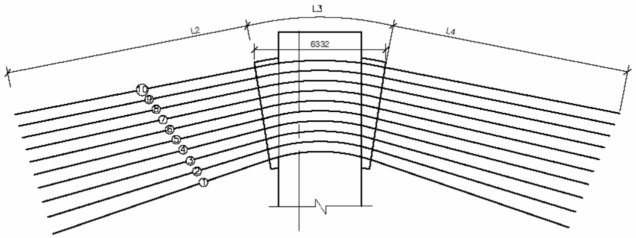


Figure 3.3.25 General View of Extradosed Bridge with Railway



EXTRADOZED CABLE DIMENSIONING



SADDLE DETAIL

TABLE: EXTRADOZED CABLES QUANTITIES

CABLE NO.	L1 m	L2 m	L3 m	L4 m	L5 m	L(L1~L5) m	CABLE TYPE	Weight/m (kg)	Cable Weight(kg)
1	2.114	30.443	4.888	30.443	2.114	70.002	2x27T15.2 SWBR7BL	29.727x2 =59.454	4161.90
2	2.092	35.279	5.040	35.279	2.092	79.782			4743.36
3	2.077	40.143	5.192	40.143	2.077	89.632			5328.98
4	2.066	45.026	5.344	45.026	2.066	99.528			5917.34
5	2.059	49.921	5.495	49.921	2.059	109.455			6507.54
6	2.052	54.826	5.647	54.826	2.052	119.403			7098.99
7	2.048	59.379	5.799	59.379	2.048	128.653			7648.94
8	2.044	64.656	5.951	64.656	2.044	139.351			8284.97
9	2.040	69.579	6.102	69.579	2.040	149.340			8878.86
10	2.038	74.504	6.254	74.504	2.038	159.338			9473.28
					TOTAL=	1144.48			68044.15

Extradozed Cable Weight for one Pier
= 68044.15x2= 136088 kgs

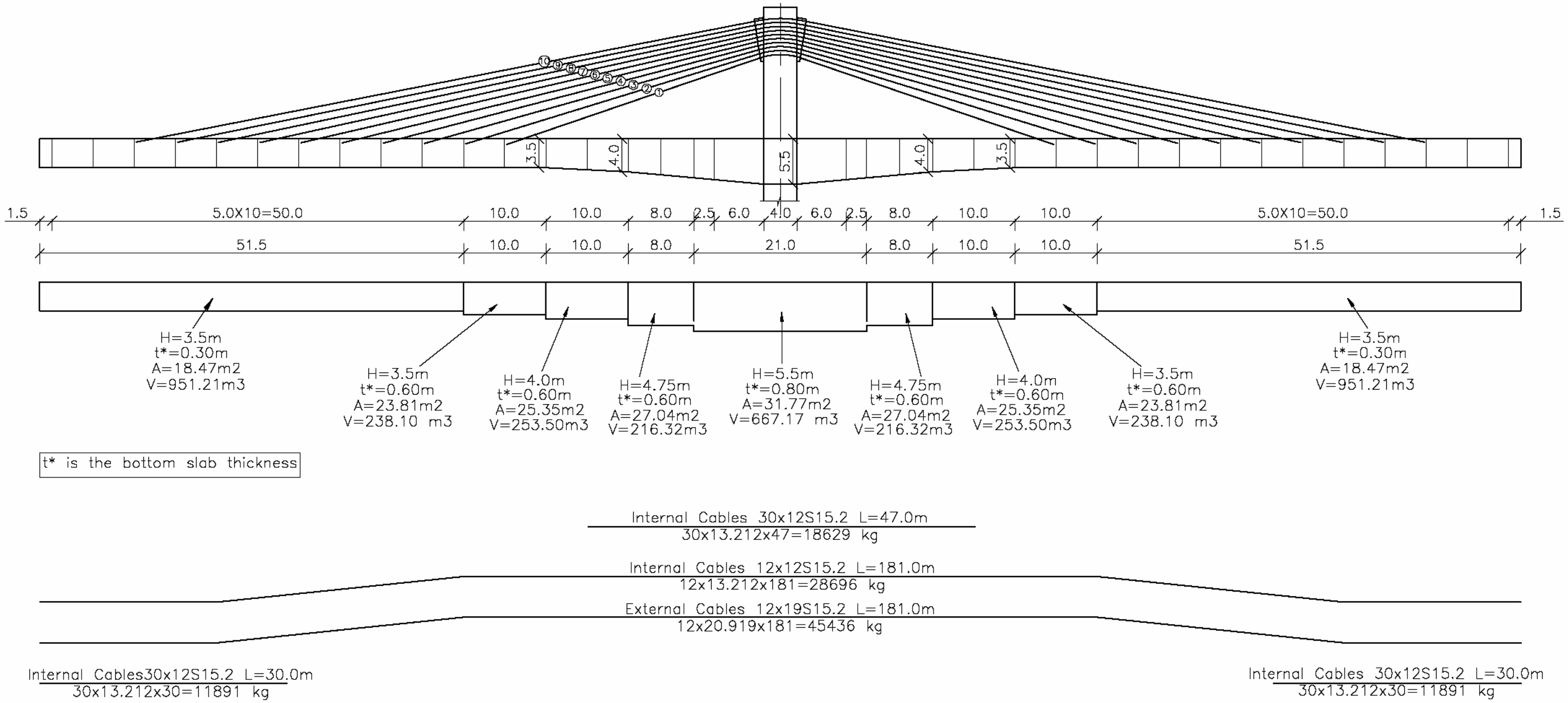
Extradozed Cable Weight for four panel (hinge to hinge)
= 136088x4= 544.35 ton

Extradozed Cable Weight for total bridge
= 544.35x7= 3811 ton

Concrete Volume for one span=3985.43x1.15=4583 m3

Extradozed cable amount for one span= 136088/4583 =29.7 kg/m3

Figure 3.3.26 Quantities of Stay Cables (Base Case)



Total weight of longitudinal Internal and External Prestressing Cables
for one span=18629+28696+45436+2x11891
=116543 kg

Total Concrete Volume= 3985.43 m3

Total Concrete Volume taking into consideration concrete volume
due to diaphragms= 3985.43x1.15= 4583 m3

Total weight of prestressing Cables/m3= 116543/4583= 25.4 kg/m3

Transverse Prestressing cables at Top Slab for one Span=4.22kg/mx24.5mx(180m/0.5)
(1S28.6@0.5m) =37220 kg

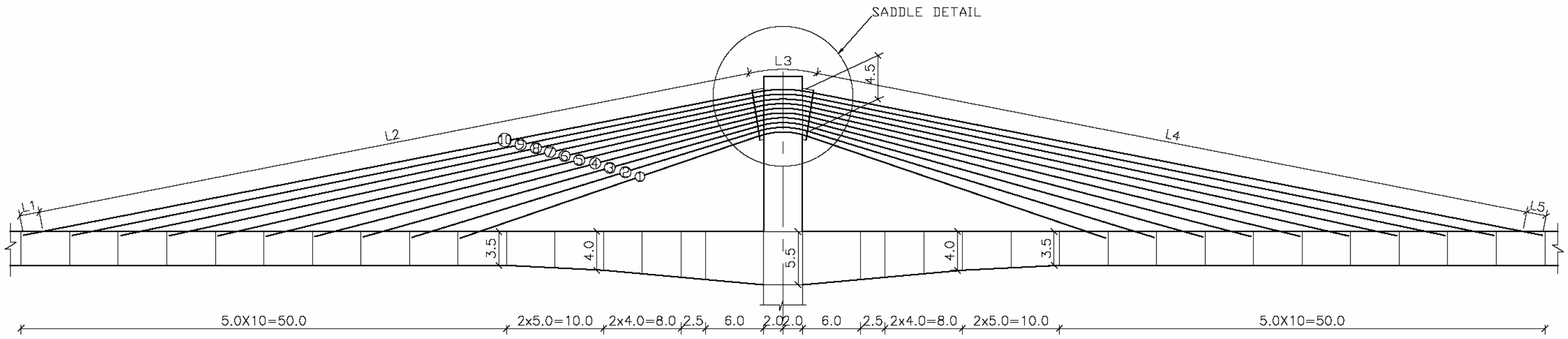
Transverse Prestressing cables at Bottom Slab for one Span=4.22kg/m x=19mx(180m/0.5)
(1S28.6@0.5m) =28865 kg

Total weight of transverse prestressing Cables/m3= (37220+28865)/4583=14.4 kg/m3

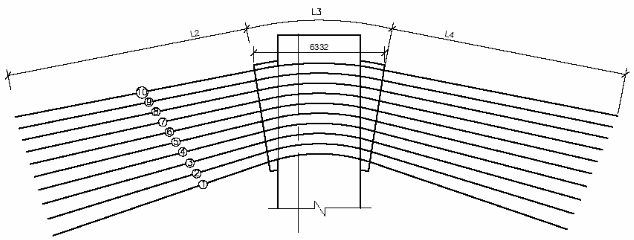
Total weight of prestressing cables for one Span=116543+37220+28865=182628 kg/m≅183 ton

Total weight of prestressing cables/m3=182628/ 4583 =39.8 kg/m3

Figure 3.3.27 Quantities of Deck Girder (Base Case)



EXTRADOSED CABLE DIMENSIONING



SADDLE DETAIL

TABLE: EXTRADOZED CABLES QUANTITIES

CABLE NO.	L1 m	L2 m	L3 m	L4 m	L5 m	L(L1~L5) m	CABLE TYPE	Weight/m (kg)	Cable Weight(kg)
1	2.114	30.443	4.888	30.443	2.114	70.002	2x37T15.2 SWBR7BL	40.737x2 =81.474	5703.34
2	2.092	35.279	5.040	35.279	2.092	79.782			6500.16
3	2.077	40.143	5.192	40.143	2.077	89.632			7302.68
4	2.066	45.026	5.344	45.026	2.066	99.528			8108.94
5	2.059	49.921	5.495	49.921	2.059	109.455			8917.74
6	2.052	54.826	5.647	54.826	2.052	119.403			9728.24
7	2.048	59.379	5.799	59.379	2.048	128.653			10481.87
8	2.044	64.656	5.951	64.656	2.044	139.351			11353.48
9	2.040	69.579	6.102	69.579	2.040	149.340			12167.33
10	2.038	74.504	6.254	74.504	2.038	159.338			12981.90
					TOTAL=	1144.48			93245.69

Extradozed Cable Weight for one Pier
= 93245.69x2= 186491 kgs
Extradozed Cable Weight for four panel (hinge to hinge)
= 186491x4= 745.96 ton
Extradozed Cable Weight for total bridge
= 745.96x7= 5222 ton
Concrete Volume for one span=4751.28x1.15=5464 m3
Extradozed cable amount for one span= 186491/5276 =35.3 kg/m3

Figure 3.3.28 Quantities of Stay Cables (Railway Provision Case)

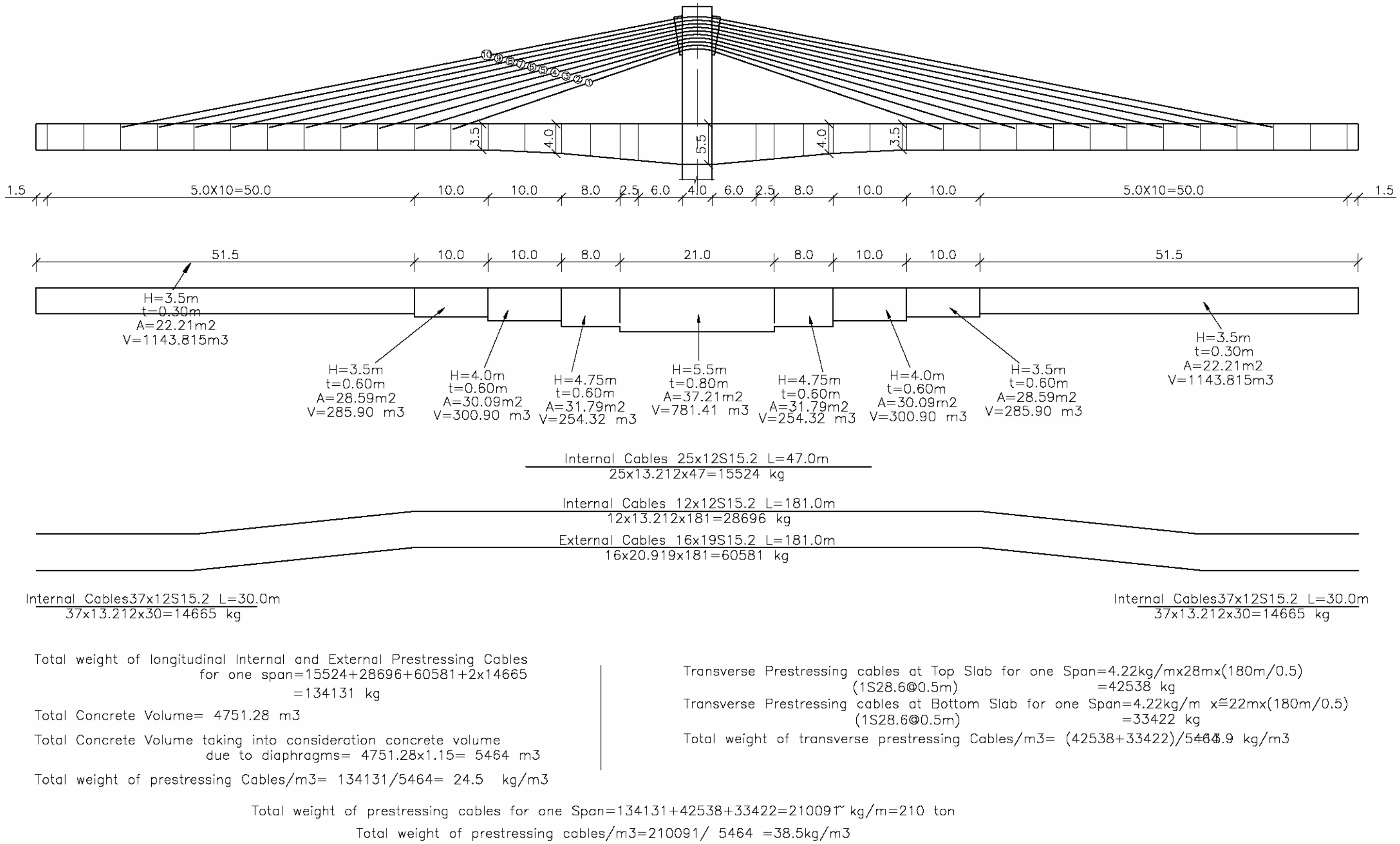


Figure 3.3.29 Quantities of Deck Girder (Railway Provision Case)

Table 3.3.13 Quantities of Major Materials for Extradosed Bridges

Element	Materials	Quality	Quantity	Unit	Base Case (21.5m wide)	Railway Provision Case (25.0m wide)
Deck Girder	Concrete	σck=40N/mm2	Volume	m3	4583	5464
	Longitudinal Interior PC Cables	12S15.2 SWPR7B	Weight	ton	74.7	77.2
			Density	kg/m3	16.3	14.1
			Total Length	m	5382	5567
	Longitudinal Exterior PC Cables	19S15.2 SWPR7B	Weight	ton	47.7	63.6
			Density	kg/m3	10.4	11.7
			Total Length	m	2172	2896
	Transverse PC Cables	1S28.6	Weight	ton	69.4	79.8
			Density	kg/m3	15.2	14.6
			Total Length	m	15660	18000
	Total PC Cables		Weight	ton	191.8	220.6
			Density	kg/m3	41.9	40.4
	Reinforcing Bar	SD345	Weight	ton	733.3	874.2
			Density	kg/m3	160	160
	Formwork		Outer Area	m2	5309	5936
			Inner Area	m2	10447	11616
Pylon	Concrete	σck=40N/mm2	Volume	m3	400	452
	Reinforcing Bar	SD345	Weight	ton	88.0	99.4
			Density	kg/m3	220	220
	Formwork		Area	m2	504	524
Stay Cables	Cables		Type		2x27S15.2	2x37S15.2
			No. of Cables		40	40
			Average Length	m	114.4	114.4
			Weight	ton	156.5	214.5
			Density	kg/m3	34.1	39.3

3.3.5 Preliminary Design of Substructures

(1) Geotechnical Pile Capacity

The geotechnical capacity of the proposed 3150mm diameter tubular steel piles have been determined after consideration and comparison of various studies by Hansen, Terzaghi, Meyerhof , Shioi/Fukui and the American Petroleum Institute Recommended Practice 2A (API RP2A). It was concluded that the recommendations of Hansen together with the limiting base and shaft resistance from API RP2A provided the most appropriate and virtually most conservative solution for the silty sand found at the proposed bridge site. It is noted that use of these recommendations should limit total pile settlement to approximately 25mm.

The calculation of the ultimate capacity of the piles is outlined below based on the geotechnical parameters and correlations derived in Appendix-4:

Table 3.3.14 (1) Design Parameters corrected for Scour Bed Level of -30.770 m PWD

Elev. (m PWD)	Depth z (m)	Scour N-Value	φ (deg)	Es (MN/m^2)
0	0.0	0	0	0
-10	0.0	0	0	0
-20	0.0	0	0	0
-30	0.0	0	0	0
-40	9.2	22	33	6
-50	19.2	33	37	10
-60	29.2	43	37	12
-70	39.2	53	37	15
-80	49.2	63	37	18
-90	59.2	64	37	19
-100	69.2	66	37	19
-110	79.2	67	37	19
-120	89.2	67	37	19

Table 3.3.14 (2) Design Parameters corrected for Scour Bed Level of -44.700 m PWD

Elev. (m PWD)	Depth z (m)	Scour N-Value	φ (deg)	Es (MN/m^2)
0	0.0	0	0	0
-10	0.0	0	0	0
-20	0.0	0	0	0
-30	0.0	0	0	0
-40	0.0	0	0	0
-50	5.3	21	33	6
-60	15.3	34	37	10
-70	25.3	45	37	13
-80	35.3	55	37	16
-90	45.3	58	37	17
-100	55.3	60	37	17
-110	65.3	62	37	18
-120	75.3	63	37	18

Tables 3.3.13 (1), (2) and Figure 3.3.30 show the corrected SPT N-value and associated

Internal Angle of Friction (φ) and Modulus of Elasticity (Em) for the design scoured river bed levels in the mid- river and edge river regimes. These values have been adjusted to take in to account the local scour due to the submerged pile geometry.

