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Investigation on Improvement of Low Cost NERD Slab System

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Abstract: The NERD center floor slab system was introduced by late Dr. A.N.S. Kulasinghe in early 1987 specially for domestic buildings which are used by middle income families in Sri Lanka. The invention of NERD system directed to identify cost effective slab construction system relative to conventional slab system and also the use of un-propped construction technique and reduction of depth of slab caused to reduce construction time and material required for construction activities. The NERD system consists with 50 mm thick in-situ concrete slab retain on trapezoidal shape pre-stressed beams which are placed by keeping 600 mm interval between each. Although, the concept of NERD system is being widely adapted in domestic building construction exposed beam under the soffit of the slab keeps away people from the use of NERD slab system. Therefore, this research has been given much more advertency to make it as flat soffit slab with the improvement of structural arrangement of the NERD system such a way that changing the shape of pre-stressed beam and thickness of in-situ concrete slab with hollow arrangement to reduce utilized concrete of the slab.

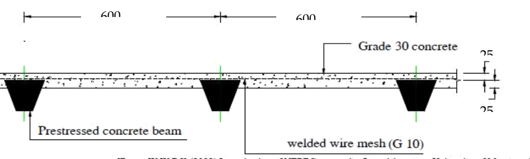
Keywords: Cost, Deflection, Flat soffit, Pre-stressed, Strength

1. Introduction

The use of concrete in structures goes back to period of ancient time. Due to the requirement of people who are using concrete as a construction material, the technology related to concrete has been updating day by day. For instance the precast concrete technology, composite slab system, light weight concrete etc. were introduced based on results of congenial experimental and theoretical investigation to modern construction industry with the use of concrete as a primary construction material.

However, the selections of any type of construction materials depend on many factors; primitive strength, durability, cost for materials etc. But nowadays it is being given much more attention to the reduction of cost and time related to construction practices as vital thing in most circumstances. Therefore, many researchers are trying to introduce cost effective techniques with less construction time period to modern construction industry. As a result of such aresearch, the NERD center floor slab system was introduced by late Dr. A.N.S. Kulasinghe in early 1987. Nowadays the slab system is being widely used in Sri Lanka in the domestic building constructions due to the many advantages over the conventional floor slab system; primitive 30 - 40% cost effectiveness, considerable reduction in weight of the slab, no propping is required for the soffit of slab, saves materials, labour and time, workable bottom space immediately after the concreting etc.

Achieving above advantages have been functioned by changing conventional floor slab system structurally based on supportive conditions of the slab, thickness of the slab, type of reinforcement use in construction practices etc. The NERD slab system consists 50 mm thick in-situ topping retained on trapezoidal pre-stressed concrete beams in 600 mm intervals as shown in Figure 1. While using grade 30 concrete for in-situ topping by placing the gauge 10, 50 mm x 50 mm welded G.I mesh as the reinforcement for slab at the center of slab, grade 40 concrete is being used to cast the pre-stressed beam which contains three numbers of 5 mm high tensile steel wires tensioned to 20 kN each by considering 30% of applied pre-stressing force as effective pre-stressing force.



[Perera W.W.P.K.(2009) Investigation of NERDC composite floor slab system. University of Moratuwa.]

Figure 1: NERD composite slab system

But, generalization of the new system caused to identify required modification for the NERD system. Since it had been identified that use of plywood board to support the fresh concrete in the 50 mm thick slab panel can be replaced by 12.5 mm thick Ferro cement panel as indicated in the Figure 2 below based on proper theoretical and experimental investigation [1].

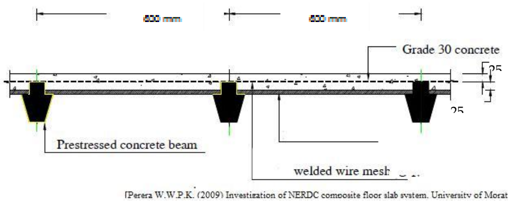


Figure 2: Modified NERD composite slab system

Even though, it has been introduced some modifications on the current NERD system there are some problems with the appearance of the soffit of the slab system. As indicated in Figure 1 and 2, it can be seen exposed beams under the soffit of the slab that create less customer attraction due to inappropriate aesthetic appearance on the soffit of the slab. Therefore, it is vital importance to introduce new system with the flat soffit arrangement for the current NERD system while maintaining 30 % - 40 % cost effectiveness compared with the conventional slab system.

