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Design of the Third Karnaphuli Bridge

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ABSTRACT: The Third Karnaphuli Bridge is the first major cable-supported crossing to be constructed in Bangladesh. It spans the KarnaphuliRiver immediately to the north of Chittagong port. The main bridge includes three 200 m long main river spans with an extradosedprestressed concrete single cell box girder deck constructed by the balanced cantilever method.The bridge is founded on alluvial soil affected by deep soil liquefaction and river scour. The foundations of the main bridge incorporate 3 m diameter bored cast in situ piles which were base grouted post-construction. Construction of the bridge commenced in July 2007 and the crossing was opened to traffic in July 2010.

1 PROJECT INCEPTION

In November 2001, a report commissioned by the Netherlands' Ministry of Economic Affairs, examined several route options for a third crossing of the Karnaphuli River including one adjacent to the existing Shah Amanat Bridge in Chittagong. Following studies by the Government of Bangladesh (GoB) Roads and Highways Department (RHD) and its Consultants, it was decided to procure the new bridge on a design and build basis. In August 2005, tenders were invited from pre-qualified contractors for a bridge having a prestressed concrete extradosed box girder deck with minimum main span lengths of 200 m. In July 2006, a US$50 million contract for the design and construction of the entire project was awarded to MBEC-ACL-COPRI JV, a joint venture between Major Bridge Engineering Co. Ltd (China), Al-Amin Construction Co. Ltd (Bangladesh) and Copri Construction Enterprises W.L.L (Kuwait), with High-Point Rendel Ltd (HPR) (UK) designer for the bridge. The project was jointly funded by Kuwait Fund for Arab Economic Development (KFAED) and the GoB, with the RHD as main client. Benaim Ltd (UK) was the checker of the definitive design and KEI-BCL-TAEP-STUP JV were the supervising consultants

2 SCHEME DESCRIPTION

2.1 *Location*

The Third Karnaphuli Bridge scheme comprises a 950 m long bridge, approach embankments, river training works and a toll plaza. The bridge (Fig. 1 - mid point coordinates 22º19'33.59"N and 91º51'11.33"E), is located 50 m upstream of the existing Shah Amanat Bridge. It has been designed with navigation spans which will allow passage of vessels to reach the planned expansion of port facilities upstream.

2.2 *Principal Dimensions*

The whole new bridge crossing consists of a 120 m long approach viaduct section (subdivided into 20 m long spans), three 200 m long main river spans and two 115 m long side spans (Fig. 2 & 3). The bridge is aligned on a 3200 m radius horizontal curve. The dual carriageway bridge carries four traffic lanes of 3.65 m width, two 1.65 m wide lanes for slow-moving vehicles and two footways. Services run in ducts located in the footways and in the bridge central reservation. The overall deck width, including the central stay cable tower and footways, is 24.47 m. The main bridge deck was constructed in situ by the balanced cantilever method using pairs of travelling forms.