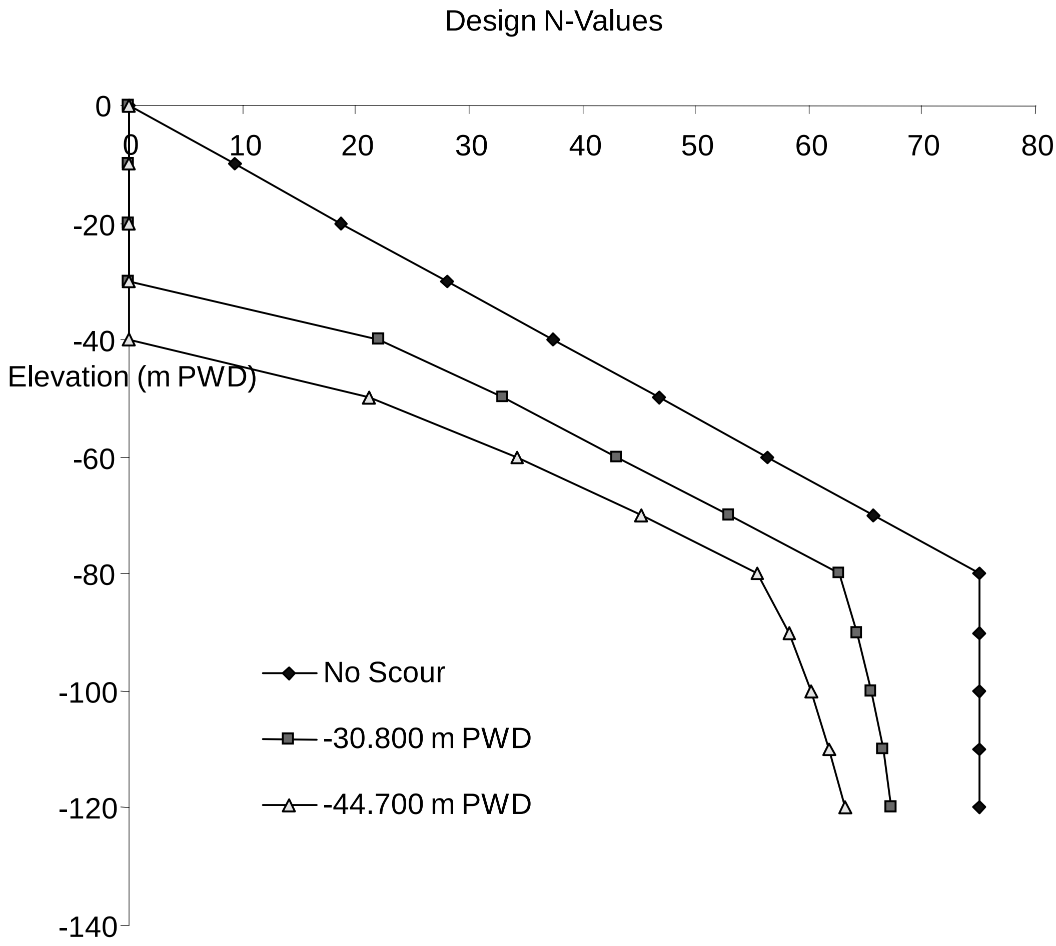


Figure 3.3.30 SPT N-Values corrected for Local Scour

The ultimate capacity of the piles due to skin friction and end bearing components together with their combined working load capacity are shown in Table 3.3.15. These are based on the following resistance limits and applied factors of safety.

Limiting shaft resistance = 180 kN/m^2
Limiting base resistance = 6900 kN/m^2

Factor of Safety on skin friction = 1.5
Factor of Safety on end bearing = 3.0
Factor of Safety on combined resistance = 2.5

Table 3.3.15 Pile Capacity Variation with Toe Level

Toe (m PWD)	Scoured Bed Level = -30.800 m PWD				Scoured Bed Level = -44.700 m PWD			
	$Q_U$base (MN)	$Q_U$side (MN)	Qallow (MN)	FoS	$Q_U$base (MN)	$Q_U$side (MN)	Qallow (MN)	FoS
-40	36	1	13	2.9	-	-	-	-
-50	54	8	23	2.7	20	0	7	2.9
-60	54	21	30	2.5	54	5	21	2.8
-70	54	39	37	2.5	54	16	28	2.5
-80	54	56	44	2.5	54	32	34	2.5
-90	54	74	51	2.5	54	50	42	2.5
-100	54	92	58	2.5	54	68	49	2.5
-110	54	110	65	2.5	54	86	56	2.5
-120	54	128	73	2.5	54	104	63	2.5

It can be seen from Figures 3.3.31 and 3.3.32 that the pile capacity adopted provides a conservative approach for this stage of the design when compared with the previously mentioned studies by Terzaghi, Meyerhof and Shioi/Fukui.

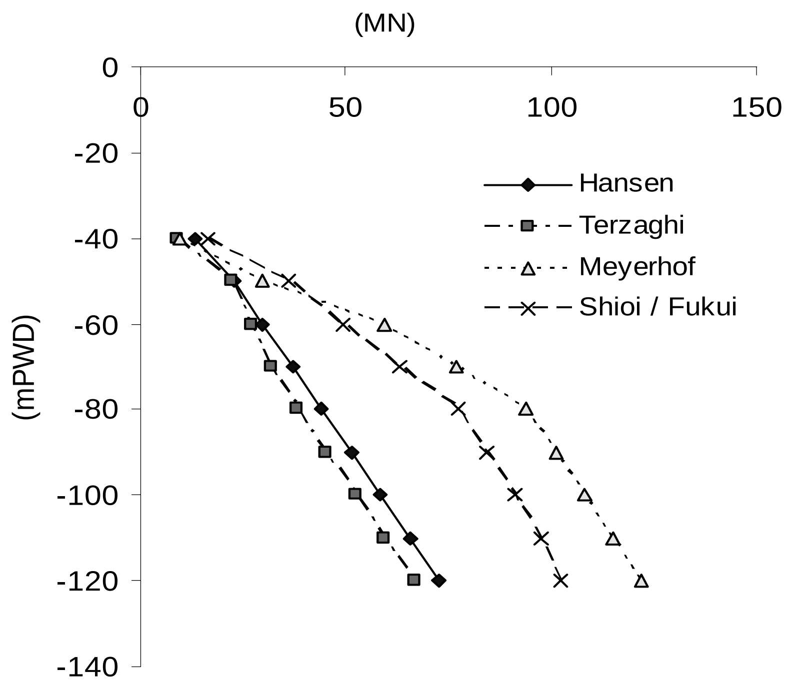


Figure 3.3.31 Comparison of Pile Capacity Studies: -30.800 m PWD Bed Level

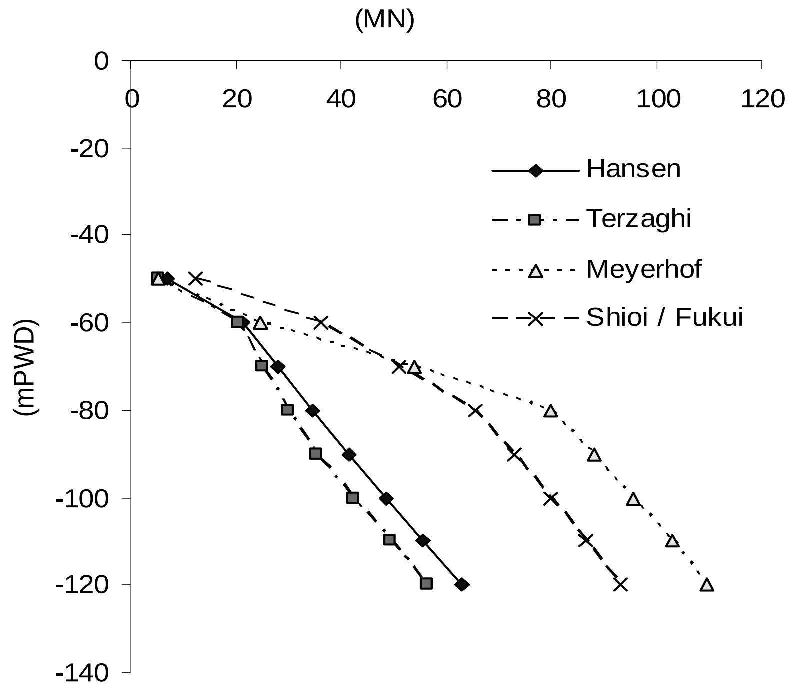


Figure 3.3.32 Comparison of Pile Capacity Studies: -44.700 m PWD Bed Level

The total scour depths referred to above differ slightly to those shown in Appendix 5. This is because the local scour element has been fine tuned based on the actual pile spacing proposed.

It should be noted that the Site Investigation indicated a significantly softer stratum below approximately -98.0m PWD in a number of the boreholes. It is therefore considered prudent at this stage of the design to limit the proposed pile toe level to no deeper than -90.0m PWD.

Given the conservative design approach described above it is concluded that pile group effects will not be critical where a minimum pile spacing of three diameters is maintained at the design ground level.

(2) Pile Design

(a) Pier Groups

For their design, the piers have been categorized into groups that have a similar loading regime. These groups are defined by either the geometry of the bridge structure or that of the river bed.

River Bed Categories:

Deep Channel (DC) – Applies to the edge of river where the combined scour effects result in a design bed level of -44.700m PWD. This only applies to the extradosed girder alternatives.

Shallow Channel – Applies to the middle of the river where the combined scour effects result in a design bed level of -30.770m PWD. This applies to both the extradosed and cable stayed girder alternatives.

Bridge Structure Categories:

Shallow Channel 1 (SC1) – This is a sub category of the above "Shallow Channel" and applies to that centre section of the structure where the superstructure is at its maximum elevation. This applies to both the extradosed and cable stayed girder alternatives.

Shallow Channel 2 (SC2) – This is the other a sub category of the "Shallow Channel" and applies to the end sections of the structure where the superstructure is at its minimum elevation. This applies only to the extradosed girder alternatives.

Outer Pier (OP) – This is a sub category of "Deep Channel" and a further sub category of "Shallow Channel 1" and "Shallow Channel 2" above. It refers to the two outermost piers within each 720m long bridge structure defined by the superstructure expansion joint. This applies only to the extradosed girder alternatives and is denoted by "P1" in superstructure preliminary design section of this study.

Inner Pier (IP) – This is also a sub category of "Deep Channel", "Shallow Channel 1" and "Shallow Channel 2" above. It refers to the two innermost piers within each 720m long bridge structure defined by the superstructure expansion joint. This applies only to the extradosed girder alternatives and is denoted by "P2" in superstructure preliminary design section of this study.

Pylon Pier (PP) – This is a further sub category of "Shallow Channel 1". It refers to the cable stay pylon piers that flank the 360m cable stayed span.

Back-span Pier (BP) – This is also a sub category of "Shallow Channel 1" and refers to the piers that support the 180m back spans to the cable stayed section of the superstructure.

The above categories of pier and their applicability to each of the bridge alternatives are shown in Table 3.3.16.

It can be seen that the extradosed bridge elements of alternative-H2 and alternative-HR2 are identical to those elements in alternative-H1 and alternative-HR respectively, and therefore no additional design was carried out for the extradosed elements for these alternatives.

Table 3.3.16 Pier Types Applicable to Bridge Alternatives

Bridge	River Channel		
Alternative	DC	SC1	(SC2)
H1	OP / IP	OP / IP	OP / IP
H2	OP / IP	OP / IP / PP / BP	OP / IP
HR	OP / IP	OP / IP	OP / IP
HR2	OP / IP	OP / IP / PP / BP	OP / IP
H3	OP / IP	OP / IP	OP / IP

(b) Pier and Pile Loads

The piles have been designed for the load factors and combinations shown in Table 3.3.17. Nominal load factor combinations and Ultimate Limit State load factor combinations have been used to determine the geotechnical and structural capacities of the piles respectively. The load type abbreviations used in the table are further explained below.

Dead Load (DL) – Structural self weight.
Superimposed Dead Load (SDL1) – Surfacing self weight
Superimposed Dead Load (SDL2) – Self weight of all other non structural permanent components
Shrinkage and Creep (S&C) – Load due to restraint against shrinkage and creep of the concrete structure.
Traffic Load (HL) – Load from highway vehicles and railway
Abnormal traffic Load (AHL) – Load from abnormal vehicle load if specified
Footway Load (F) – Not applied to the Padma Bridge
Wind Load (W) – Load due to wind
Temperature (T) – load due to temperature difference and restraint against temperature effects (except friction)
Temperature Frictional Restraint (TF) – Not applicable to extradosed and cable stayed girders for Padma Bridge.
Differential Settlement (DS) – Load due to differential settlement of supports.
Earth Pressure (E) – Horizontal and vertical load due to retained soil
River Stream Flow (SF) – Drag load on submerged structure due to flow in river during maximum scour
Buoyancy (B) – Hydrostatic loads on submerged structure
Wave Load (WL) – Drag and inertia load due to wave impact on the structure
Boat Impact (BI) – Ship collision loading
Earthquake Load (EQ) – Seismic loading including liquefaction effect where applicable
Construction (C) – Temporary construction loads

A number of these minor effects have been simplified during the analysis for expedience at this stage of the design.

Table 3.3.17 Design Load Combinations

Load Combination	Limit State	Load or Environmental Effect																		
		DL		SDL 1	SDL 2	S&C	HL	AHL	F	W	T	TF	DS	E	SF	B	WL	BI	EQ	C
		Steel	Conc																	
1	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00	1.40 1.00		1.50 1.00				1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00				
2	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00		1.30 1.10	1.50 1.00				1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00				
3	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00	1.20 1.00		1.50 1.00	1.10 1.00			1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00	1.30 1.00			
4	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00		1.10 1.00	1.50 1.00	1.10 1.00			1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00	1.30 1.00			
5	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00			1.50 1.00	1.40 1.00			1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00	1.30 1.00			
6	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00			1.50 1.00			1.30 1.00	1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00				
7	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00	1.20 1.00		1.50 1.00		1.30 1.00		1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00				
8 * * *	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00	1.25 1.00	1.25 1.00	1.50 1.00				1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00	1.30 1.00			
9	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00							1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00				
10	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00							1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00			1.00 1.00	
11	ULS NOM	1.05 1.00	1.15 1.00	1.30 1.00	1.20 1.00	1.20 1.00							1.20 1.00	1.50 1.00	1.30 1.00	1.20 1.00		1.00 1.00		
12	ULS NOM			1.30 1.00	1.20 1.00	1.20 1.00				1.10 1.00	1.30 1.10			1.50 1.00	1.30 1.10	1.20 1.00	1.30 1.00			1.10 1.00

* * * The loads to be considered are the permanent loads and the secondary live loads together with the appropriate primary live loads associated with them.

(c) Nominal Pile Load Effects

In order to determine the optimum number of piles required, the design has investigated the effect of providing an 8, 10 or 12 pile bent to each extradosed girder pile group. A similar spread of piles has also been considered for the cable stayed alternatives... The resulting maximum and minimum axial pile loads, together with their critical combination, are shown in Tables 3.3.18 to 3.3.22. The loads exclude pile self weight.

The pile groups have been modeled assuming a depth to pile fixity of approximately three diameters below the design river bed levels. It can be seen from the tabulated pile toe levels that the ratio of the pile embedment length to pile diameter is approximately 16. A value greater than 12 suggests that the piles will behave as long slender piles, which will fail by fracture of the pile at the point of maximum bending moment, rather than short rigid piles which would fail as the passive resistance of the soil is exceeded. This confirms that it is appropriate to model the lateral soil structure stiffness based on the pile stiffness alone at this stage of the design. The vertical soil structure stiffness has been represented by the 25mm settlement under capacity load as discussed earlier in this section.

At this stage of the design the live load horizontal deflections have been limited to approximately 25mm to 38mm as guided by AASHTO Standard Specification for Highway Bridges. However, this should be investigated further during the detailed design stage particularly for out of phase seismic movements.

It can be seen from these tables that the vertical load combination 3 dominates for the greatest mass structures such as the cable stay pylon piers whilst the intermediate and least mass structures are dominated by the seismic and ship collision load combinations 10 and 11 respectively. It should be noted however that there is generally less than 10% variation in pile loads due to combination 3, 10 and 11 for all the extradosed alternatives.

The proposed pile group arrangement is highlighted in bold. Although the tables show the deep channel pile toe levels to be marginally deeper that the -90.0m PWD limit for Alternative-HR it is anticipated that these pile lengths may be reduced due to their stiffness interaction with the shallow channel piers within the same four span bridge structure.

(d) Ultimate Limit State Pile Load Effects

A similar spread of pile bents and respective loadings have been considered for the structural capacity of the piles in order to further determine the optimum number of piles and their required section properties.

The factored (Ultimate Limit State) design load effects for each bridge alternative and pile group considered are shown in Tables 3.3.23 to 3.3.27 together with the pile wall thickness required for both hollow and composite steel tube pile construction. The minimum wall thickness for a 3150mm diameter grade FE510 hollow steel pile and composite steel pile is taken as 50mm and 48mm respectively to maintain slenderness limits. A maximum wall thickness of 63mm has been adopted, where possible, in order to avoid a reduction in yield stress of the steel plate. The critical load combinations are generally the same those for the nominal pile load effects above.

As each pier has been modeled independently of adjacent piers, the results for adjacent extradosed girder piers should be averaged to reflect the actual restraint provided by the adjoining superstructure. As it is not practical to provide a protective coating to the piles a sacrificial steel thickness of 6mm should be added to the pile wall thickness. This is to allow for the expected loss of section due to corrosion in the submerged environment during the design life of the structure.

During the detailed design stage additional capacity may have to be provided to the piles in order to impose plastic hinging on the columns above. This ensures that the weakest point is above the river water level and will therefore facilitate inspection and repair after an extreme seismic event.

The proposed pile group arrangement is highlighted in bold.

Table 3.3.18 Nominal Pile Loads Alternative-H1

			No. of Piles		
			12	**10**	**8**
Shallow Channel 1	**Outer Pier:**				
	Max	Axial Load (kN)	33000	39500	**45900**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Min	Axial Load (kN)	11700	13900	**20100**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Pile cap live load displacement (mm)		6	9	**11**
	Pile toe elevation required (m PWD)		-64.5	-74	**-83**
	Inner Pier:				
	Max	Axial Load (kN)	33100	39600	**46100**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Min	Axial Load (kN)	11800	14000	**20600**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Pile cap live load displacement (mm)		5	8	**9**
	Pile toe elevation required (m PWD)		-64.5	-74	**-83**
Shallow Channel 2	**Outer Pier:**				
	Max	Axial Load (kN)	33000	39500	**46200**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Min	Axial Load (kN)	11800	13900	**21000**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Pile cap live load displacement (mm)		5	7	**9**
	Pile toe elevation required (m PWD)		-64.5	-74	**-83.5**
	Inner Pier:				
	Max	Axial Load (kN)	33000	39600	**45700**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Min	Axial Load (kN)	11800	14000	**21700**
		Critical Combination	10 & 11	10 & 11	**10 & 11**
	Pile cap live load displacement (mm)		4	6	**8**
	Pile toe elevation required (m PWD)		-64.5	-74	**-82.5**
Deep Channel	**Outer Pier:**				
	Max	Axial Load (kN)	**40000**	46700	54100
		Critical Combination	**10 & 11**	10 & 11	10 & 11
	Min	Axial Load (kN)	**7300**	9800	16900
		Critical Combination	**10**	10	10
	Pile cap live load displacement (mm)		**11**	13	16
	Pile toe elevation required (m PWD)		**-87.5**	-97	-107.5
	Inner Pier:				
	Max	Axial Load (kN)	**40200**	46800	54500
		Critical Combination	**10 & 11**	10 & 11	10 & 11
	Min	Axial Load (kN)	**7200**	9900	16600
		Critical Combination	**10**	10	10
	Pile cap live load displacement (mm)		**8**	11	13
	Pile toe elevation required (m PWD)		**-88**	-97	-108

Table 3.3.19 Nominal Pile Loads Alternative-H2

				No. of Piles				
				24	**16**	**12**	**10**	**8**
Shallow Channel 1	**Pylon Pier:**							
		Max	Axial Load (kN)	**43600**	66800	-	-	-
			Critical Combination	**3**	3	-	-	-
		Min	Axial Load (kN)	**10400**	25800	-	-	-
			Critical Combination	**3**	3	-	-	-
	Pile cap live load displacement (mm)			**12**	16	-	-	-
	Pile toe elevation required (m PWD)			**-79**	-112.5	-	-	-
	Back Span Pier:							
		Max	Axial Load (kN)	-	-	-	23900	**26000**
			Critical Combination	-	-	-	11	**11**
		Min	Axial Load (kN)	-	-	-	-1600	**2000**
			Critical Combination	-	-	-	11	**11**
	Pile cap live load displacement (mm)			-	-	-	0	**0**
	Pile toe elevation required (m PWD)			-	-	-	-51.5	**-55**
SC1	**Outer Pier:**			-	-	As Alternative-H1		
	Inner Pier:			-	-			
SC2	**Outer Pier:**			-	-	As Alternative-H1		
	Inner Pier:			-	-			
DC	**Outer Pier:**			-	-	As Alternative-H1		
	Inner Pier:			-	-			