2. Method

2.1. Theoretical analysis

Structural arrangement of current system was changed both in pre-stressed beam and in-situ concrete slab in some extend to design flat soffit arrangement for the current NERD slab. Therefore, the shape of the pre-stressed beam was changed from trapezoidal shape to inverted T beam section while introducing hollow core slab arrangement instead of the 50 mm thick in-situ floor slab in current system. The use of hollow core system guides to improve the fire resistance with excellent deflection and vibration characteristics [2].

However, the anticipated theoretical basis for invention of any kind of new structure is really vital importance and required. Therefore, the theoretical analysis for both pre-stressed beam and composite hollow slab system were done based on recommendation given in BS standards ([[3], [4], [5], [6]). The sectional properties of pre-stressed beam were checked for serviceability limit state and transfer state under the applied dead and imposed load relative to un-propped construction technique. Accordingly the way of beam casting in NERD system, the pre-stressed beams are not being subjected to dead and imposed load during the transfer state. But in the serviceability limit state they are being subjected to the dead load of slab and beam and imposed load. Therefore, modification on the equations of the pre-stressed beam design were done to derive equation given below which are relevant to the NERD system.

$$\frac{M_d}{Z1} + \frac{M_{imax}}{Z1_{comp}} \le \alpha f_{a,maxt} - f_{a,min} \quad (1)$$

$$\frac{M_d}{Z2} + \frac{M_{imax}}{Z2_{comp}} \le f_{a,max} - \alpha f_{a,mint} \quad (2)$$

Where,

Pi - Initial jacking force
Pj - Effective jacking force
e - Eccentricity of tendons
M_d &M_i - Dead load and imposed on the beam
$Z_{1\&2}$ - Bottom and top sectional properties of beam section
$Z_{1\&2,\ comp}$ - Bottom and top sectional properties of composite section

In the design of in-situ concrete slab, it was realized that the increment of slab thickness is essential to achieve the requirement of the flat soffit arrangement. But, the increment of the slab thickness directly adds additional construction cost for slab system. Therefore, the voided arrangement with cuboids was used to reduce the utilized concrete for the slab.

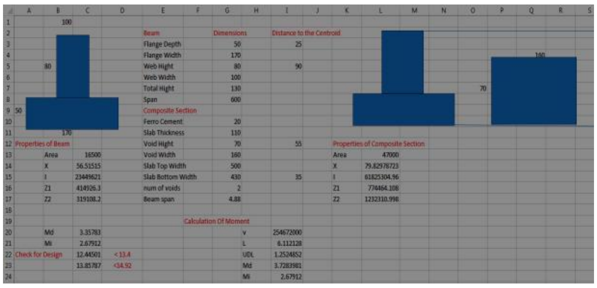


Figure 3: Excel sheet analysis for composite slab

Accordingly above modified equations and the recommendations given in BS 8110, theoretical analysis for composite slab was carried out to identify the beam and slab section for the most economical flat soffit slab arrangement [7]. For the easiness of the analysis it was prepared excel sheet analysis to identify the most suitable beam dimensions, shape and void sizes for different slab thicknesses as shown in the Figure 3. In addition to those variables, the beam span, interval between two beams and types of reinforcement used in slab concrete were considered to optimize existing NERD slab system.

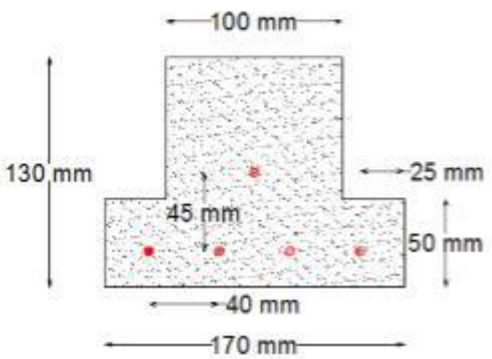


Figure 4: Pre-Stressed Beam

Based on the results of the above analysis, the dimensions and shape of the pre-stressed beam were selected as shown in Figure 4. Suitable eccentricity for tendons and jacking force in the wires in the inverted T beam section were identified based on the result of drawn magnel diagram as shown in Figure 5 [8]. Accordingly, the results of the magnel diagram the eccentricity for the tendon and effective pre-stressed force in the tendons were taken as 29 mm and 72 KN respectively. The tendons in pre-stressed beam were arranged as two layer by keeping 45 mm space between each layer while maintaining 40 mm interval between each tendons in bottom layer as shown in Figure 4.