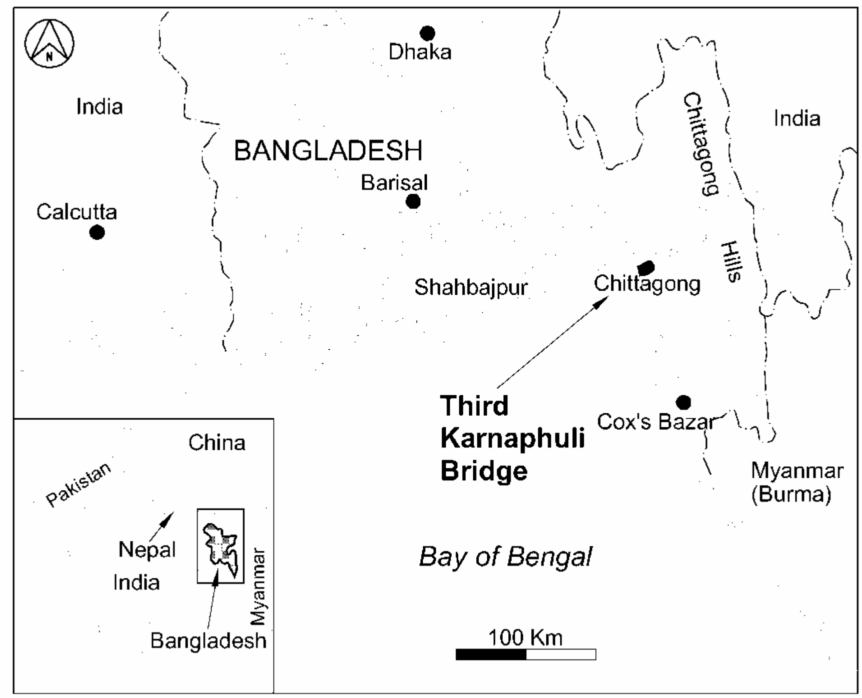

Figure 1.Location Plan

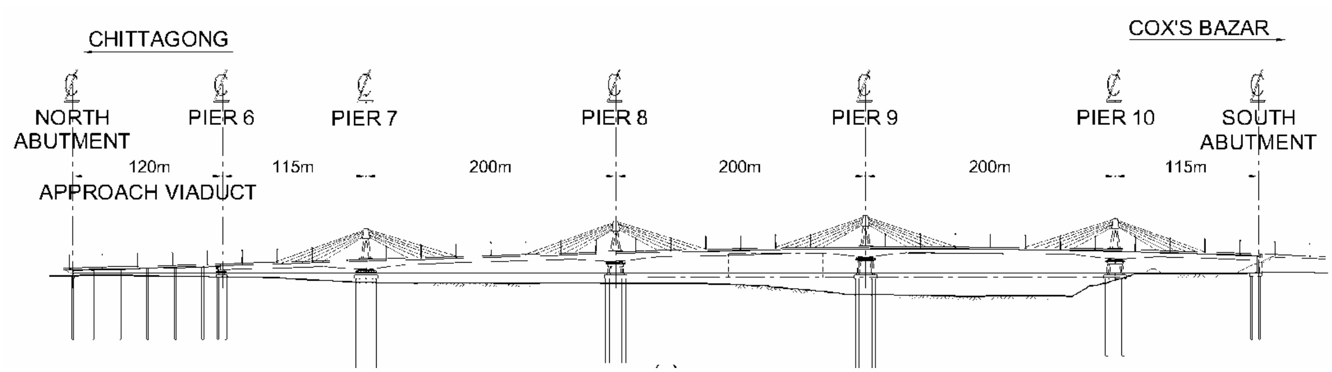

Figure 2.Elevation of Bridge

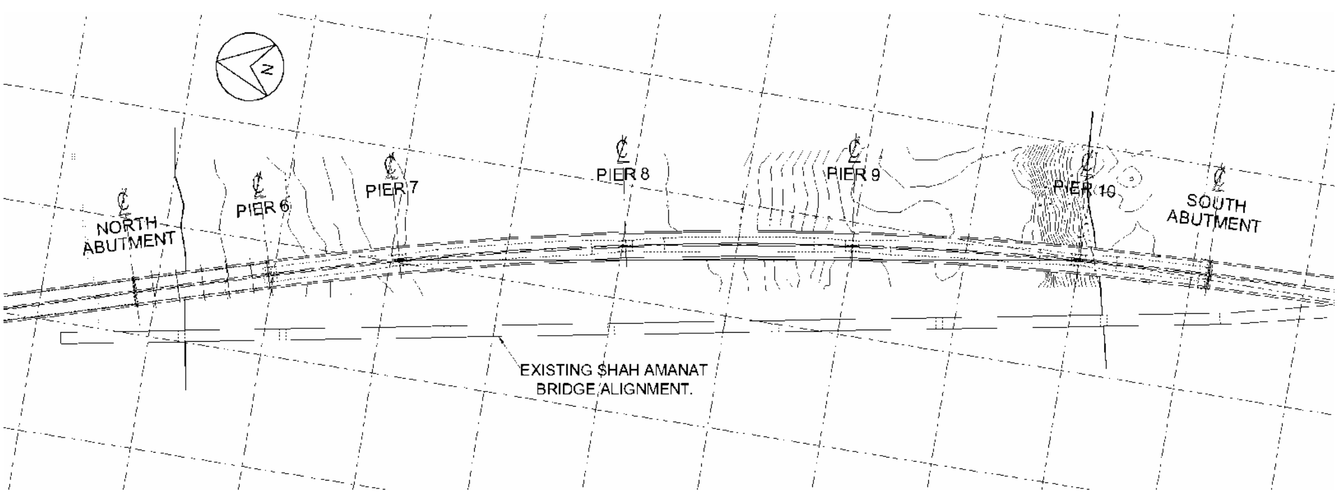

Figure 3.Plan of Bridge

2.3 *Substructure*

Each of the four main river piers (i.e. Piers 7 to 10) is founded on four 3 m diameter vertical piles (in square configuration) of maximum 77.0 m length (from base of pile cap). The six approach viaduct piers and the north abutment are each supported by a row of four parallel 1.5 m diameter vertical piles of maximum 52.4 m length (from ground level). The south abutment is founded on two parallel lines of six 1.5 m diameter vertical piles of maximum 54.0 m length. All piles are reinforced concrete bored (under bentonite slurry) cast in situ and are installed with sonic logging tubes. The piles are founded in alluvial strata partially affected by maximum 30 m deep soil liquefaction and river scour. The toe of the main pier piles was base grouted post-installation to improve their performance and for quality assurance.

3 SITE INFORMATION AND DESIGN CODES

3.1 *Site*

The site lies in the flat coastal region of Chittagong at the foot of the Chittagong Hills. The KarnaphuliRiver drains part of this hill system and deposits alluvial and deltaic strata in the adjoining flat plain. At the bridge location, the KarnaphuliRiver generally flows in a main channel approximately 280 m wide. It extends the 950 m width of the flood channel, between the riverbanks on each side, during the annual monsoon season. At the project site, the river is tidal and fluctuates in level by up to 5.9 m.

3.2 *Design Standards*