Table 3.3.20 Nominal Pile Loads Alternative-HR

			No. of Piles		
			12	**10**	**8**
Shallow Channel 1	**Outer Pier:**				
	Max	Axial Load (kN)	36800	**44000**	52900
		Critical Combination	3 & 10	**3 & 10**	3 & 10
	Min	Axial Load (kN)	13000	**15700**	21800
		Critical Combination	10	**10**	10
	Pile cap live load displacement (mm)		25	**36**	44
	Pile toe elevation required (m PWD)		-70	**-80**	-93
	Inner Pier:				
	Max	Axial Load (kN)	36900	**44200**	53100
		Critical Combination	3 & 10	**3 & 10**	3 & 10
	Min	Axial Load (kN)	13200	**15900**	22500
		Critical Combination	10	**10**	10
	Pile cap live load displacement (mm)		21	**29**	35
	Pile toe elevation required (m PWD)		-70	**-80.5**	-93
Shallow Channel 2	**Outer Pier:**				
	Max	Axial Load (kN)	36600	**43800**	52300
		Critical Combination	3 & 10	**3 & 10**	3 & 10
	Min	Axial Load (kN)	13300	**15700**	32800
		Critical Combination	10	**10**	10
	Pile cap live load displacement (mm)		20	**29**	37
	Pile toe elevation required (m PWD)		-69.5	**-80**	-92
	Inner Pier:				
	Max	Axial Load (kN)	36700	**44000**	52900
		Critical Combination	3 & 10	**3 & 10**	3 & 10
	Min	Axial Load (kN)	13400	**15900**	24100
		Critical Combination	10	**10**	10
	Pile cap live load displacement (mm)		16	**23**	29
	Pile toe elevation required (m PWD)		-70	**-80**	-93
Deep Channel	**Outer Pier:**				
	Max	Axial Load (kN)	**44200**	51400	59600
		Critical Combination	**10**	10	3 & 10
	Min	Axial Load (kN)	**8000**	11100	18900
		Critical Combination	**10**	10	10
	Pile cap live load displacement (mm)		**39**	52	65
	Pile toe elevation required (m PWD)		**-93**	-103.5	-115.5
	Inner Pier:				
	Max	Axial Load (kN)	**44400**	51500	60700
		Critical Combination	**10**	10	3 & 10
	Min	Axial Load (kN)	**8100**	11200	18800
		Critical Combination	**10**	10	10
	Pile cap live load displacement (mm)		**31**	42	52
	Pile toe elevation required (m PWD)		**-93.5**	-104	-117

Table 3.3.21 Nominal Pile Loads Alternative-HR2

			No. of Piles				
			24	**16**	**12**	**10**	**8**
Shallow Channel 1	**Pylon Pier:**						
	Max	Axial Load (kN)	**49600**	73700	-	-	-
		Critical Combination	**3**	3	-	-	-
	Min	Axial Load (kN)	**10600**	22800	-	-	-
		Critical Combination	**3**	3	-	-	-
	Pile cap live load displacement (mm)		**46**	59	-	-	-
	Pile toe elevation required (m PWD)		**-88**	<-120	-	-	-
	Back Span Pier:						
	Max	Axial Load (kN)	-	-	20100	24200	**26300**
		Critical Combination	-	-	11	11	**11**
	Min	Axial Load (kN)	-	-	-1100	-1400	**2300**
		Critical Combination	-	-	11	11	**11**
	Pile cap live load displacement (mm)		-	-	0	0	**0**
	Pile toe elevation required (m PWD)		-	-	-47.5	-52	**-55**
SC1	**Outer Pier:**		-	-	As Alternative-HR		
	Inner Pier:		-	-			
SC2	**Outer Pier:**		-	-	As Alternative-HR		
	Inner Pier:		-	-			
DC	**Outer Pier:**		-	-	As Alternative-HR		
	Inner Pier:		-	-			

Table 3.3.22 Nominal Pile Loads Alternative-H3

			No. of Piles		
			12	**10**	**8**
Shallow Channel 1	**Outer Pier:**				
Shallow Channel 1	Max	Axial Load (kN)	-	36200	**41500**
Shallow Channel 1	Max	Critical Combination	-	11	**11**
Shallow Channel 1	Min	Axial Load (kN)	-	10700	**17500**
Shallow Channel 1	Min	Critical Combination	-	11	**11**
Shallow Channel 1	Pile cap live load displacement (mm)		-	9	**11**
Shallow Channel 1	Pile toe elevation required (m PWD)		-	-69	**-76.5**
Shallow Channel 1	**Inner Pier:**			11	**11**
Shallow Channel 1	Max	Axial Load (kN)	-	36200	**41500**
Shallow Channel 1	Max	Critical Combination	-	11	**11**
Shallow Channel 1	Min	Axial Load (kN)	-	10700	**17500**
Shallow Channel 1	Min	Critical Combination	-	11	**11**
Shallow Channel 1	Pile cap live load displacement (mm)		-	8	**9**
Shallow Channel 1	Pile toe elevation required (m PWD)		-	-69	**-76.5**
Shallow Channel 2	**Outer Pier:**				
Shallow Channel 2	Max	Axial Load (kN)	-	36200	**42000**
Shallow Channel 2	Max	Critical Combination	-	11	**11**
Shallow Channel 2	Min	Axial Load (kN)	-	10700	**17100**
Shallow Channel 2	Min	Critical Combination	-	11	**11**
Shallow Channel 2	Pile cap live load displacement (mm)		-	7	**9**
Shallow Channel 2	Pile toe elevation required (m PWD)		-	-69	**-77.5**
Shallow Channel 2	**Inner Pier:**				
Shallow Channel 2	Max	Axial Load (kN)	-	36200	**41500**
Shallow Channel 2	Max	Critical Combination	-	11	**11**
Shallow Channel 2	Min	Axial Load (kN)	-	10700	**17500**
Shallow Channel 2	Min	Critical Combination	-	11	**11**
Shallow Channel 2	Pile cap live load displacement (mm)		-	6	**8**
Shallow Channel 2	Pile toe elevation required (m PWD)		-	-69	**-76.5**
Deep Channel	**Outer Pier:**				
Deep Channel	Max	Axial Load (kN)	34700	**40200**	46600
Deep Channel	Max	Critical Combination	11	**11**	11
Deep Channel	Min	Axial Load (kN)	4400	**6700**	12300
Deep Channel	Min	Critical Combination	11	**11**	11
Deep Channel	Pile cap live load displacement (mm)		10	**13**	16
Deep Channel	Pile toe elevation required (m PWD)		-81	**-87.5**	-97
Deep Channel	**Inner Pier:**				
Deep Channel	Max	Axial Load (kN)	34700	**40200**	46500
Deep Channel	Max	Critical Combination	11	**11**	11
Deep Channel	Min	Axial Load (kN)	4400	**6700**	12600
Deep Channel	Min	Critical Combination	11	**11**	11
Deep Channel	Pile cap live load displacement (mm)		8	**11**	13
Deep Channel	Pile toe elevation required (m PWD)		-81	**-87.5**	-96.5

Table 3.3.23 Ultimate Limit State Pile Loads and Properties Alternative-H1

		No. of Piles		
		12	10	8
Shallow Channel 1	**Outer Pier:**			
	Maximum Axial Load (kN)	39200	47200	**57000**
	Maximum Bending Moment (kNm)	60700	75700	**97800**
	Maximum Shear Force (kN)	2200	2700	**3300**
	Hollow Tube Wall Thickness (mm)	50	50	**63**
	Composite Tube Wall Thickness (mm)	48	48	48
	Inner Pier:			
	Maximum Axial Load (kN)	39300	47300	**56500**
	Maximum Bending Moment (kNm)	40700	48500	**64900**
	Maximum Shear Force (kN)	1500	1700	**2200**
	Hollow Tube Wall Thickness (mm)	50	50	**50**
	Composite Tube Wall Thickness (mm)	48	48	48
Shallow Channel 2	**Outer Pier:**			
	Maximum Axial Load (kN)	39700	48300	**58400**
	Maximum Bending Moment (kNm)	63100	80800	**106500**
	Maximum Shear Force (kN)	2300	2800	**3400**
	Hollow Tube Wall Thickness (mm)	50	50	**65**
	Composite Tube Wall Thickness (mm)	48	48	48
	Inner Pier:			
	Maximum Axial Load (kN)	39800	48400	**58200**
	Maximum Bending Moment (kNm)	39900	47300	**65600**
	Maximum Shear Force (kN)	1400	1600	**2000**
	Hollow Tube Wall Thickness (mm)	50	50	**50**
	Composite Tube Wall Thickness (mm)	48	48	48
Deep Channel	**Outer Pier:**			
	Maximum Axial Load (kN)	**53100**	61500	77500
	Maximum Bending Moment (kNm)	**80300**	107000	149100
	Maximum Shear Force (kN)	**2100**	2700	3200
	Hollow Tube Wall Thickness (mm)	**51**	63	83
	Composite Tube Wall Thickness (mm)	48	48	48
	Inner Pier:			
	Maximum Axial Load (kN)	**53200**	61600	78500
	Maximum Bending Moment (kNm)	**58900**	65200	102000
	Maximum Shear Force (kN)	**1500**	1600	2100
	Hollow Tube Wall Thickness (mm)	**50**	52	75
	Composite Tube Wall Thickness (mm)	48	48	48

Table 3.3.24 Ultimate Limit State Pile Loads and Properties Alternative-H2

		No. of Piles				
		24	**16**	**12**	**10**	**8**
Shallow Channel 1	**Pylon Pier:**					
	Maximum Axial Load (kN)	**51400**	78200	-	-	-
	Maximum Bending Moment (kNm)	**89800**	142700	-	-	-
	Maximum Shear Force (kN)	**3500**	5300	-	-	-
	Hollow Tube Wall Thickness (mm)	61	88	-	-	-
	Composite Tube Wall Thickness (mm)	**48**	48	-	-	-
	Back Span Pier:					
	Maximum Axial Load (kN)	-	-	-	26200	**28900**
	Maximum Bending Moment (kNm)	-	-	-	29200	**43300**
	Maximum Shear Force (kN)	-	-	-	1200	**1800**
	Hollow Tube Wall Thickness (mm)	-	-	-	50	**50**
	Composite Tube Wall Thickness (mm)	-	-	-	48	48
SC1	**Outer Pier:**	-	-	As Alternative-H1		
	Inner Pier:	-	-			
SC2	**Outer Pier:**	-	-	As Alternative-H1		
	Inner Pier:	-	-			
DC	**Outer Pier:**	-	-	As Alternative-H1		
	Inner Pier:	-	-			

Table 3.3.25 Ultimate Limit State Pile Loads and Properties Alternative-HR

		No. of Piles		
		12	**10**	**8**
Shallow Channel 1	**Outer Pier:**			
	Maximum Axial Load (kN)	44700	**53900**	66100
	Maximum Bending Moment (kNm)	80600	**99900**	131000
	Maximum Shear Force (kN)	2900	**3500**	4400
	Hollow Tube Wall Thickness (mm)	50	**62**	86
	Composite Tube Wall Thickness (mm)	48	48	48
	Inner Pier:			
	Maximum Axial Load (kN)	44900	**54100**	66100
	Maximum Bending Moment (kNm)	46500	**55700**	75500
	Maximum Shear Force (kN)	1700	**1900**	2500
	Hollow Tube Wall Thickness (mm)	50	**50**	56
	Composite Tube Wall Thickness (mm)	48	48	48
Shallow Channel 2	**Outer Pier:**			
	Maximum Axial Load (kN)	44900	**54400**	66200
	Maximum Bending Moment (kNm)	82500	**104900**	132200
	Maximum Shear Force (kN)	2900	**3600**	4400
	Hollow Tube Wall Thickness (mm)	50	**62**	80
	Composite Tube Wall Thickness (mm)	48	48	48
	Inner Pier:			
	Maximum Axial Load (kN)	45000	**54600**	66800
	Maximum Bending Moment (kNm)	48700	**55500**	75900
	Maximum Shear Force (kN)	1700	**1900**	2400
	Hollow Tube Wall Thickness (mm)	50	**50**	58
	Composite Tube Wall Thickness (mm)	48	48	48
Deep Channel	**Outer Pier:**			
	Maximum Axial Load (kN)	**61800**	71400	95700
	Maximum Bending Moment (kNm)	**110100**	152600	219400
	Maximum Shear Force (kN)	**2700**	3500	4100
	Hollow Tube Wall Thickness (mm)	**63**	88	125
	Composite Tube Wall Thickness (mm)	48	48	54
	Inner Pier:			
	Maximum Axial Load (kN)	**62000**	71600	97600
	Maximum Bending Moment (kNm)	**72800**	80800	137700
	Maximum Shear Force (kN)	**1800**	1800	2300
	Hollow Tube Wall Thickness (mm)	**57**	60	93
	Composite Tube Wall Thickness (mm)	48	48	48

Table 3.3.26 Ultimate Limit State Pile Loads and Properties Alternative-HR2

		No. of Piles				
		24	**16**	**12**	**10**	**8**
Shallow Channel 1	**Pylon Pier:**					
	Maximum Axial Load (kN)	**59000**	84000	-	-	-
	Maximum Bending Moment (kNm)	**130000**	206400	-	-	-
	Maximum Shear Force (kN)	**5100**	7700	-	-	-
	Hollow Tube Wall Thickness (mm)	82	119	-	-	-
	Composite Tube Wall Thickness (mm)	**48**	51	-	-	-
	Back Span Pier:					
	Maximum Axial Load (kN)	-	-	-	26500	**29500**
	Maximum Bending Moment (kNm)	-	-	-	29300	**43500**
	Maximum Shear Force (kN)	-	-	-	1200	**1800**
	Hollow Tube Wall Thickness (mm)	-	-	-	50	**50**
	Composite Tube Wall Thickness (mm)	-	-	-	48	48
SC1	**Outer Pier:**	-	-	As Alternative-HR		
	Inner Pier:	-	-			
SC2	**Outer Pier:**	-	-	As Alternative-HR		
	Inner Pier:	-	-			
DC	**Outer Pier:**	-	-	As Alternative-HR		
	Inner Pier:	-	-			

Table 3.3.27 Ultimate Limit State Pile Loads and Properties Alternative-H3

		No. of Piles		
		12	**10**	**8**
Shallow Channel 1	**Outer Pier:**			
	Maximum Axial Load (kN)	-	-	**53000**
	Maximum Bending Moment (kNm)	-	-	**106000**
	Maximum Shear Force (kN)	-	-	**3200**
	Hollow Tube Wall Thickness (mm)	-	-	**64**
	Composite Tube Wall Thickness (mm)	-	-	48
	Inner Pier:			
	Maximum Axial Load (kN)	-	-	**53700**
	Maximum Bending Moment (kNm)	-	-	**61800**
	Maximum Shear Force (kN)	-	-	**1800**
	Hollow Tube Wall Thickness (mm)	-	-	**50**
	Composite Tube Wall Thickness (mm)	-	-	48
Shallow Channel 2	**Outer Pier:**			
	Maximum Axial Load (kN)	-	-	**51800**
	Maximum Bending Moment (kNm)	-	-	**102000**
	Maximum Shear Force (kN)	-	-	**3300**
	Hollow Tube Wall Thickness (mm)	-	-	**62**
	Composite Tube Wall Thickness (mm)	-	-	48
	Inner Pier:			
	Maximum Axial Load (kN)	-	-	**52000**
	Maximum Bending Moment (kNm)	-	-	**54700**
	Maximum Shear Force (kN)	-	-	**1800**
	Hollow Tube Wall Thickness (mm)	-	-	**50**
	Composite Tube Wall Thickness (mm)	-	-	48
Deep Channel	**Outer Pier:**			
	Maximum Axial Load (kN)	46300	**53800**	68200
	Maximum Bending Moment (kNm)	80800	**108600**	152800
	Maximum Shear Force (kN)	2100	**2700**	3300
	Hollow Tube Wall Thickness (mm)	50	**61**	82
	Composite Tube Wall Thickness (mm)	48	48	48
	Inner Pier:			
	Maximum Axial Load (kN)	46300	**53800**	69600
	Maximum Bending Moment (kNm)	55000	**58500**	97200
	Maximum Shear Force (kN)	1400	**1500**	2000
	Hollow Tube Wall Thickness (mm)	50	**50**	65
	Composite Tube Wall Thickness (mm)	48	48	48

(3) Pile Cap Design

(a) Geometry

The pile cap geometry has been determined after consideration of the following criteria.

- Maximum soffit level to be 2m below Standard Low Water Level in order to protect the piles from direct ship impact.
- Top of pile cap to have sufficient clearance above Standard High Water Level to maximize window for in-situ construction option.
- Rounded edges to be provided to minimize impedance of river flow and subsequent loads on the substructure.
- Pile cap plan dimensions will be determined by constraints of minimum pile and pier column spacing.

(b) Load Effects

The factored (Ultimate Limit State) load effects for each bridge alternative and pier group considered are shown in Tables 3.3.28 and 3.3.29 together with overall pile cap plan dimensions. These values represent the average effect across the pile cap in. order to derive material quantities and costs.

All extradosed alternatives are generally dominated by the vertical and seismic load combinations 3 and 10, respectively. In addition, the temperature restraint combination 7 is also critical for these outer piers. Load combination 3 is critical for the greater mass structures such as the cable stay pylon piers, whilst the ship collision combination 11 is also critical for the lesser mass structure such as Alternative-H3 and the cable stay back-span piers.

It should be noted that there is generally less than 10% variation in pile cap load effects due to load combinations 3, 7, 10 and 11 for each bridge alternative.

(4) Pier Column Design

(a) Geometry

The spacing of the twin pier columns is primarily dictated by the geometry of the superstructure and cable pylons above. The overall dimensions of each pier column are then dictated by the vertical and horizontal loads acting upon it. Each column is considered to have a hollow rectangular section in plan for the purpose of this preliminary design. However, this shape may be tailored to suite aesthetic requirements during the detailed design stage. The voided core is filled with mass concrete to a level of 4.0m above Standard High Water Level in order to protect the columns from direct ship impact.

(b) Load Effects

The factored (Ultimate Limit State) load effects for each bridge alternative and pier group considered are shown in Table 3.3.30 together with their proposed column dimensions.

Bending moments on the twin columns are tabulated for the bridge longitudinal direction only, as these will be critical due to their monolithic and cable connection to the superstructure above. For this stage of the design it is considered appropriate for the transverse bending moments to be resolved into a couple acting on the columns.

All extradosed Outer Pier column designs are dominated by the temperature restraint and ship collision load combinations 7 and 11 respectively. However, the Inner Pier column design is controlled by the vertical and seismic load combinations 3 and 10, for railway and road only alternatives respectively, together with the ship collision load combination 11. The cable stay Back-span Pier design is governed by the ship collision load combination 11 whilst the Pylon Pier design is controlled by the vertical and seismic load combinations 3 and 10.

(5) Major Quantities

As there are six different arrangements of pier for each extradosed bridge alternative, that is inner and outer piers for the deep, shallow 1 and shallow 2 channels, Table 3.3.31 shows the average quantity of materials required to construct one extradosed bridge pier for each alternative.

The reinforcement and formwork quantities tabulated for the pile cap is for the permanent case, external face only. These values may be increased by up to 100% depending on the contractors' preferred method of construction.