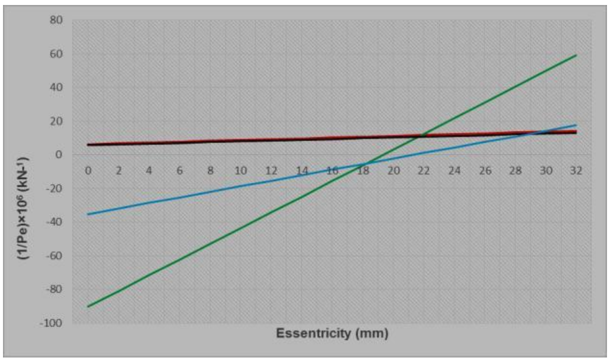


Figure 5: Magnel diagram

Accordingly, results obtained from the Theoretical analysis depth of in-situ concrete slab was taken as 110 mm while placing 20 mm Ferro cement panel to support the fresh concrete in the slab as shown in Figure 6. The sizes of cuboid void placed in the slab panel was maintained as 160 mm × 150 mm × 70 mm while maintaining 40 mm space between each cuboid to place the 6 mm diameter mild steel with the 190 mm space between each bar as the reinforcement for the slab. Improving of composite action between beam and slab was functioned by placing shear links at 225 mm interval along the span of the pre-stressed beam [9].

Table 1: Variation of beam interval with beam span

Beam Span (m)	Beam Interval (mm)	Number of Cuboids: Along the span Direction	Number of Cuboids: Perpendicular to the span direction
≤ 3.05	1200	16	5
3.66	1000	19	4
4.27	800	22	3
4.88	600	25	2

The design of improved slab system was done for the four beam spans as shown in the Table 1 while changing the interval between two beams instead of changing depth of the beam as in current system. And, the number of cuboids in the slab were changed along the span of the slab and perpendicular to the span direction as shown in Table 1 without changing the dimensions, arrangement of the reinforcement and the spaces between cuboids.

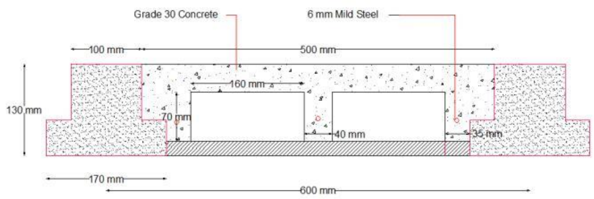


Figure 6: Structural arrangement of the optimized NERD slab

"by the cost of computing power and later by the successful widespread adoption of CAD" (Eastman, 2008).

Building Information Modeling (BIM) is one such approach which is now considered as a whole new process and methodology rather than just a technology. Building Information Modeling (BIM) is getting increasingly valuable in construction industry.

Therefore, the basic objective of this research is to study BIM and BIM tools to determine its benefits for construction managers. In this research, the uses of BIM which include 3D coordination, visualization, cost estimation, construction planning and monitoring, prefabrication and record model are discussed in detail. And the source of these common problems of construction managers is identified. Finally a model is proposed to implement BIM as BIM tools has potential to minimize most of these problems.

1. Research Methodology:

Literature Review: Literature review is used to systematically review earlier writings so as to learn more about the subject and other topics which collectively establish the context of this study. The goal was to gain a comprehensive understanding of the Building Information modeling and how the BIM is currently addressing the problems of construction managers in construction industry. It was completed in two phases. Firstly, it was used to gain a comprehensive understanding of BIM, BIM scope for construction managers and its adoption in construction industry. Previously conducted researches were reviewed to obtain knowledge about barriers and challenges which inhibits BIM adoption in industry.