In accordance with the RHD's requirements, the tender and definitive design of the permanent works for the entire bridge was carried out following AASHTO LRFD (2004) and British Standard BS 5400 (1990) as described below. The standards for the construction materials, ground investigations and in situ and laboratory testing were AASHTO and ASTM. Other compatible codes and technical literature were adopted for specific design issues not covered by the main standards. The entire bridge was designed following the strength and serviceability limit state approach.According to the Bangladesh National Building Code (1993), the new bridge site lies within seismic Zone-2, corresponding to a 100 years return ground surface acceleration of 0.05g. The design seismic zone coefficient (Z) for the area is 0.15 for horizontal excitation and 0.7 x Z for vertical excitation. As the bridge will become an important link in the Bangladeshi national highway network, it is classified as an "essential" structure as defined in AASHTO LRFD (2004) and consequently had to be designed accordingly in terms of strength and serviceability.

4 SUBSTRUCTURE GEOTECHNICAL DESIGN

4.1 *General*

The design of the piled foundations was carried out according to AASHTO LRFD (2004). This standard permits the use of both empirical and "rational" design formulae (i.e. formulae adopting strength and stiffness values as input data) to calculate vertical pile capacity. Empirical formulae are often based on "local" experience which depends, amongst other factors, on the specific ground conditions, the method of pile construction, and pile length and diameter. Despite the considerable amount of reliable SPT data available for the site, the use of empirical design formulae was not deemed to be appropriate given the peculiarity of the local geological sequence and the unusual large diameter of the main pier piles. Instead the design of the piles for vertical capacity was carried out using "rational" formulae and the geotechnical properties input for the soil were derived from the laboratory test results and site specific correlations with the SPT data.