Table 3.3.28 Ultimate Limit State Pile Cap Loads and Properties - Extradosed Alternatives

		Outer Pier		Inner Pier	
		Longitudinal	Transverse	Longitudinal	Transverse
Alternative-H1					
SC1	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	7200	21200	6600	21200
	Moment - Hog (kNm/m)	7800	9200	5700	9200
	Shear (kN/m)	3700	4500	2700	4500
SC2	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	7600	21400	6600	21400
	Moment - Hog (kNm/m)	8200	10600	5700	10600
	Shear (kN/m)	3900	4700	2700	4700
DC	Plan Dimension (m)	17.5	37	17.5	37
	Moment - Sag (kNm/m)	5500	16700	3800	16600
	Moment - Hog (kNm/m)	5500	8700	3600	8900
	Shear (kN/m)	3100	4000	2400	4000
Alternative-HR					
SC1	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	5800	18500	4800	18300
	Moment - Hog (kNm/m)	7700	6800	4900	6800
	Shear (kN/m)	3700	3900	2300	3900
SC2	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	6000	18600	5000	18400
	Moment - Hog (kNm/m)	7900	6900	4800	7100
	Shear (kN/m)	3800	4000	2400	4000
DC	Plan Dimension (m)	17.5	37	17.5	37
	Moment - Sag (kNm/m)	7700	20300	4700	20400
	Moment - Hog (kNm/m)	7400	10600	4400	10900
	Shear (kN/m)	4100	4900	3000	4900
Alternative-H3					
SC1	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	7500	19400	5800	19300
	Moment - Hog (kNm/m)	7200	10400	5000	10400
	Shear (kN/m)	3800	4400	2400	4400
SC2	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	7200	18900	5800	18900
	Moment - Hog (kNm/m)	7300	9400	9400	9400
	Shear (kN/m)	3600	4200	2300	4200
DC	Plan Dimension (m)	17.5	31.5	17.5	31.5
	Moment - Sag (kNm/m)	5800	14500	3800	15000
	Moment - Hog (kNm/m)	6600	8900	6500	9200
	Shear (kN/m)	3500	3900	2100	3900

Table 3.3.29 Ultimate Limit State Pile Cap Loads and Properties – Cable Stay Alternatives

		Back-span Pier		Pylon Pier	
		Longitudinal	Transverse	Longitudinal	Transverse
Alternative-H2					
SC1	Plan Dimension (m)	17.5	31.5	35	54
	Moment - Sag (kNm/m)	2900	8600	22000	29300
	Moment - Hog (kNm/m)	2800	1000	8400	4800
	Shear (kN/m)	1500	3000	4600	3000
	Extradosed Inner and Outer Piers as Alternative-1				
Alternative-HR2					
SC1	Plan Dimension (m)	17.5	31.5	35	54
	Moment - Sag (kNm/m)	3200	10500	28700	32400
	Moment - Hog (kNm/m)	2800	7800	11600	6600
	Shear (kN/m)	1500	3000	5900	3600
	Extradosed Inner and Outer Piers as Alternative-HR				

Table 3.3.30 Ultimate Limit State Pier Column Loads and Properties

		Plan Dimensions (mm)			Axial Load (kN)		Moment	Shear (kN)	
		Breadth	Depth	Wall thk.	Max.	Min.	(kNm)	Long.	Tran.
Alternative-H1									
SC1	Outer Pier	5500	6000	1250	144400	122700	211900	11600	28400
	Inner Pier	5500	6000	1250	145100	123300	196200	14800	28400
SC2	Outer Pier	5500	6000	1250	141400	118800	169600	13900	28400
	Inner Pier	5500	6000	1250	142000	119400	140600	14700	28400
DC	Outer Pier	5500	6000	1250	141400	118800	169600	13900	28400
	Inner Pier	5500	6000	1250	142000	119400	140600	14700	28400
Alternative-H2									
SC1	B-span Pier	4000	4000	1250	25900	3700	23500	14200	28400
	Pylon Pier	9000	9000	3000	320400	281100	1632500	41700	28400
	Extradosed Outer and Inner Piers as Alternative-H1								
Alternative-HR									
SC1	Outer Pier	7000	6000	1250	181800	140700	247800	17300	28400
	Inner Pier	7000	6000	1250	182900	141800	227900	11500	28400
SC2	Outer Pier	7000	6000	1250	178200	137000	311600	18200	28400
	Inner Pier	7000	6000	1250	179300	138100	189000	11400	28400
DC	Outer Pier	7000	6000	1250	178200	137000	311600	18200	28400
	Inner Pier	7000	6000	1250	179300	138100	189000	14800	28400
Alternative-HR2									
SC1	B-span Pier	4000	4000	1250	32700	4100	23500	14200	28400
	Pylon Pier	9000	9000	3000	366200	300900	2061000	63700	28400
	Extradosed Outer and Inner Piers as Alternative-HR								
Alternative-H3									
SC1	Outer Pier	5000	6000	1250	123700	105400	172300	11400	28400
	Inner Pier	5000	6000	1250	123800	105500	159500	14700	28400
SC2	Outer Pier	5000	6000	1250	119900	100600	159700	13700	28400
	Inner Pier	5000	6000	1250	112000	100600	114400	14600	28400
DC	Outer Pier	5000	6000	1250	119900	100600	159700	13700	28400
	Inner Pier	5000	6000	1250	112000	100600	114400	14600	28400

Table 3.3.31 Quantities of Major Materials for Extradosed Bridge Pier

Element	Materials	Quality	Quantity	Unit	Minimum Width (17.1m wide)	Base Case (21.5m wide)	Railway Provision (25m wide)
Pier Column	Concrete	σ_{ck}=50N/mm^2	Volume	m^3	850	967	1111
	Reinforcement	SD345	Weight	ton	161	225	268
			Density	kg/m^3	253	310	320
	Formwork		Outer Area	m^2	801	1006	1077
			Inner Area	m^2	405	502	557
Pile Cap	Concrete	σ_{ck}=50N/mm^2	Volume	m^3	3914	4005	4005
	Reinforcement	SD345	Weight	ton	213	218	212
			Density	kg/m^3	55	54	53
	Formwork		Area	m^2	1075	1098	1098
Piles	Concrete	σ_{ck}=20N/mm^2	Volume	m^3	475	491	664
	Steel Tube	FE510	Diameter	mm	3150	3150	3150
			Length	m	664	729	862
			No	-	8.3	8.5	10.3
			Weight	ton	3150	3409	4092

3.3.6 Construction and Maintenance of Extradosed Bridges

(1) Construction

The extradosed girder bridge was originally developed in France and is now an increasingly popular form of construction in Asia, the Middle East, Europe and the USA. A sample list of this type of bridge constructed to date, together with the contractors responsible for their construction, is shown in Table 3.3.32. Many more of this form of bridge structure is also in the planning stage world wide, and will be complete by the time of the Padma Bridge tender offer.

The structural system of an extradosed girder has the combined characteristics of an externally prestressed structure and a cable-stayed structure. Moreover, the relative stiffness of the superstructure gives a structural behavior tending to the externally prestressed structure. It is therefore considered that any construction contractor with experience of these forms of structures should be competent to tender for the Padma Bridge superstructure contract.

The external prestressed form of structure referred to above is one where the longitudinal prestress tendons are not encased within the superstructure reinforced concrete web and flange. They are generally adjacent the flange within the superstructure void for easy inspection and maintenance. The deflected tendon shape is maintained by a series of deflector blocks or saddles attached to or cast with the concrete web and flange. This arrangement has been developed to minimize the consequences of incomplete tendon grout and to provide enhanced inspection.

As there will be many contractors competent to tender for these technically demanding works, additional emphasis should be given to their quality of performance. That is, the implementation of quality assurance and management procedures that ensure the quality control of materials and workmanship used on the project. This will reduce the risk of delays to the construction programme and the subsequent maintenance required throughout the design life of the structure.

Particular emphasis should be given to the following during superstructure construction:

- Consistent attainment of concrete target strength to ensure strength and durability
- Concrete workability to assist flow and prevent segregation, bleeding, honeycombing and voids
- Correct concrete curing procedure
- Monitoring and testing of aggregates, cement and water used for concrete production
- Validation of reinforcement, prestress and extradose cable tendons and accessories
- Maintain reinforcement in the design position
- Correct tension and sequence of tensioning applied to the prestress and extradosed cable tendons.
- Particular care given to alignment and quality of tendon, ducts, anchorages and deflectors both in the superstructure and cable pylons.
- Constant monitoring of the superstructure deflection and comparison with the design values.
- Construction plant load does not exceed that assumed for the design
- Corrosion protection is applied correctly to all exposed prestress and extradose cables and protected during construction.

Table 3.3.32 Examples of Extradosed Bridge Construction

Bridge	Max Span (m)	Usage	Completed	Country	Contractor
Himi Bridge	180	Road	-	Japan	Sumitomo Corporation Zentitake Corporations
Hozu Bridge	100	-	2001	Japan	-
Ibi Gawa Bridge	271.5	Road	2001	Japan	DPS Bridge Works Mitsubishi Heavy Industries Ltd PS Corporation Sumitomo Construction Taisei Corporation Yokogawa Bridge Corporation
Keong-An Bridge	270	Road	2003	South Korea	Hyundai Engineering and Construction
Kiso Gawa Bridge	275	Road	2001	Japan	Kajima Corporation Nippon Koatsu NKK Corporation Oriental Construction Co. Zenitake Corporation
Mactan-Mandaue Bridge, Second	185	Road	-	Philippines	Kajima Corporation Sumitomo Construction
Miyakodagawa Bridge	133	-	2000	Japan	-
Odawara Blueway Bridge	122	Road	1994	Japan	-
Pakse Bridge	143	Road	2000	Laos / Thailand	Hazama Corporation Shimizu Corporation
Pyung-Yeo 2 Bridge	120	Road	Under construction	South Korea	SK Engineering and Construction
Rittoh Bridge	170	Road	Under construction	Japan	DPS Bridge Works Japan Industrial Land Development Maeda Corporation PS Mitsubishi Construction
Saint-Rémy-de-Maurienne, Pont de	52.5	Road	-	France	Fougerolle Gerland Nicoletti Spada
Sapporo Railway Bridge	55	Railway	1999	Japan	Taisei Corporation
Shikari Bridge	140	-	2000	Japan	Kajima Corporation Oriental Construction Co.
Shin-Karato Bridge	140	Road	1998	Japan	Nippon Koatsu Concrete Oriental Construction Co. PS Corporation
Socorridos, Ponte dos	106	-	1993	Portugal	-
Tsukuhara Bridge	180	Road	1998	Japan	-

(2) Maintenance

As mentioned previously the extradosed girder bridge has many similar characteristics to the externally prestressed girder bridge and as such requires a similar maintenance regime. The major difference between these systems is that the extradosed girder has an effective continuation of the external prestress cables extending above the deck level and anchored to the cable pylons. This arrangement therefore requires a different access system during inspection and maintenance periods. However, as the towers are short, 16m above deck level, compared to cable stay pylons, they will be accessible by means of standard inspection plant and equipment.

A major maintenance benefit of the proposed extradosed structure for the main Padma River Bridge is that monolithic connections are proposed at the pier supports negating the requirements for bearings and their subsequent maintenance and potential replacement.

Regardless of quality of design and construction it is inevitable that a bridge structure will suffer some degradation due to the effects of age, environment and loading that often exceeds the design code requirements. The extent of this degradation is highly influenced by the extent of the inspection and maintenance programme.

A typical inspection program would include but not be limited to the following:

- Inspection for unusual cracks or deformations to concrete particularly to high stress areas
- Decay of expansion joints and uneven road levels either side of the joint
- Change to structure geometry
- Evidence of corrosion to reinforcement, other embedded steel and prestress / extradose cable
- Erosion and abrasion of the substructure due to fast moving particles / objects in the river
- Collision damage
- Fire damage
- Deterioration of concrete due to chemical agents
- Integrity of corrosion protection
- Deflections due to dead and live loading

This should be coupled with a regular or, depending on the results of the above, a special testing programme to determine the cause of any deterioration together with maintenance and repair proposals.

3.4 BRIDGE DESIGN OPTIONS

As stated in 3.2.1 the main bridge portion is 5400 m long. This can be divided by 180 m, the most preferable span length recommended in 3.2.3, and then it makes a span arrangement of (30x180m).

Taking into account the thermal expansion, one continuous superstructure as a whole cannot be longer than approximately 750 meters, thus (4x180m=720m) constitutes a standard module of superstructure, and expansion joints must be placed at spacing of 720 meters or less between the neighboring structures.

In case of adopting a cable stayed girder bridge, as requested by the Bangladesh side, in a part of the main bridge, one module of 720 meters will be substituted by a composite cable stayed girder, of which span arrangement is (48+132+360+132+48=720m).

Regarding the requirements on vertical alignment, major portions of the bridge have to keep the maximum slope of 3%, but partially where the slope is not long enough such as approach viaduct 45 is allowed for roadway.

The maximum slope of the railway is limited to 1%.

At least one or preferably three navigational route has to secure the vertical clearance of 60 feet, ant other route 40 feet for the whole navigational course of 4800 meters wide.

Although it was concluded in 3.2.3 that the options of bridging all 180 meter long spans by the PC Extradosed type is the most favorable in terms of the bridge construction cost, options where a portion of the PC Extradosed is substituted by the composite Cable-stayed bridge, which is strongly requested to include by the Bangladesh side.

Consequently the following four combinations, all of which satisfy the above-mentioned requirements, are to be examined;

1) Alternative-H1: PC Extradosed Girder Bridge without Railway Provision
2) Alternative-H2: PC Extradosed Girder Bridge and PC Cable Stayed Girder Bridge without Railway Provision
3) Alternative-HR: PC Extradosed Girder Bridge with Railway Provision
4) Alternative-HR2: PC Extradosed Girder Bridge and PC Cable Stayed Girder Bridge with Railway Provision
5) Alternative-H3: PC Extradosed Girder Bridge without Railway Provision (Minimum width)

3.4.1 Alternative –H1: PC Extradosed Girder Bridge without Railway Provision

(1) Superstructure

The Figure 3.4.1shows a general arrangement of spans for the Alternative-H1, and Figure 3.4.2 and Figure 3.4.3 show the elevation and cross-section views more in detail.

This is the base case and also the most economical where all the spans at the main bridge portion are composed of PC Extradosed Girders, and the viaducts are of PC T-section girder bridges.
Five modules of 720 meters long and two modules of 540 meters long constitute the main bridge portion of 5400 meters to be preferably spanned by the 180 m extradosed bridges.
The viaducts on both banks are of PC T-girder type, whose span is 30 meters. The left bank viaduct is 60 meters long, and the right 120 meters.
The total bridge length is consequently 5580 meters.

The cross-section of the deck is 21.5 meters, which can accommodate 2-lane for vehicles on each direction with margins for the cable anchors.

(2) Substructure

The Figures 3.4.4 and 3.4.5 show the general arrangements of the edge river and mid river substructures respectively. A more extensive foundation is required to the piers at the edge of the river due to the greater scour resulting in a lower design river bed level.

In order to minimize the effect of river flow the pile caps are provided with rounded corners. The 3150mm diameter hollow steel tubular piles have a maximum rake of 1 in 6 to the vertical, and are to be in-filled with a mass concrete toe plug to generate additional end bearing.

The four edge river piers comprise a 17.5m x 37.0m x 7.1m deep pile cap supported on 12 number piles with average maximum wall thickness of 58mm. Average toe elevation is -87.5m PWD providing an approximate depth of embedment of 43m below the design river bed level.

The twenty-six mid river piers comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number piles with average maximum wall thickness of 63mm. Average toe elevation is -83.5m PWD providing an approximate depth of embedment of 53m below design river bed level.

The twin pier columns forming the base of the extradosed cable towers are each 5.5m x 6.0m in overall cross sectional area with minimum wall thickness of 1.25m, with similar connecting cross-beam 5.5m deep. This cross beam may be alternatively hidden within the superstructure.

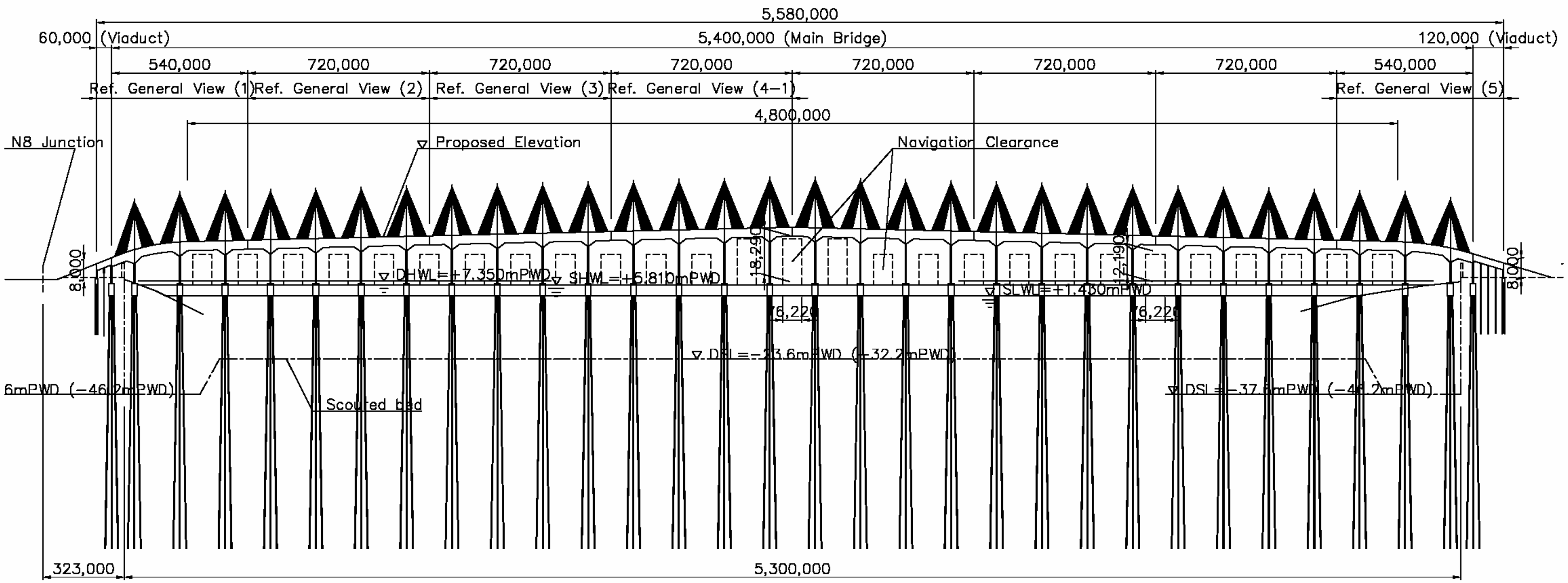


Figure 3.4.1 General View (Alternative-H1) (Alternative-1)

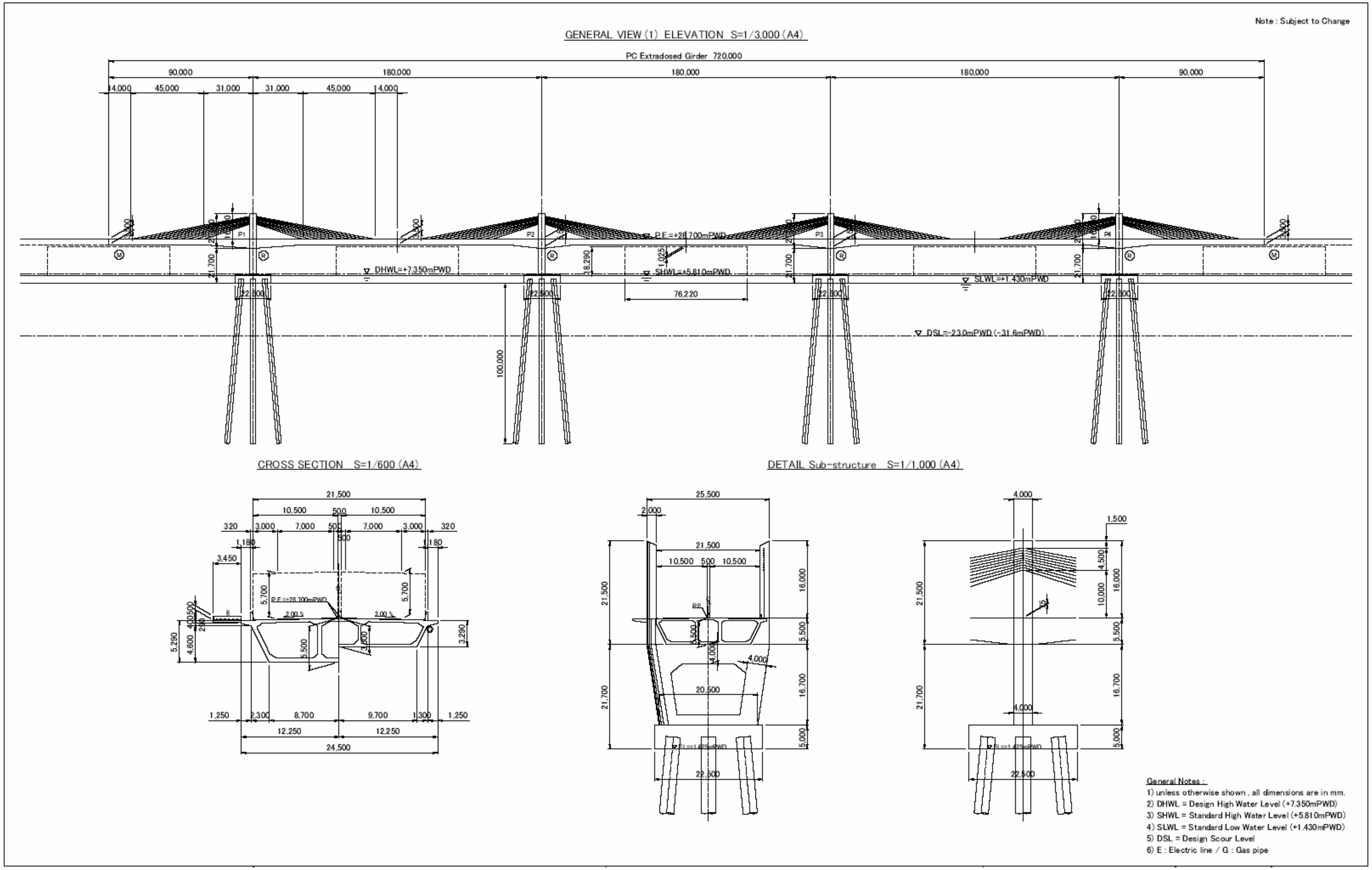


Figure 3.4.2 Main Bridge Portion (Alternative-H1, H2)

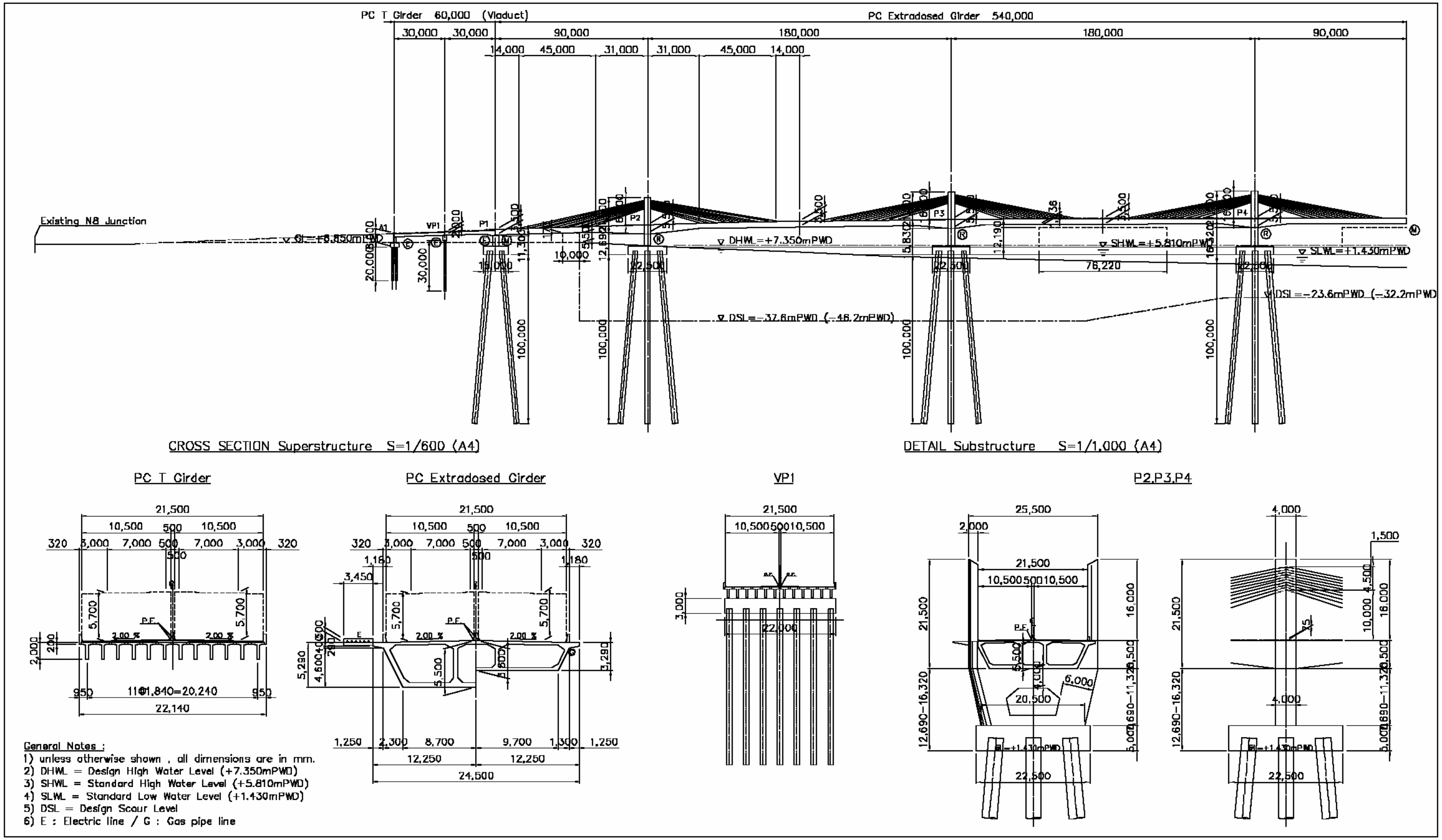


Figure 3.4.3 Viaduct Portion (Alternative-H1, H2)

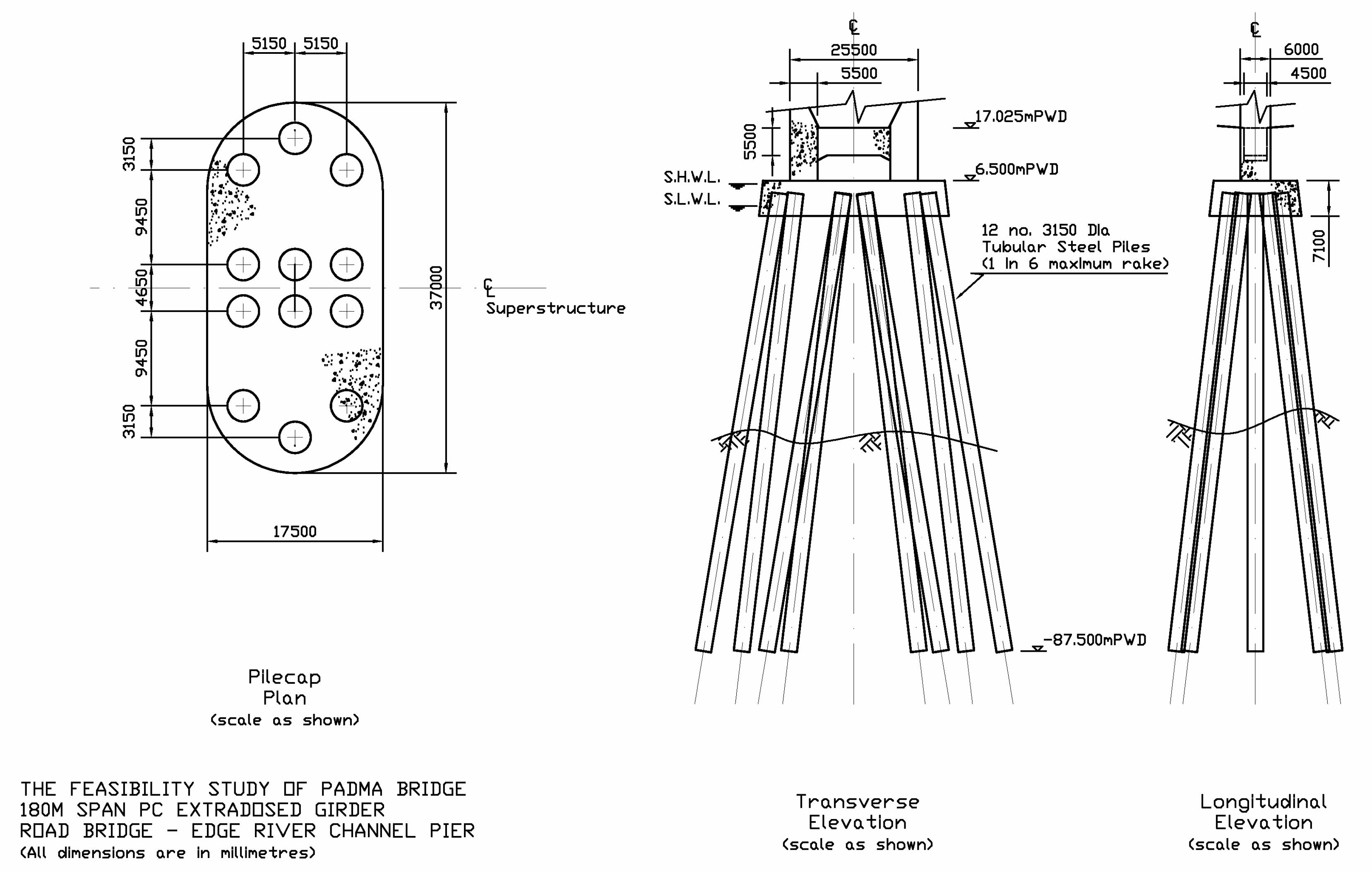


Figure 3.4.4 Edge River Substructure General Arrangement (Alternative-H1)

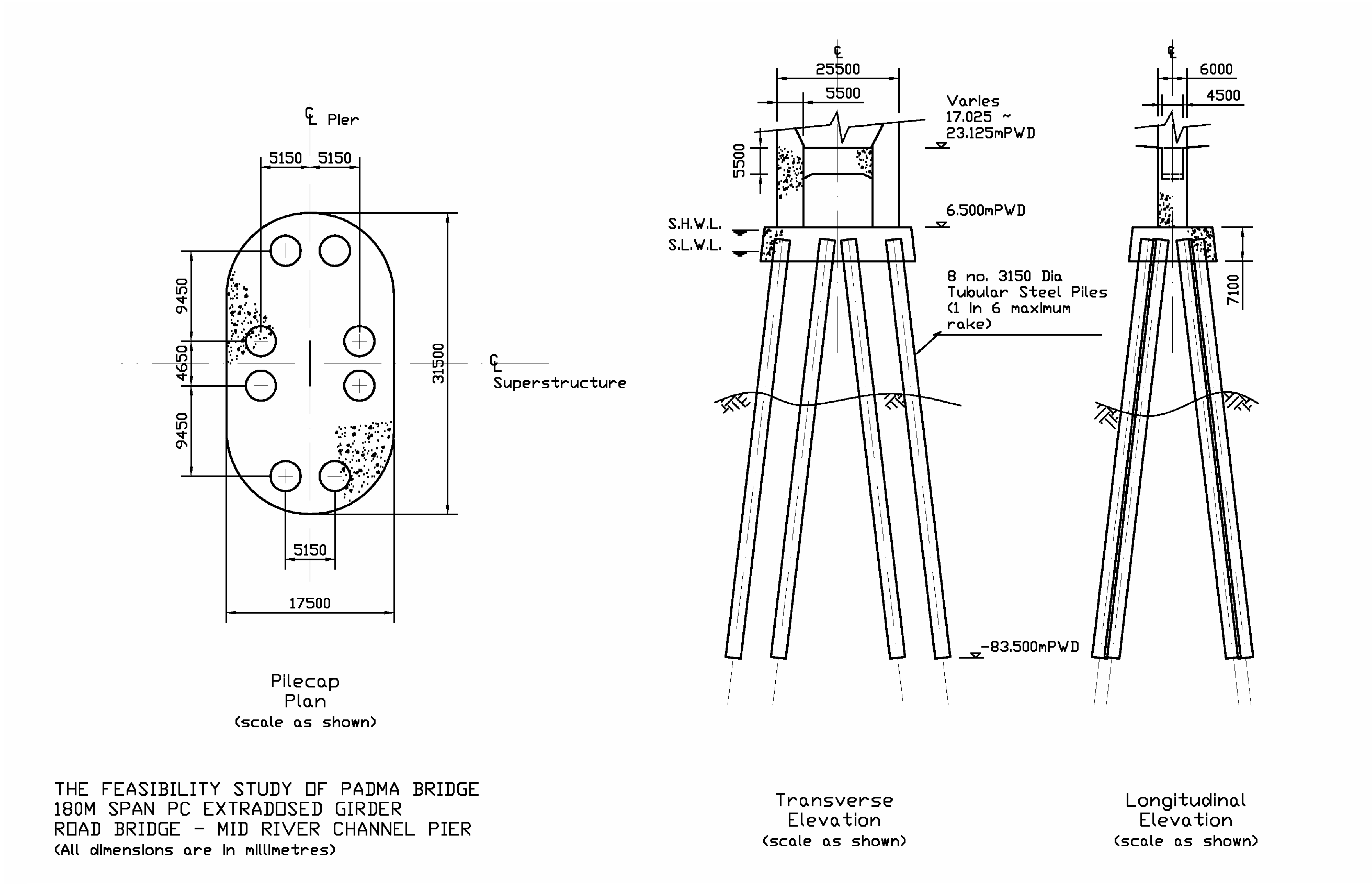


Figure 3.4.5 Mid-River Substructure General Arrangement (Alternative-H1)

3.4.2 Alternative –H2: PC Extradosed Girder Bridge and PC Cable Stayed Girder without Railway Provision

(1) Superstructure

The Figure 3.4.6 shows a general arrangement of spans for the Alternative-H2.

Only the difference from that of the Alternative-H1 is that one module of 720 meters long is substituted by a composite cable stayed girder bridge, and all other elements are of the exactly same as in the Alternative-H1.

The cable stay bridge spans 360 meters which is unnecessarily wide for the required navigation clearance, and on the contrary several piers must be added in the back spans in order to pull down the cable tension forces coming from the main span. This substitution will surely raise the construction cost.

Other disadvantages of the substitution by the composite cable stay bridge are repaint work of the steel deck girder and aerodynamic instability. Steel deck girder is adopted for the main span to reduce the self-weight and this element usually has to be repaired every twenty years. As the cable stay bridge is much longer than the extradosed bridge, it is comparatively more instable aerodynamically. Further studies must be made not only against vortex shedding but flutter oscillation, which is divergent amplitude response to the wind.

Three main navigation routes 60 feet high are secured below the cable stay bridge, but total number of the routes decreases from 27 to 26 by the subsutiturion.

(2) Substructure

The form of substructure supporting the extradosed bridge arrangement is identical to Alternative-H1.

Two forms of pier and substructure are required to support the cable stayed bridge module. The Pylon Piers will flank the main span and carry the majority of the superstructure load via the cable stays. The Back Span Piers will carry the tensile cable forces together with direct loads from river influences. All pile caps are provided with rounded corners to minimize the effect of river flow, and are supported by 3150mm diameter steel tubular piles with a maximum rake of 1 in 6 to the vertical.

The two pylon piers each consist of a 35.0m x 54.0m x 7.1m deep pile cap supported on 24 number composite steel piles with maximum wall thickness of 54mm. Pile toe elevation is -79.0m PWD providing an approximate depth of embedment of 49m below the design river bed level. The twin pier columns forming the base of the cable stay pylons are each 9.0m x 9.0m in overall cross sectional area at their base with wall thickness of 3m. A cross beam of approximate dimensions 2.0 x 3.0m is located below the superstructure level to provide transverse stiffness. The general arrangement of the pylon substructure is shown in Figure 3.4.7.

The four back span piers each consist of a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number hollow steel tubular piles, maximum wall thickness of 56mm, with mass concrete toe plugs. Pile toe elevation is -55.0m PWD providing an approximate depth of embedment of 25m below the design river bed level. The twin pier columns are each 4.0m x 4.0m in overall cross sectional area with a wall thickness of 1.25m. These support the 2.0 x 3.0m bearing cross beam. The general arrangement of the back span substructure is shown in Figure 3.4.8.

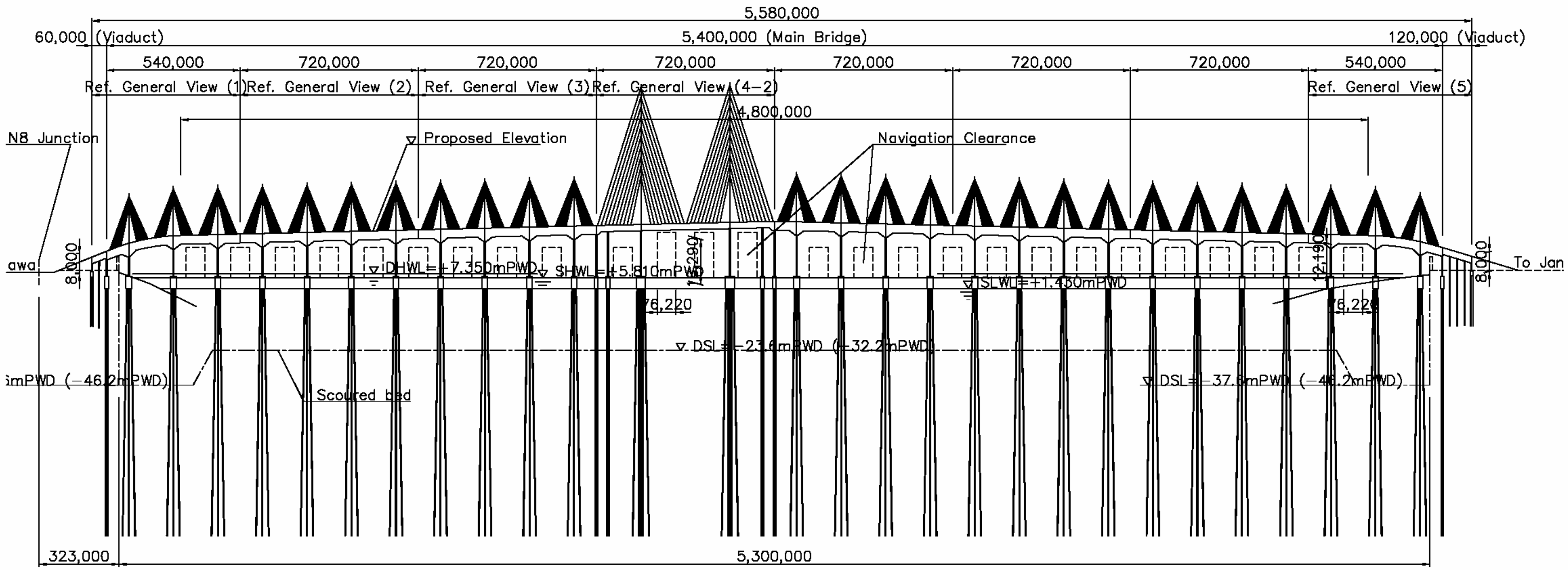


Figure 3.4.6 General View (Alternative-H2)