4.2 *Base Grouting*

The ground investigations carried out during the initial phase of the design had revealed that the site ground conditions are vertically and laterally variable, and partly unforeseeable. There was a risk that the working piles could be founded on compressible strata with poor resistance. The preliminary calculations indicated that the 3 m diameter main pier working piles, due to their large diameter, would provide roughly 38% of their resistance in end-bearing. The contribution of the shaft to pile resistance, despite the "considerable" pile length, was proportionally less due to the potential for deep scour and soil liquefaction. In order to guarantee the full design performance of the piles in end-bearing it was decided to carry out post-installation compaction base grouting. This involved the slow injection at high pressure of a viscous grout mix to "compact" the strata and debris (resulting from the pile shaft excavation) located below and at the pile toe, thus mobilising the full pile end bearing at a smaller displacement. This is a critical aspect when using large diameter piles. The peak pile end-bearing resistance may be assumed to be mobilised at 10% of the pile diameter (i.e. 300 mm here), at which displacement shear will already have occurred along the shaft, thus providing very limited residual resistance, and the pile settlement will also already be excessive. Base grouting will also restore the original in situ strength of the soil which was expected to be affected by the anticipated lengthy excavation under bentonite slurry of the deep, large diameter pile shafts. A particular arrangement of six U-shaped “tubes-à-manchette” cast into the toe of the pile was adopted to base grout a substantial proportion of the large pile footprint. The effectiveness of the base grouting was verified by comparing the results of three base grouted test piles to the un-grouted test pile at Pier 3. The compaction grouting was found to be very effective inrestoring and indeed improving the original in situ soil resistance. The pile testing also showed that some of the grout injected at the toe of the piles had randomly migrated upwards from the pile toe along the shaft interface often substantially improving the frictional resistance of the bottom 3-4 m of the piles. The advantages of using compaction grouting were also confirmed by comparing the sonic logging integrity test results for the non-base-grouted test pile at Pier 3 to the other three base-grouted test piles. The non-base-grouted pile showed a “soft” pile toe compared to the “sound” pile toes of the other three piles. This was despite the base-grouted piles being substantially deeper and therefore comparatively more likely to suffer this problem due to greater elastic expansion of clay and granular strata, plastic expansion of expansive clayey strata due to re-hydration and the accumulation of a greater thickness of debris at the base prior to concrete placement.

4.3 *Pile Testing*

The definitive design of the main bridge piers incorporated four 3 m diameter bored cast in situ vertical piles at each pier, with length ranging from 62.0 m to 77.0 m (from base of pile cap), and able to withstand service loads of between 45 MN and 53 MN. The two abutments and the approach viaduct were founded on a total of thirty-six 1.5 m diameter bored cast in situ vertical piles of maximum length of 54.0 m (from ground level) able to withstand service loads of up to 4.2 MN.The preliminary pile design was validated and optimised by comparison with the data derived from four vertical bored cast in situ reinforced concrete test piles. These piles were located in very close proximity to three of the four main bridge piers (Pier 7, 8 and 10) and one close to the approach viaduct piles at Pier 3. The test piles were of smaller diameter (i.e. 1.5 m) compared to the main pier working piles and ranged in length from 52.3 m to 74.6 m (from the base of pile cap). For technical, practical and health and safety reasons, the pile testing was carried out using a self balancing hydraulic jack-up system (similar to the Osterberg-cell®).The test piles were fully instrumented with strain gauges and other monitoring apparatus in order to obtain a breakdown of the pile resistance and displacement along the various sections of the pile shaft and in end-bearing. The pile testing regime gave very clear and consistent estimates of the resistance of the soil, which were then correlated with the laboratory geotechnical and SPT data.

4.4 *Trial Pile*

Given the unusually large diameter of the piles, the pile length and the site logistics it was considered necessary to construct a full size trial pile which was located close to Pier 7. This was similar in construction to the anticipated working piles, including steel reinforcement cages, sonic logging tubes and base grouting. The pile was not load-tested both because of the practical difficulties (given its capacity) and due to the very satisfactory and consistent results already obtained for the test piles. The construction of the trial pile proved to be extremely valuable as it provided considerable information about the constructability aspects including drilling of the shafts, cleaning of the shaft base, concreting operations, base grouting and construction timing.