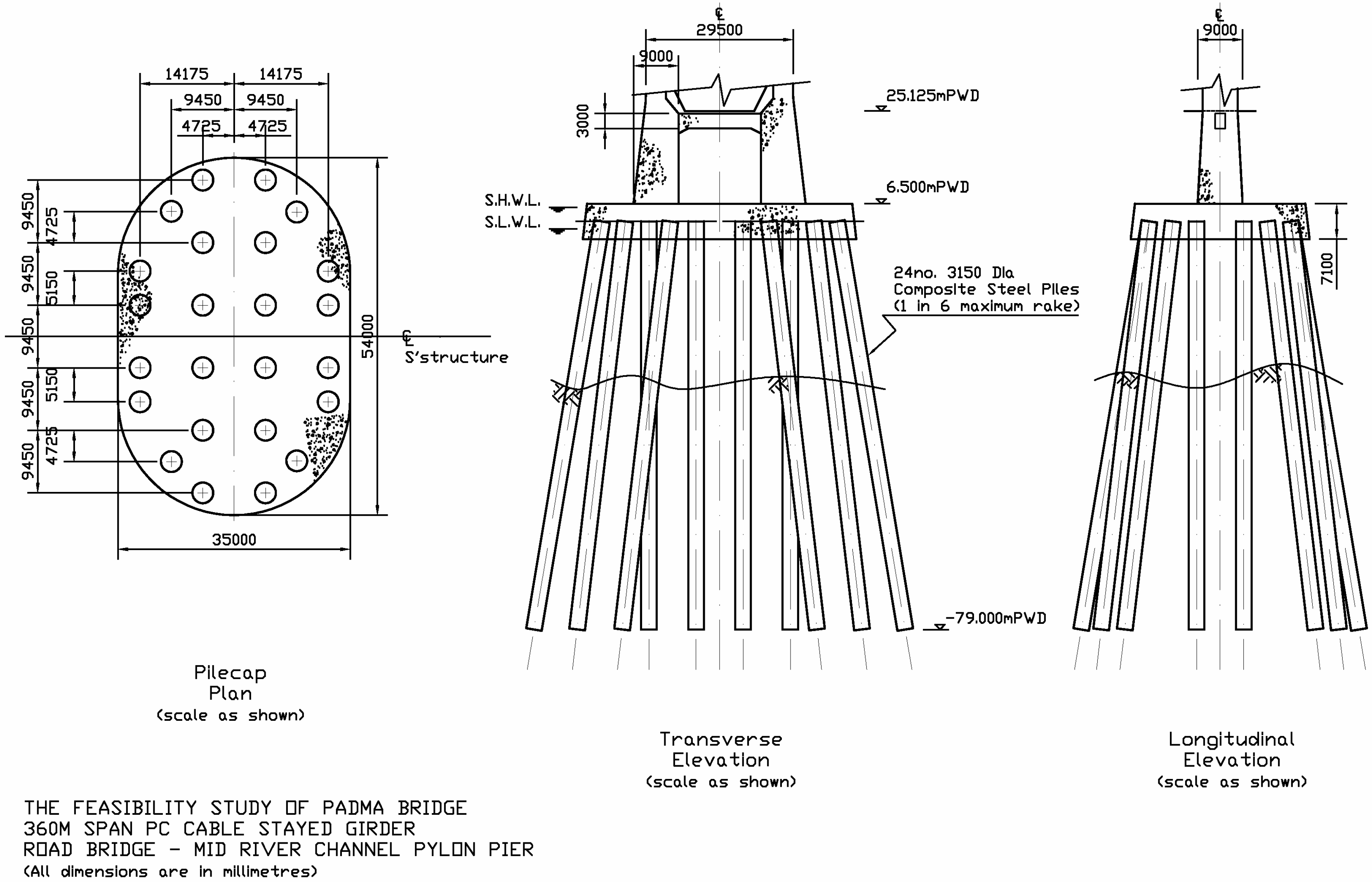


Figure 3.4.7 Mid-River Pylon Substructure General Arrangement (Alternative-H2)

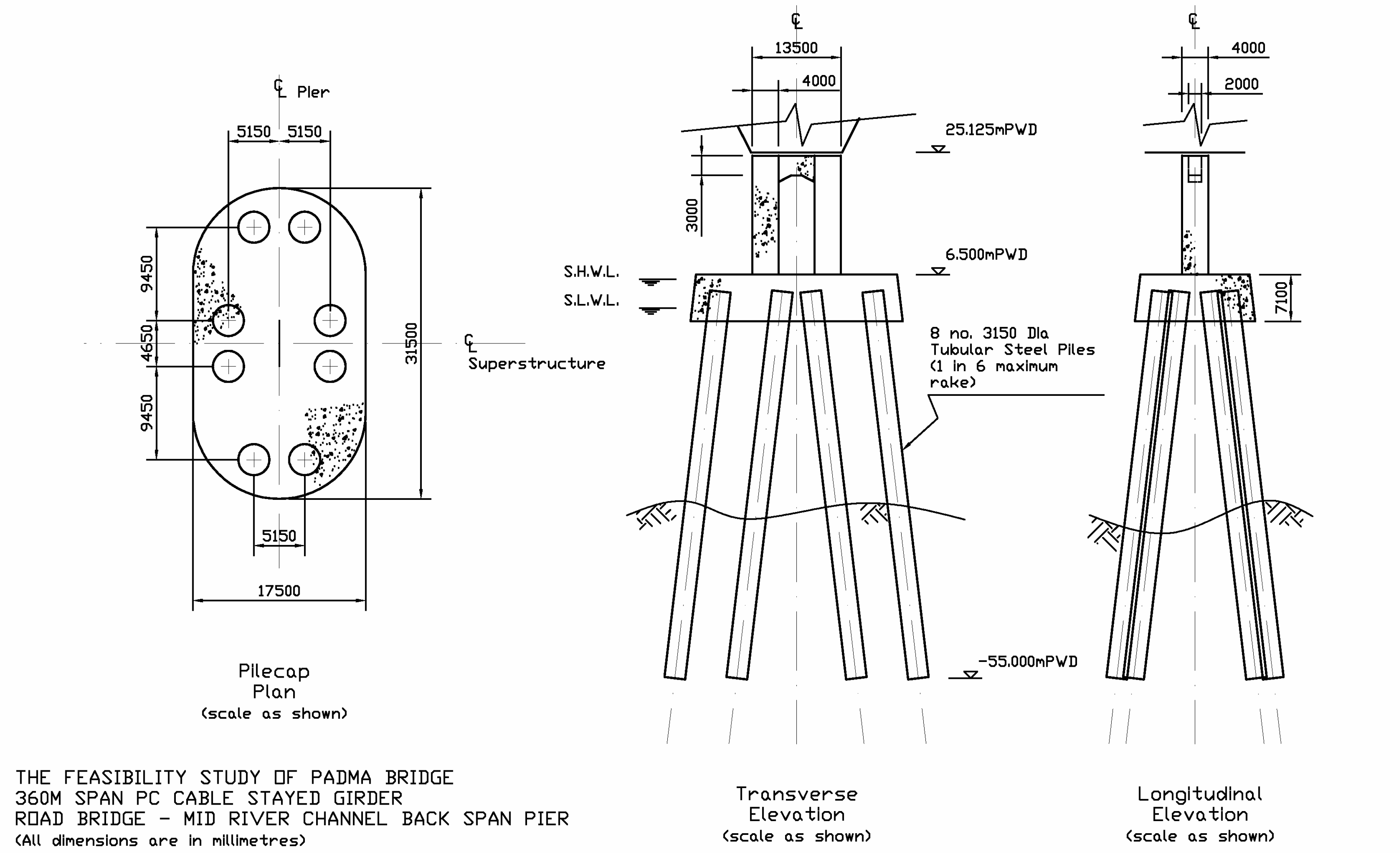


Figure 3.4.8 Mid-River Back Span Substructure General Arrangement (Alternative-H2)

3.4.3 Alternative –HR: PC Extradosed Girder Bridge with Railway Provision

(1) Superstructure

The Figure 3.4.9 shows a general arrangement of spans for the Alternative-HR, and Figure 3.4.10 and Figure 3.4.11 show the elevation and cross-sectional views more in detail.

The main bridge portion, 5400 meters long same as other alternatives, is composed mostly of PC extradosed structures, but PC continuous box girder structures are added on both banks, comprising 7 modules of 720 meters and 2 modules of 360 meters in extradosed structure, and also 2 continuous box structure of 180 meters long.

As this alternative accommodates a railway, the deck is wider by 3.5 meters. Furthermore the difference of the slope limit between the highway and railway has to be considered in the span arrangement, in other words where the highway goes up or down along a slope steeper than 1%, the railway can not go together with the highway on the same deck, and thus they have to be on separated decks. On both ends of the bridge where there are slopes the railway takes gentler gradient away from the highway, as shown in the cross-sectional view of the Figure 3.4.10.

As the deck of this alternative is wider than that of the Alternative-H1, and the live load of the railway is much heavier than that of the highway the construction cost is, as a matter of course, higher.

Same number of navigation routes is to be secured as the Alternative-H1.

(2) Substructure

The Figures 3.4.12 and 3.4.13 show the general arrangements of the edge river and mid river substructures respectively. The addition of railway live load and associated increased superstructure width requires a more substantial substructure compared to Alternative-H1.

In order to minimize the effect of river flow the pile caps are provided with rounded corners. The 3150mm diameter hollow steel tubular piles have a maximum rake of 1 in 6 to the vertical, and are to be in-filled with a mass concrete toe plug to generate additional end bearing.

The four edge river piers comprise a 17.5m x 37.0m x 7.1m deep pile cap supported on 12 number piles with average maximum wall thickness of 66mm. Average toe elevation is -93.0m PWD providing an approximate depth of embedment of 48m below the design river bed level.

The twenty-six mid river piers each comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 10 number piles with average maximum wall thickness of 62mm. Average toe elevation is -80.0m PWD providing an approximate depth of embedment of 49m below design river bed level.

The twin pier columns forming the base of the extradosed cable towers are each 6.0 m x 7.0m in overall cross sectional area with minimum wall thickness of 1.25m, with similar connecting cross-beam 5.5m deep. This cross beam may be alternatively hidden within the superstructure.

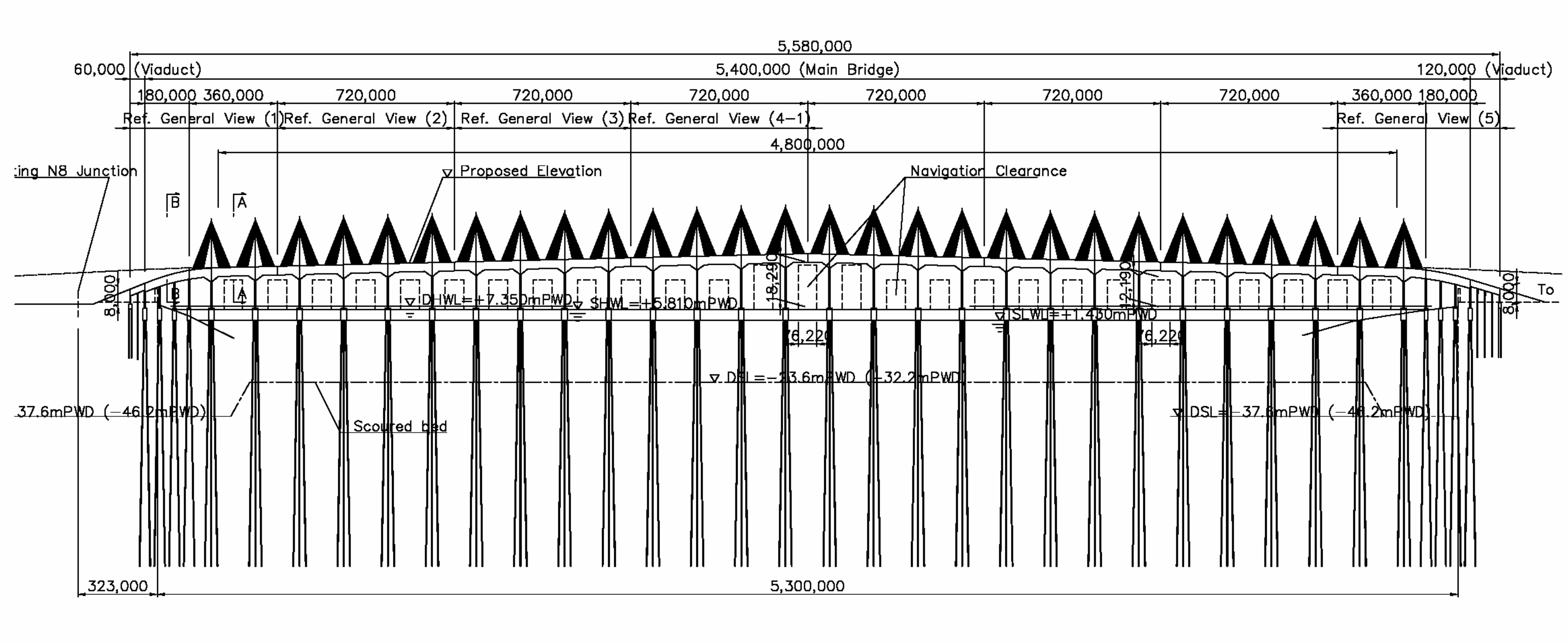


Figure 3.4.9 General View (Alternative-HR)

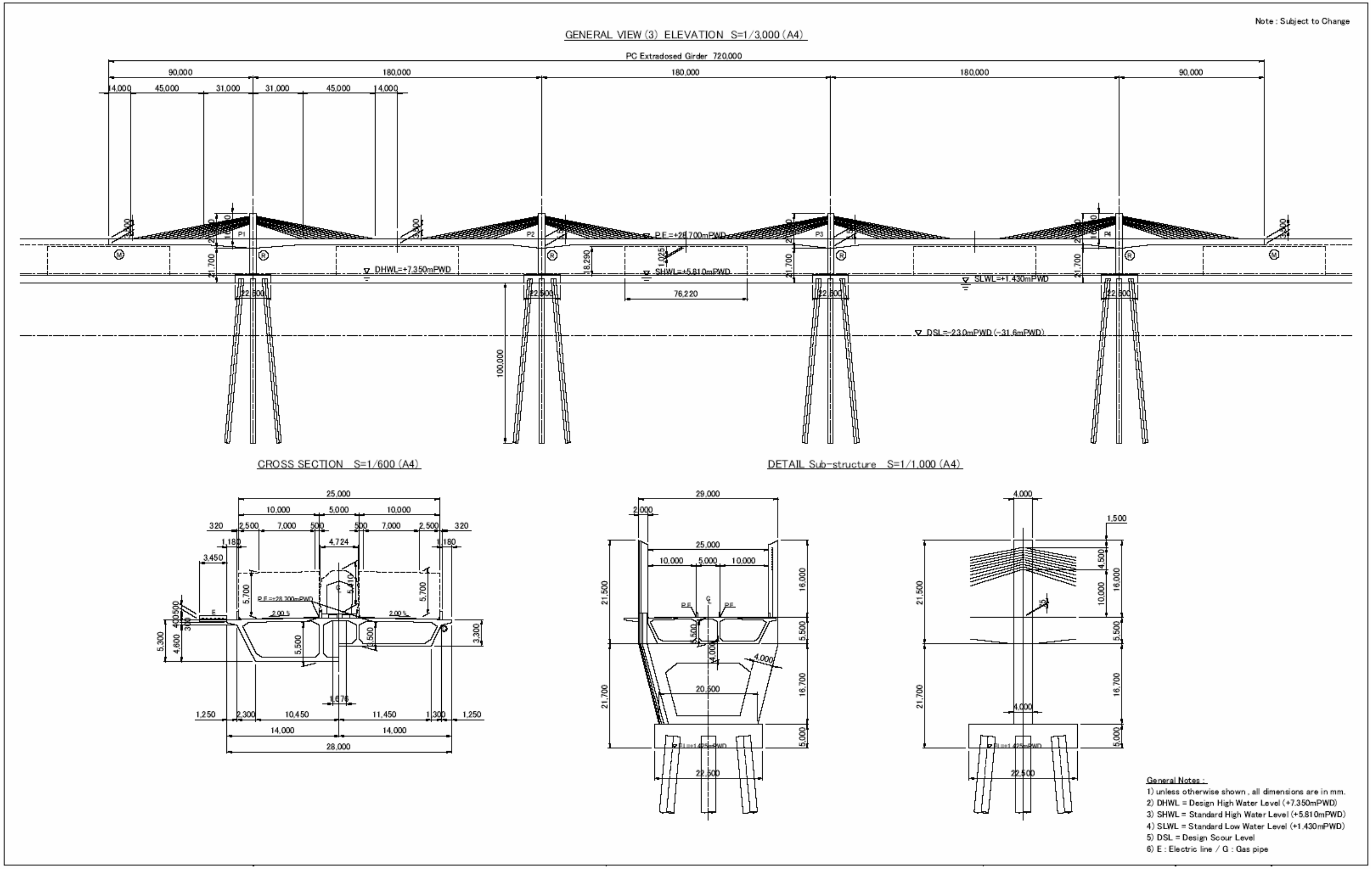


Figure 3.4.10 Main Bridge Portion (Alternative-HR, HR2)

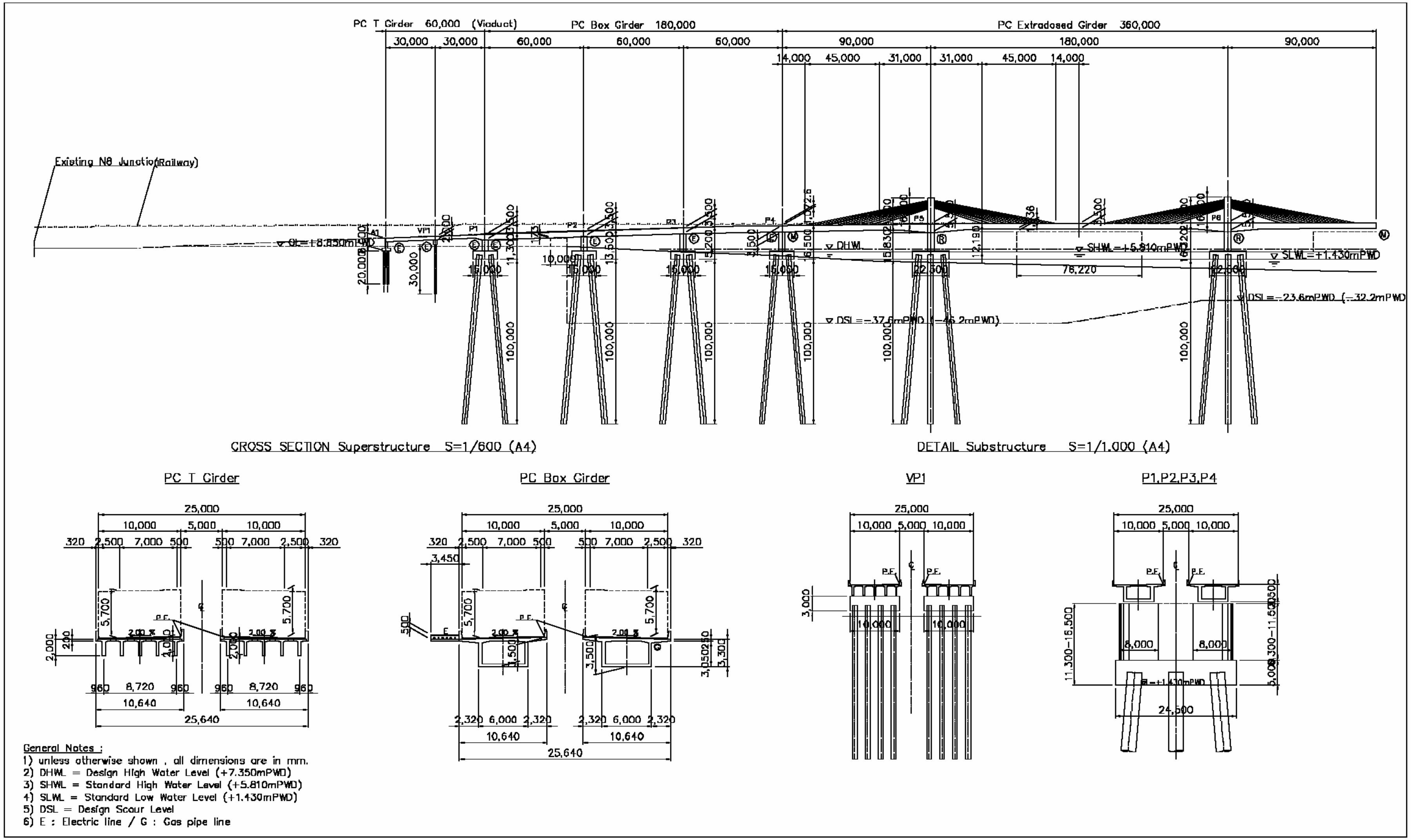
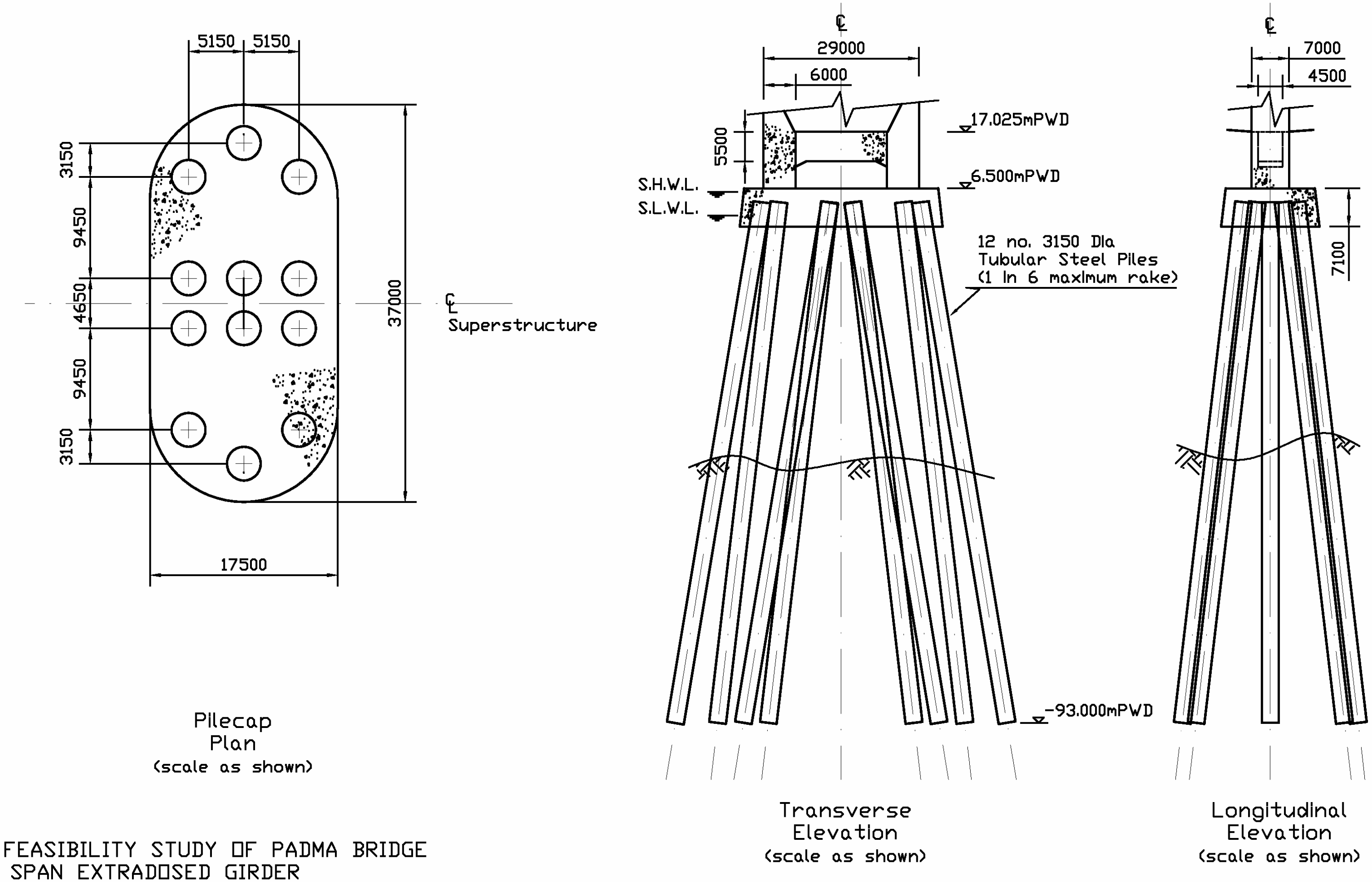


Figure 3.4.11 Viaduct Portion (Alternative-HR, HR2)



THE FEASIBILITY STUDY OF PADMA BRIDGE
180M SPAN EXTRADOSED GIRDER
ROAD BRIDGE WITH RAILWAY PROVISION - EDGE RIVER CHANNEL PIER
(All dimensions are in millimetres)

Figure 3.4.12 Edge River Substructure General Arrangement (Alternative-HR)

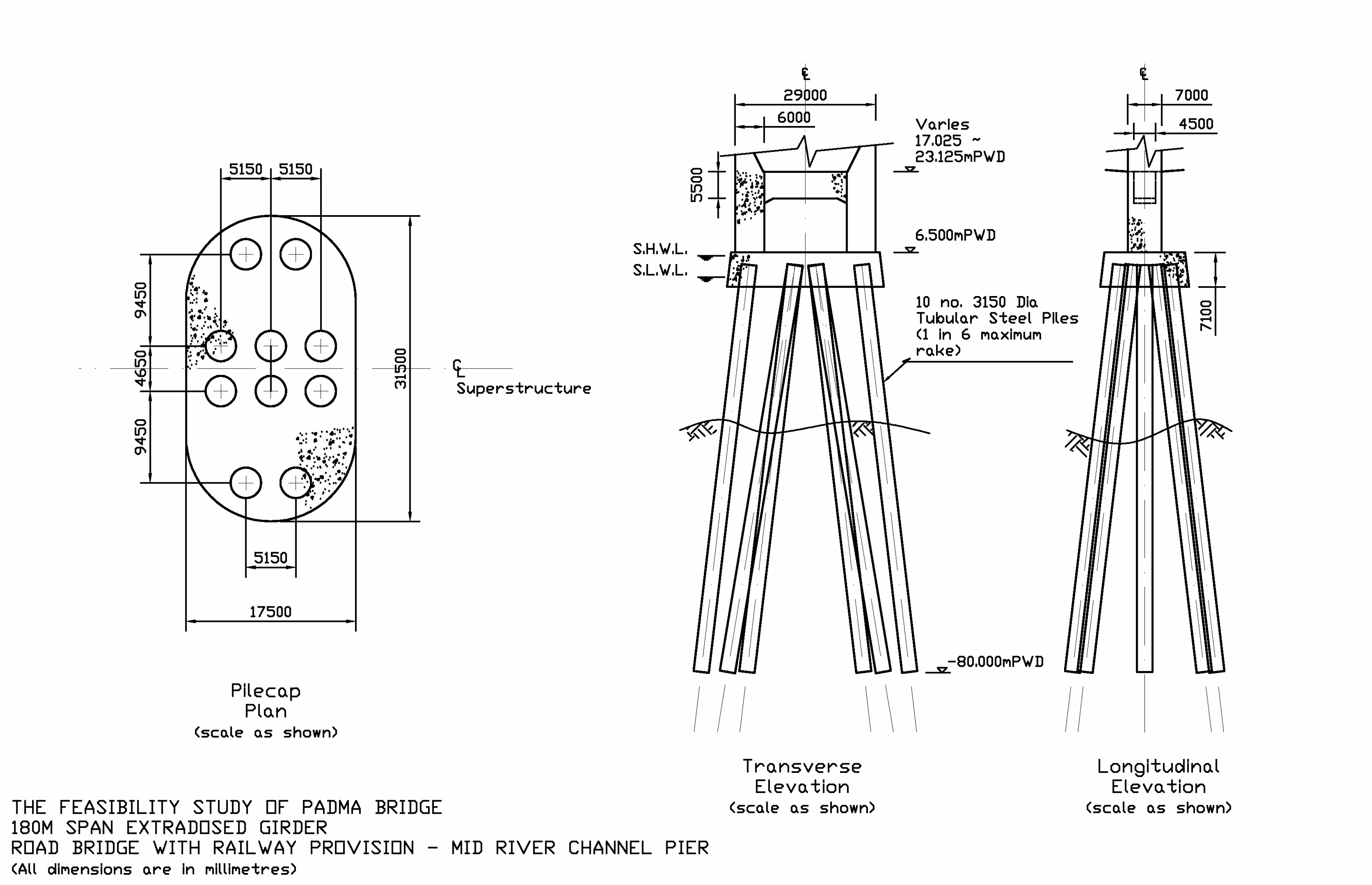


Figure 3.4.13 Mid-River Substructure General Arrangement (Alternative-HR)

3.4.4 Alternative –HR2: PC Extradosed Girder Bridge and PC Cable Stayed Girder with Railway Provision

(1) Superstructure

The Figure 3.4.14 shows a general arrangement of spans for the Alternative-HR2.

This is an alternative having similar characteristics to the Alterntative-H2 and the Alternative-HR at the same time, having a cable stay bridge and railway.

The alternative is the most costly due to the provision of railway and also the long span cable stay bridge.

(2) Substructure

The form of substructure supporting the extradosed bridge arrangement is identical to Alternative-HR.

As Alternative-H2, two forms of pier and substructure are required to support the cable stayed bridge module. The Pylon Piers will flank the main span and carry the majority of the superstructure load via the cable stays. The Back Span Piers will carry the tensile cable forces together with direct loads from river influences. All pile caps are provided with rounded corners to minimize the effect of river flow, and are supported by 3150mm diameter steel tubular piles with a maximum rake of 1 in 6 to the vertical.

The two pylon piers each consist of a 35.0m x 54.0m x 7.1m deep pile cap supported on 24 number composite steel piles with maximum wall thickness of 54mm. Pile toe elevation is -88.0m PWD providing an approximate depth of embedment of 57m below the design river bed level. The twin pier columns forming the base of the cable stay pylons are each 9.0m x 9.0m in overall cross sectional area at their base with wall thickness of 3m. A cross beam of approximate dimensions 2.0 x 3.0m is located below the superstructure level to provide transverse stiffness. The general arrangement of the pylon substructure is shown in Figure 3.4.15.

The four back span piers each consist of a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number hollow steel tubular piles, maximum wall thickness of 56mm, with mass concrete toe plugs. Pile toe elevation is -55.0m PWD providing an approximate depth of embedment of 25m below the design river bed level. The twin pier columns are each 4.0m x 4.0m in overall cross sectional area with a wall thickness of 1.25m. These support the 2.0 x 3.0m bearing cross beam. The general arrangement of the back span substructure is shown in Figure 3.4.16.

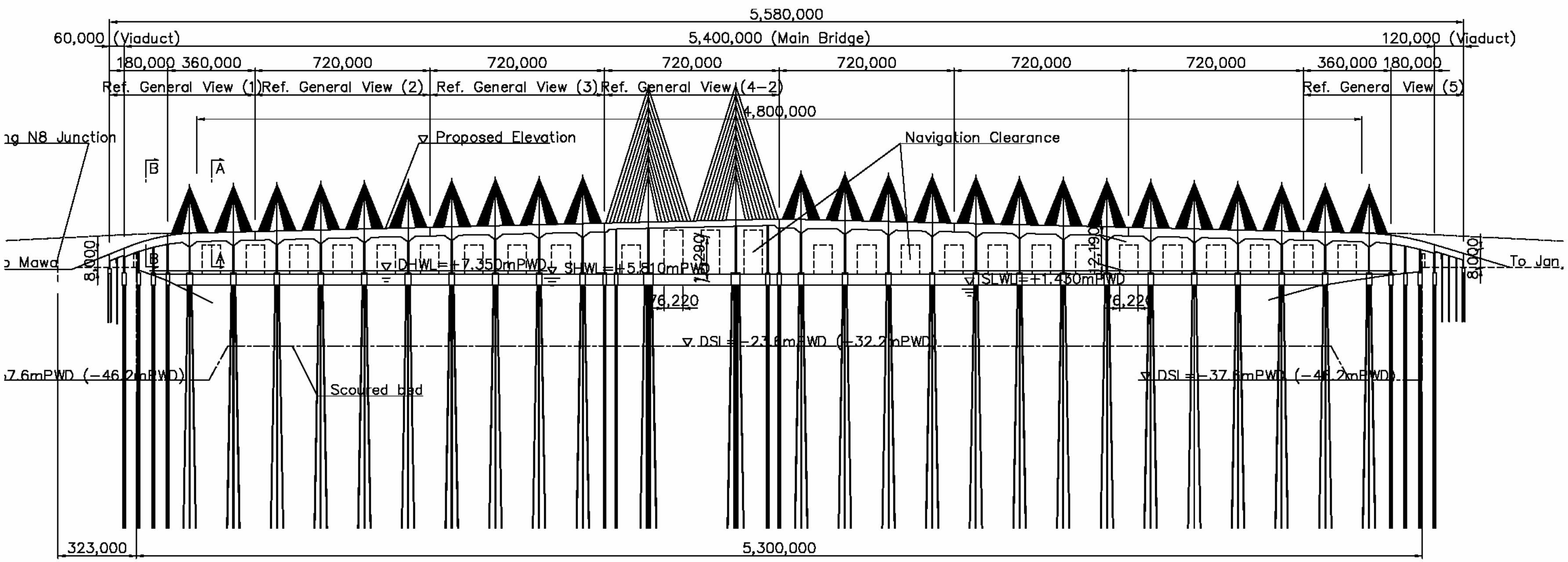


Figure 3.4.14 General View (Alternative-HR2)

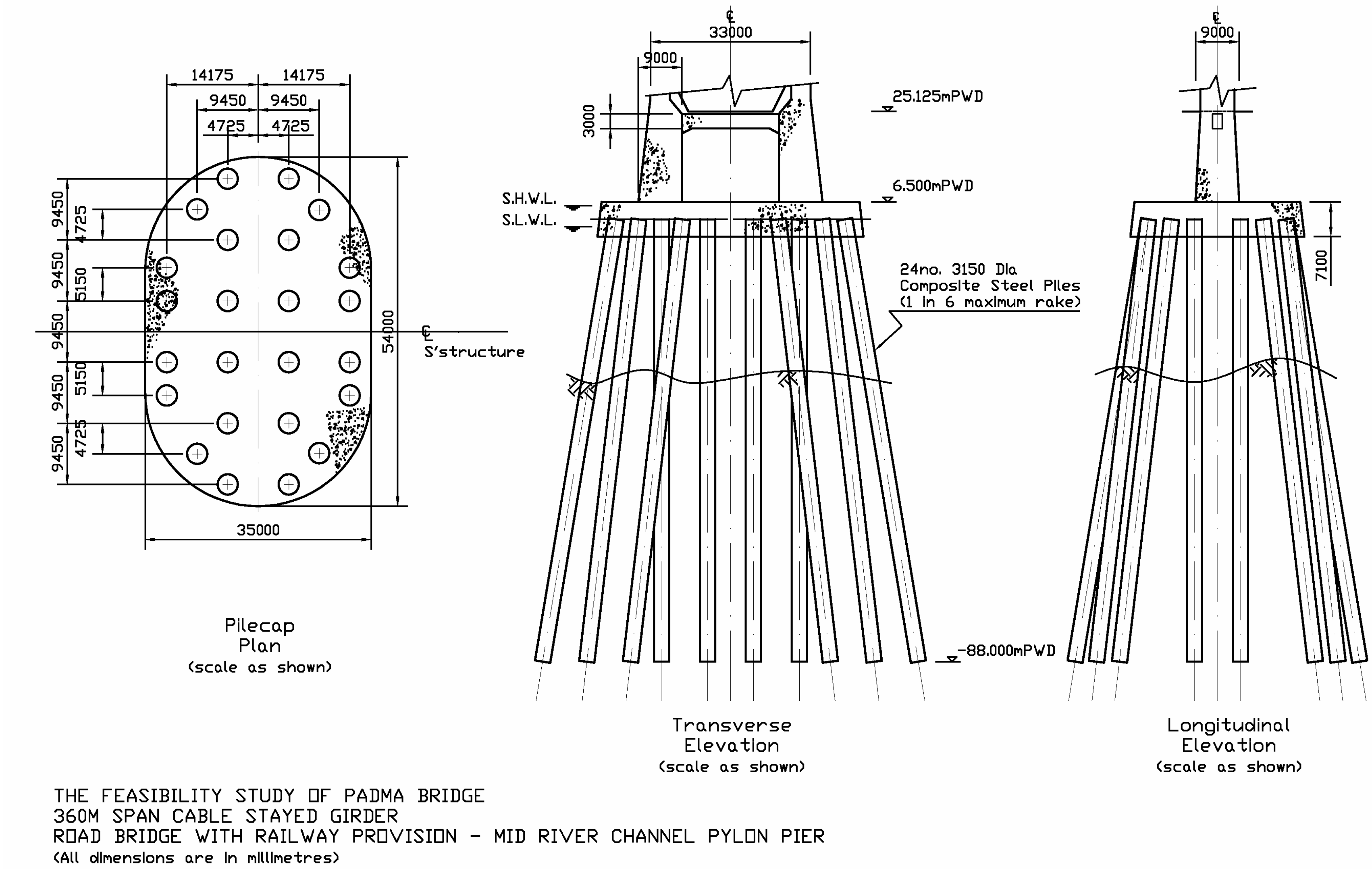


Figure 3.4.15 Mid-River Pylon Substructure General Arrangement (Alternative-HR2)

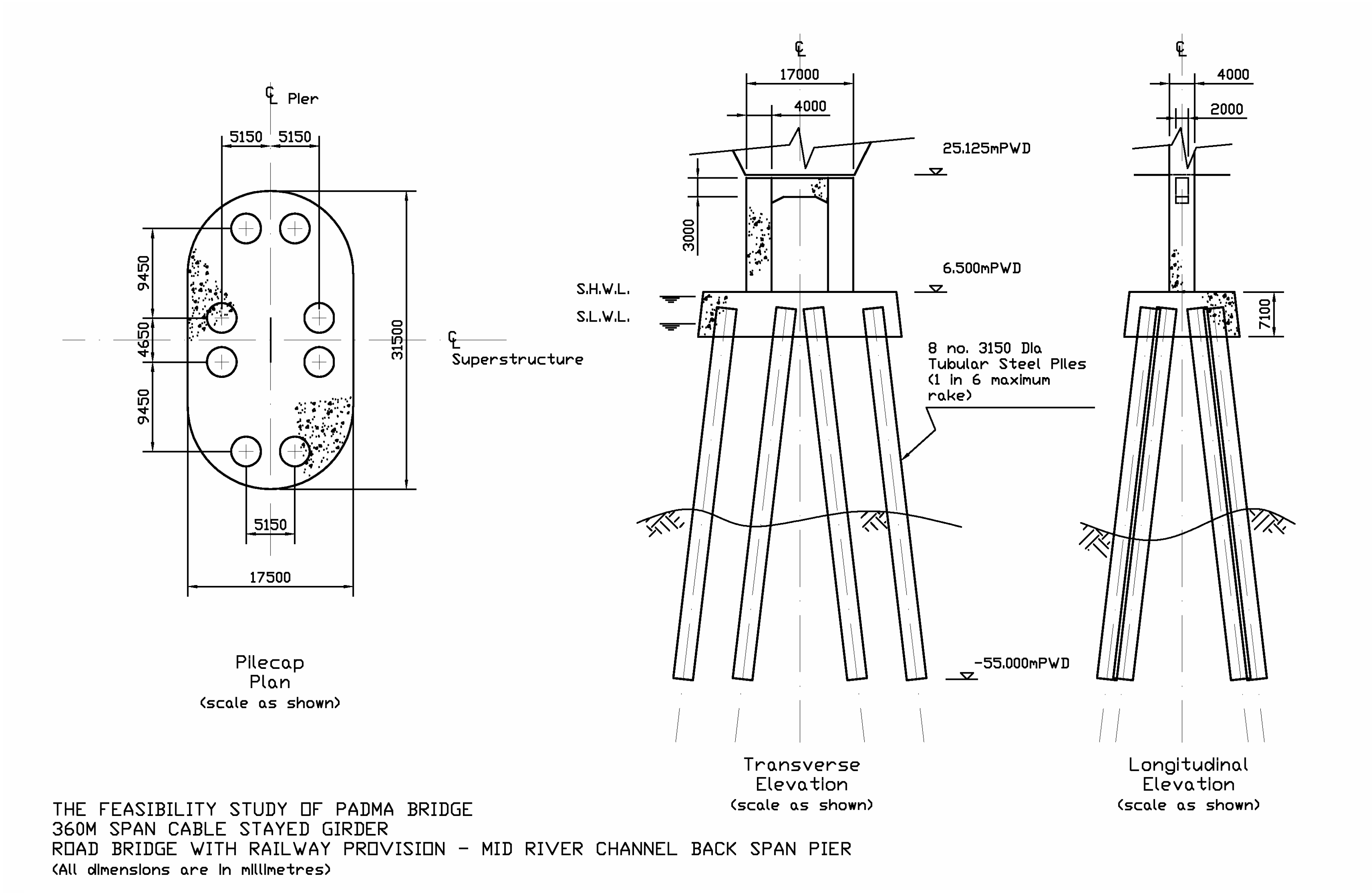


Figure 3.4.16 Mid-River Back Span Substructure General Arrangement (Alternative-HR2)

3.4.5 Alternative –H3: PC Extradosed Girder Bridge with Railway Provision (Minimum width)

(1) Superstructure

The Figure 3.4.17 gives the typical cross-sections for the Alternative-H3.