5 SUBSTRUCTURE STRUCTURAL DESIGN

5.1 *Design principles*

AASHTO LRFD (2004) requires that bridge foundations are designed using limit state principles to resist service loads, ultimate (factored service) loads and extreme loads (e.g. seismic and ship impact). The working piles were designed to transfer into the ground the vertical and horizontal loads arising from self weight of the structure, soil downdrag on piles, construction effects e.g. out of balance loads from cantilever construction, live loading, wind, ship impact hydrodynamic flow and seismic action. As experienced on other long span concrete bridges in Bangladesh and India (e.g. Castelli& Wilkins 2004, Farooq& Barr 1999), the design of the bridge substructure is strongly conditioned by seismic effects. These are both responsible for additional vertical loading and often trigger the liquefaction of recently deposited and saturated sandy strata. At this particular site, soil liquefaction, river scour, and river dredging, may locally affect the upper 30 m embedded length of piles. This soil cannot therefore be relied upon to permanently provide full lateral and vertical resistance. Piles consequently had to be proportioned with due regard to the effects of slenderness, deck displacement and pier fixity.

5.2 *Appearance*

Above pile level, the shape and appearance of the main piers was determined mainly by structural requirements. At Piers 8, 9 and 10, the four inclined circular columns provide a direct transfer of load from the bridge deck bearings into the piles. At Pier 7, where there was insufficient clearance between deck and pile cap to fit columns, a solid plinth had to be used.

5.3 *Shock Transmission Units*

The bridge was fitted with shock transmission units (STU) at each main pier to lock the deck to the substructure under seismic loading. These permit the horizontal seismic inertial forces to be resisted by shear and bending in the piles of all piers. Combinations and permutations of scoured and un-scoured piers and piers partially founded in liquefied and non-liquefied soil under seismic loading had to be considered to find the maximum effect at any location. Using this approach it was identified that, for instance, Pier 7 in its un-scoured state, was much stiffer than the adjacent piers when they were scoured, therefore attracting substantial seismic loads to it. For this reason Pier 7 piles were more heavily reinforced than those at other main piers.

5.4 *Computer Analysis*

In order to accurately predict the seismic effects, the entire structural system, including deck, towers, cables, piers and piles was modelled as a whole using four-dimensional static and dynamic design and analysis software RM2006®, and a seismic response spectrum analysis carried out. The soil/structure interaction was modelled using non-linear lateral and axial soil springs (derived from p-y and t-z curves respectively) attached to the base and along the length of the pile elements.The design of the main bridge piers was carried out using envelopes of load effects from the main bridge computer analysis. The pier tabletops were additionally designed for local forces from the shock transmission unit anchorage brackets and the bridge bearings.The seismic design was in accordance with the provisions of AASHTO LRFD (2004) allowing for development of plastic hinging in some members.

6 EXTRADOSED BRIDGE AND CABLE DESIGN

6.1 *General Description*

Extradosed bridge decks gain their distinctive character and appearance from the use of a stay cable support member with height (above deck level) of between 8 % and 12 % of the deck span length (e.g. Kasuga 2006). Using this approach, the tower tops of the bridge were set at 25 m above deck level to obtain the maximum benefit from the stay cables whilst keeping the towers within reasonable slenderness limits. The stay cables are continuous between deck anchorages, which are located at distances of between 36 m and 76 m from the piers (Fig. 4). The steel deviator saddles in the towers (through which the cables pass) are located at heights of between 19 m and 24 m above deck level.

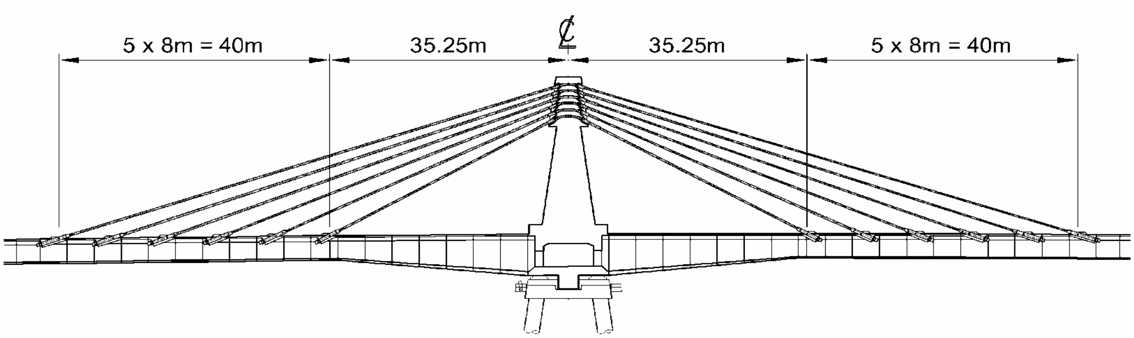

Figure 4.Layout of Stay Cables

6.2 *Allowable Stress in Stay Cables*

AASHTO LRFD (2004) covers some aspects of stay cable design such as wind/vibration, catenary behaviour, and cable removal/replacement, but it does not deal with the limitations on permissible stresses in extradosed bridge cables.In a conventional cable-stayed bridge, the cable support system is stiffer than the deck support system therefore a greater proportion of vertical live load is carried by the cables compared with an extradosed bridge design. Because of their greater unsupported length, cables are also generally subjected to a larger fluctuation of stress due to wind. Due to their exposure to fatigue loading, the stresses in the stays of a cable-stayed bridge are usually limited to approximately 45 % of the theoretical breaking strength (e.g. BS EN 1993-1-11 2006). However for extradosed bridges, some designers permit use of higher stresses in recognition of the less onerous fatigue loading (Kasuga 2006).For the ThirdKarnaphuliBridge, the stay cables were jacked at the time of installation to 45 % of breaking strength. At the end of construction the maximum stress in the stay cables, due to force redistribution effects, varied between 42 % and 63 % of breaking strength depending on the location. The calculated stress variation in the cables due to live load and wind fluctuations was 25 MPa. The stay cable system was fatigue tested using the acceptance test of the Post-Tensioning Institute (1998). This demonstrated that there were no wire breakages after two million cycles of 159 MPa, corresponding to a range of between 35 % and 45 % of breaking strength. These test results were extrapolated for higher stresses using the Goodman relationship which proved that the cables could sustain up to 100 MPa stress range at 60 % of breaking strength.

6.3 *Cable forces*

Unlike cable-stayed bridge decks, extradosed decks do not require cable “tuning” at the end of construction when forces are adjusted to match the pre-determined values assumed in design. In extradosed decks the final cable forces are dependent on various factors including the accuracy of the initial stressing operations, variation in the deck/cable stiffness ratio and the actual weight of the deck. These changes and uncertainties in the cable force during construction had to be taken into account in the design by use of partial load factors.

6.4 *Durability*

The stay cables were designed to be robust and durable in order to minimise maintenance. For this reason, the strands were individually galvanised, greased and sheathed for corrosion protection. In order to facilitate possible cable replacement, the cable system was designed to permit re-stressing from within the deck void, and complete replacement (during the lifetime of the bridge) of the whole cable or individual strands.

7 SUPERSTRUCTURE DESIGN

7.1 *Design principles*

The main bridge consists of a prestressed concrete box girder deck supported by four vertical load bearings at each of the main piers (Fig. 5). This configuration allows some deck moment to be carried over into the substructure under unequal span live loading thereby reducing the midspan moment variation. Pier 6 and the south abutment, which are at the two ends of the main bridge, are provided with two bearings each. The deck is prestressed longitudinally by internal bonded prestressing tendons, either twenty-seven 15.7 mm diameter strands or twelve 15.7 mm diameter strands, passing through the top and bottom slabs. Additional support is provided by six stay cables, comprising ninety-one 15.7 mm diameter strands, passing over reinforced concrete towers which are integral with the deck. The stay cables, which are anchored at the centreline of the deck, act like external prestressing enhancing the vertical shear capacity close to the piers and resisting deck flexure.