This is an alternative having almost same features as those of the Alterntative-1except the cross-section.

The alternative was added to the aforesaid four to show how much it would reduce the cost if the width would be decreased to 17.1 meters as specified by the RHD standard.

(2) Substructure

The Figures 3.4.18 and 3.4.19 show the general arrangements of the edge river and mid river substructures respectively. The reduced substructure is largely due to the reduction in superstructure self weight when compared to Alternative-H1.

In order to minimize the effect of river flow the pile caps are provided with rounded corners. The 3150mm diameter hollow steel tubular piles have a maximum rake of 1 in 6 to the vertical, and are to be in-filled with a mass concrete toe plug to generate additional end bearing.

The four edge river piers comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 10 number piles with average maximum wall thickness of 62mm. Average toe elevation is -87.5m PWD providing an approximate depth of embedment of 43m below the design river bed level.

The twenty-six mid river piers each comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number piles with average maximum wall thickness of 63mm. Average toe elevation is -78.0m PWD providing an approximate depth of embedment of 47m below design river bed level.

The twin pier columns forming the base of the extradosed cable towers are each 5.0 m x 6.0m in overall cross sectional area with minimum wall thickness of 1.25m, with similar connecting cross-beam 5.5m deep. This cross beam may be alternatively hidden within the superstructure.

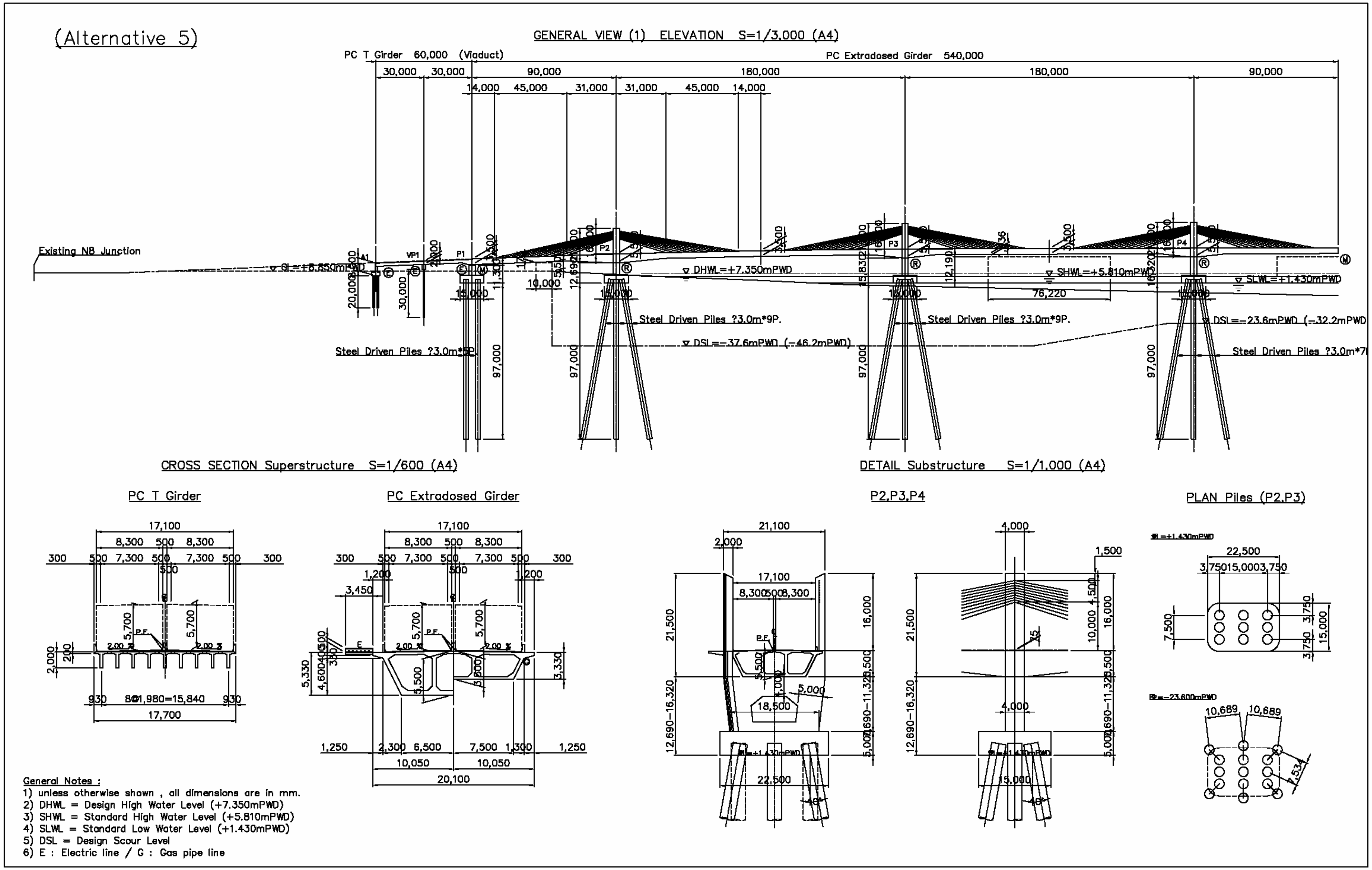


Figure 3.4.17 Typical Structure and Cross-section(Alternative-H3)

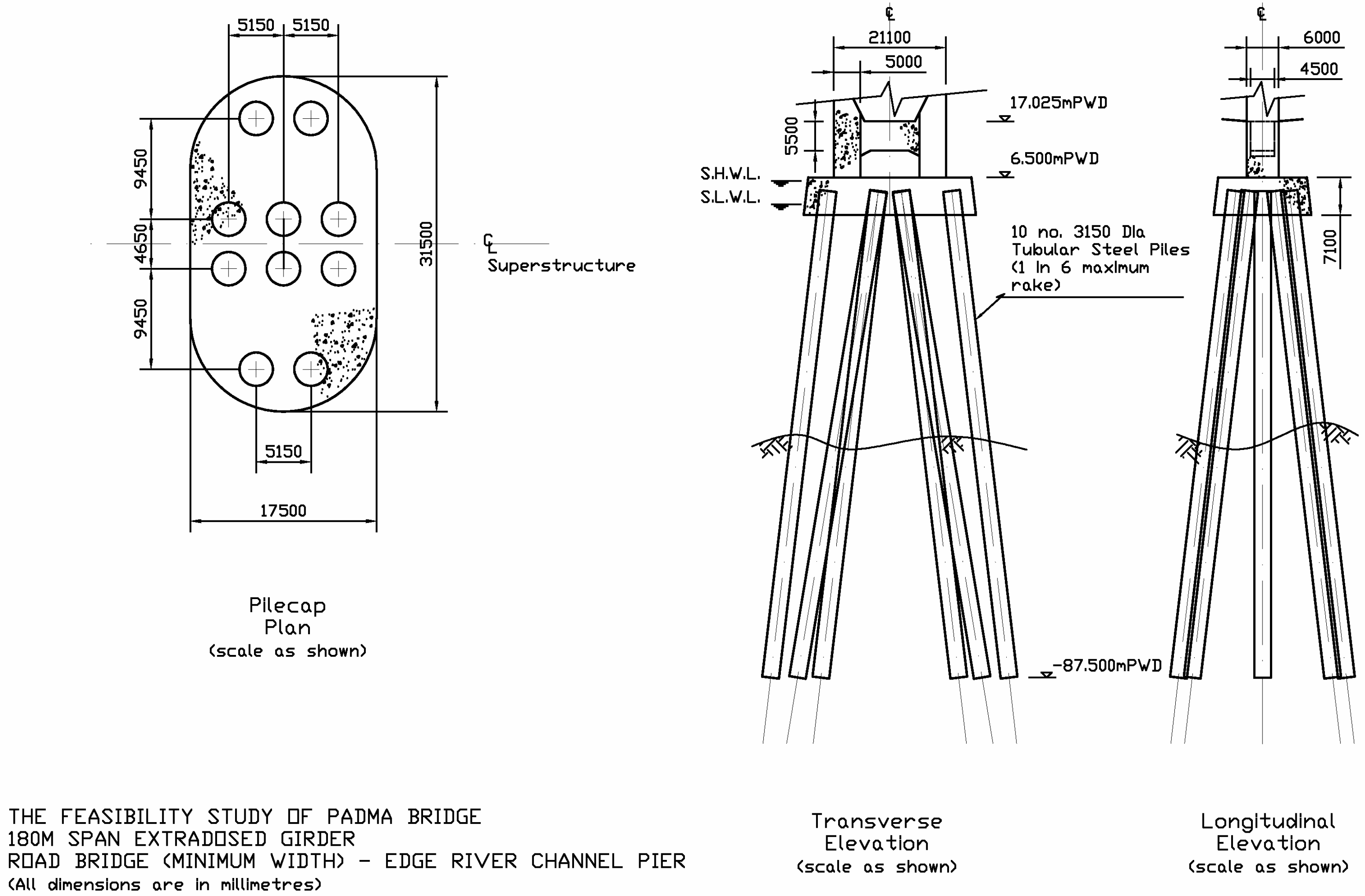


Figure 3.4.18 Edge River Substructure General Arrangement (Alternative-H3)

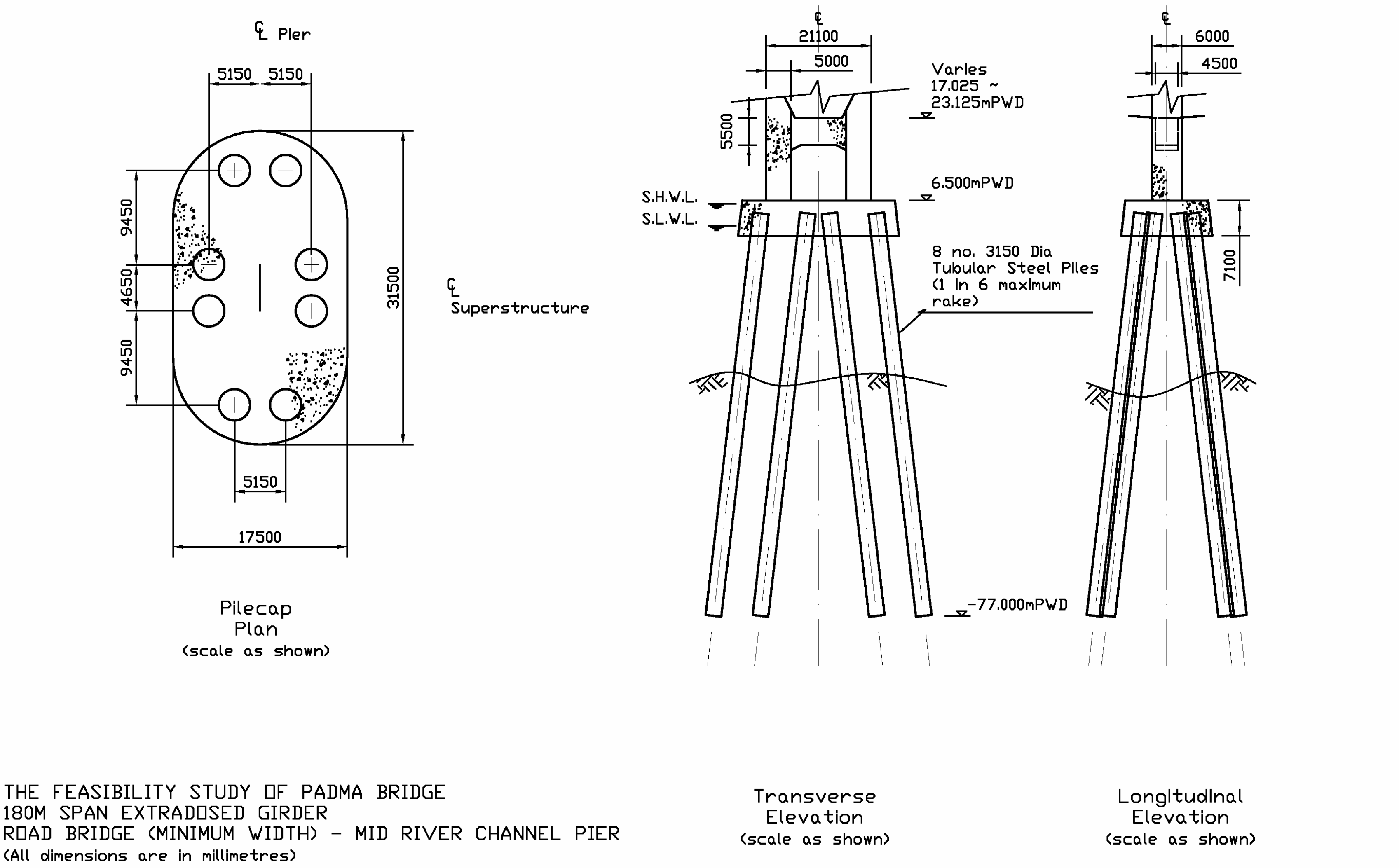


Figure 3.4.19 Mid-River Substructure General Arrangement (Alternative-H3)

3.5 SUMMARY OF QUANTITIES

Summary of quantities required for construction of the superstructures and substructures are shown in the Table 3.5.1 and 3.5.2 respectively.

Table 3.5.1 Summary of Quantities of Superstructure

Alternative –H1

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	137,490			1,912	139,402
	Tendon Weight	ton	5,754			68	5,822
	Rebar	ton	21,999			179	22,178
Pylon	Concrete Volume	m	12,000				12,000
	Rebar	ton	2,640				2,640
Stay Cable	Cable & Fixture Weight	ton	4,695				4,695

Alternative –H2

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	119,158	7,488		1,912	128,558
	Tendon Weight	ton	4,987	337		68	5,392
	Rebar	ton	19,066	1,213		179	20,458
Pylon	Concrete Volume	m	10,400	3,325			13,725
	Rebar	ton	2,288	632			2,920
Stay Cable	Cable & Fixture Weight	ton	4,069	802			4,871
Steel Girder	Steel Weight	ton		4,851			4,851

Alternative –HR

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	152,992		7128	1,912	162,032
	Tendon Weight	ton	6,177		250	68	6,495
	Rebar	ton	24,478		1069	179	25,726
Pylon	Concrete Volume	m	12,656				12,656
	Rebar	ton	2,783				2,783
Stay Cable	Cable & Fixture Weight	ton	6,006				6,006

Alternative –HR2

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	131,136	8558	7128	1,912	148,734
	Tendon Weight	ton	5,294	385	250	68	5,997
	Rebar	ton	20,981	1386	1069	179	23,615
Pylon	Concrete Volume	m	10,848	3800			14,648
	Rebar	ton	2,386	722			3,108
Stay Cable	Cable & Fixture Weight	ton	5,148	916			6,064
Steel Girder	Steel Weight	ton		6098			6,098

Alternative –H3

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	109,305			1,520	110,825
	Tendon Weight	ton	4,574			54	4,628
	Rebar	ton	17,489			142	17,632
Pylon	Concrete Volume	m	9,540				9,540
	Rebar	ton	2,099				2,099
Stay Cable	Cable & Fixture Weight	ton	3,733				3,733

Table 3.5.2 Summary of Quantities of Substructure

			Alternative-H1		Alternative-H2		Alternative-HR		Alternative-HR2		Alternative-H3	
		Unit	Main Bridge	Viaduct	Main Bridge	Viaduct	Main Bridge	Viaduct	Main Bridge	Viaduct	Main Bridge	Viaduct
Concrete Volume	Pier	m3	29,015	873	32,935	873	33,320	856	36,780	853	25,507	683
	Pilecap	m3	134,879	487	189,235	487	140,066	526	197,106	526	131,680	381
	Total	m3	163,894	1,360	222,170	1,360	173,386	1,378	233,886	1,378	157,187	1,064
	Σ	m3	165,254		223,530		174,764		235,264		158,251	
			100%		135%		106%		142%		96%	

Form	Pier	m2	45,248	1,315	46,220	1,315	49,033	1,341	49,659	1,341	36,192	1,047
	Pilecap	m2	32,936	221	38,292	221	32,936	235	38,292	235	32,238	176
	Total	m2	78,184	1,536	84,512	1,536	81,969	1,576	87,951	1,576	68,430	1,223
	Σ	m2	79,720		86,048		83,545		89,527		69,653	
			100%		108%		105%		112%		87%	

Rebar	Pier	t	6,757	162	7,387	162	8,026	156	8,936	156	4,843	125
	Pilecap	t	6,546	22	8,147	22	6,360	24	8,260	24	6,400	17
	Total	t	13,303	184	15,534	184	14,386	180	17,196	180	11,243	142
	Σ	t	13,487		15,718		14,566		17,376		11,385	
			100%		117%		108%		129%		84%	

Pile	Type		Steel Driven	Cast in Palce	Steel Driven	Cast in Palce	Steel Driven	Cast in Palce	Steel Driven	Cast in Palce	Steel Driven	Cast in Palce
	Dia	m	3.15	1.20	3.15	1.20	3.15	1.20	3.15	1.20	3.15	1.20
Vertical Pile	No.		-	56	16	56	-	64	16	64	-	47
	Length	m	-	1,379	1,280	1,379	-	1,576	1,440	1,576	-	1,140
Racked Pile	No.		256	-	288	-	308	-	332	-	248	-
	Length	m	21,856	-	23,488	-	25,866	-	27,238	-	19,914	-
	Σ of No.		256	56	304	56	308	64	348	64	248	47
	Σ of Len.	m	21,856	1,379	24,768	1,379	25,866	1,576	28,678	1,576	19,914	1,140
			100%		113%		118%		131%		91%	