7.2 *Tower Saddles*

Multi-pipe steel deviator saddles are used in the tower tops. This type of saddle was preferred to a single pipe deviator tube as it provides a better distribution of stress into the towers and facilitates the replacement of individual strands if needed. A fixed “blocking” system was used to prevent slippage at the saddle due to unequal forces in the stay cables under live loading. Dampers are provided at cable anchorages to minimise fatigue effects due to wind vibration.

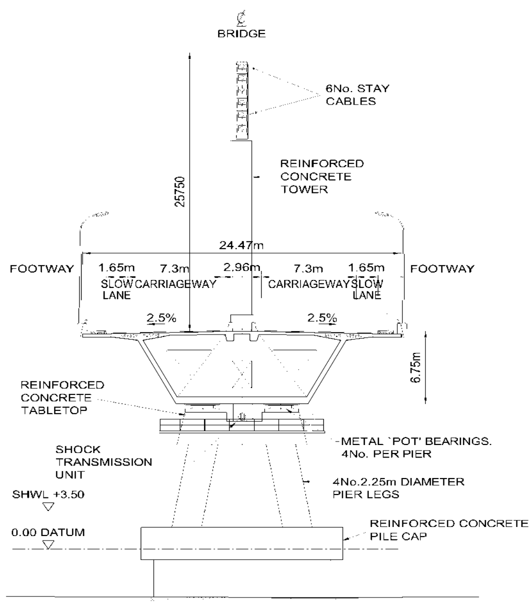

Figure 5.Deck Section At Pier

7.3 *Deck Cross Section*

The deck is braced against transverse distortion by diagonal reinforced concrete struts (Fig. 6). These provide a "load path" to transfer the vertical component of stay cable forces into the main webs and they also reduce the effective transverse span of the top slab for carrying local traffic loading. In the stayed deck segments the struts are prestressed using fifteen 15.7 mm diameter strands to resist the axial tensile force. The top slab is transversely prestressed over the stayed deck section using four 15.7 mm diameter strands in flat ducts at 450 mm centres. This prestressing is necessary to resist tensile in-plane stresses associated with spread of the central stay cable anchorage force outwards to the webs.Internal diaphragms are provided in the box girder deck at the piers (above each pair of bearings) and also at the end of the haunched section where the bottom compression flange needs to be stabilised (and the compressive stress is at a maximum). At all sections, other than those adjacent to the diaphragms, the permissible compressive stress in the bottom flange had to be reduced to take account of the flange slenderness.

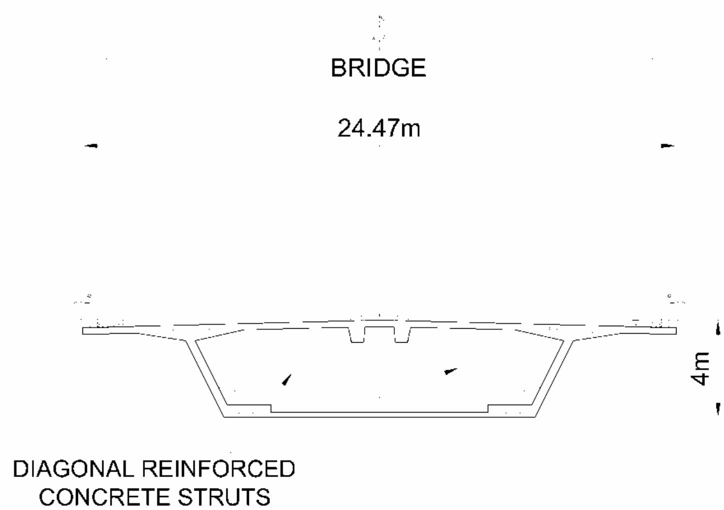

Figure 6.Deck Cross Section at Midspan

7.4 *Computer modelling*

The global design of the main bridge deck was required to comply with AASHTO LRFD (2004). This standard indicates that at ServiceLimitState, fibre stresses shall remain within permissible limits and at StrengthLimitState the sections shall have adequate shear, torsional and flexural strength. These design requirements were automatically verified using software RM2006®. However local elements, such as the prestressing and the stay cable anchorages, required specific design procedures.Every stage of construction (e.g. traveller movement, segment casting, stressing etc.) was modelled, including the effects of creep and shrinkage, to provide an envelope of design load effects. Since single-stage stressing was specified for the stay cables, the deck profile at each stage of segment casting needed to be accurately predicted from the time-dependent analysis. This was required to ensure that the final stresses in the stay cables and deck remained within permissible limits as the deck profile changed during later phases of construction.Three types of transverse sections were considered in the design: pier and abutment diaphragms at bearings, non-stayed deck sections, and stayed deck sections. The design of diaphragms was by use of "strut and tie" idealisations and conventional shear/torsion codified design methods. The pier diaphragms were particularly complex (Fig. 7) since tower forces were transferred into the bearings through large transfer beams which connect to the vertical diaphragm walls. Finite element modelling was carried out to understand the flow of forces.The transverse deck sections were modelled in three dimensions using engineering analysis software for bridges LUSAS® to check the effects of the cantilever construction sequence, particularly with regard to strut construction and "locked-in" effects.

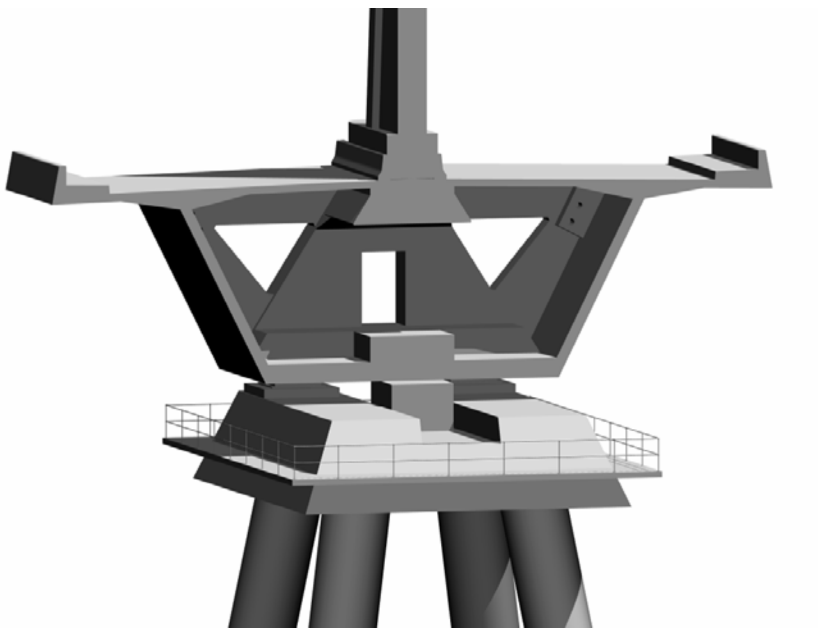

Figure 7.Pier Diaphragm Detail

8 CONCLUSION

Design and construction of the Third Karnaphuli Bridge has been completed on time and on budget as a result of the excellent cooperation between the Contractor, Designer, independent Design Checker, Client and the Supervision Consultants.Progress to successful completion of this Design and Build project has benefited from having a viable tender design concept which has changed very little during construction. The considerable investment made by the Contractor in additional ground investigations and pile testing has enabled the development of a safe and efficient substructure design.The concrete extradosed design has been shown to be an economical solution for bridging the Karnaphuliriver.

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