Case Study: A case study approach is adopted to understand the solution of problems related to construction management and how BIM is minimizing the source of these problems.

Survey Questionnaire: Surveys are part of quantitative research and the focus of quantitative research is on objective measures rather than subjective experience. Surveys include cross sectional and longitudinal studies using questionnaires or interviews for data collection with the intent of estimating the characteristics of a large population of interest based on a smaller sample from that population *(RUIKAR, 2004).* Due to the nature of research question, a survey approach was used to obtain primary research data. The reason to select questionnaire is because questionnaire offer several advantages as they are widely distributed and low cost, interviewer bias is eliminated, anonymity of respondents, respondent can answer at leisure. In addition to that, *NAOUM (1998)* described questionnaire as the most popular form of getting primary data from a relatively large number of respondents within a limited time frame.

1. Literature Review:

Building Information Modeling (BIM):

The Building Information Model is primarily a three dimensional digital representation of a building and its intrinsic characteristics. According to the National BIM Standard, Building Information Model is "a digital representation of physical and functional characteristics of a facility and a shared knowledge resource for information about a facility forming a reliable basis for decisions during its life-cycle; defined as existing from earliest conception to demolition" ("About the National BIM Standard-United States", 2010). National Institute of Sciences (2007) defines BIM as:

"A BIM or Building information model uses digital technology of last generation to model a computable representation of all the functional and physical characteristics of the facilities and concern information during its life cycle and is intended to be a source of information for the service owner/operator to use and maintain that service during its life cycle."

According to Autodesk (2002), BIMs have three basic characteristics. Create and operate in digital databases for collaboration. To manage the change through these databases in order to make a change in any part of the database is coordinated with all other parts. Capture and preserve the data for reuse by adding industry-specific applications.

The findings of NIBS (2007), MAUNULA (2008)&SUCCAR (2008) that acronym BIM is used in three different ways. BIM can refer three different schools of thought which consider BIM as:

i. A product
ii. An activity/Process
iii. A system , a whole new concept

Problems faced with Construction Management:

In a CAD project, after the hundred percentage construction documentation and addendum and revision information has been turned over to the contractor, construction begins. Before the first scoop of dirt is moved, it is the construction manager responsibility to verify that the immediate information need is adequate and the most recent. Further issues for the project need to be identified and put on a path of resolution for all scopes of work. In theory, this review period for the project allows the construction manager to identify issues and give responsibility to the correct subcontractors. In itself, this is an arduous process.

Analyzing and overlaying CAD files and sheet drawings is time consuming work, it lack in visibility and it is prone to errors and missing information. In reality the construction manager is often cannot juggle managing the project documentation, the trades, the field management, and the construction managers own management team.

Advantages of BIM for Construction Management:

Each professional and user will have a point of view about the benefits of BIM in construction industry especially for construction managers. Their judgment approach or orientation may be different but as whole they are agree that BIM has remarkable effects on construction management.

a. Visualization

Building Information modeling (BIM) is an excellent visualization tool. It can provide a three-dimensional virtual representation of the building. It can help the contesting companies during bidding process because it can provide a better understanding what they have to construct and how it will look like after completion. It is possible due to renderings, walkthrough videos extracted from BIM model by using BIM tools.

b. 3D Coordination

Coordination is very important for a construction manager, especially when dealing with dense urban environment or challenging site. Coordination involves working and communicating with subcontractor, supervisors, materials suppliers, fabricators and equipment suppliers. In addition to juggling the scheduling, managing the budget, sorting through constructability issues, and managing relationship, the construction manager is also responsible who is doing what work on a project. Therefore the coordination efforts in advance of construction will reduce design errors and provide better understating of work ahead of time to be done.

c. Prefabrication

Offsite fabrication required considerable planning and accurate design information. It is becoming more common for contractors to fabricate component offsite to reduce labor cost construction time and better quality control. In offsite fabrication, different component or items can be assembled or fabricate in controlled environment and with greater precision and if any alteration is required then more option are available than site. A ductwork contractor can also use BIM model to installed branches and leave opening where required so that later on diffusers or hood can be girder boundaries. The internal pre-stressing of the upper section of the beam was replaced by external cables arranged over a small-sized mast located atop of the pier of the bridge he proposed (Figure 1).

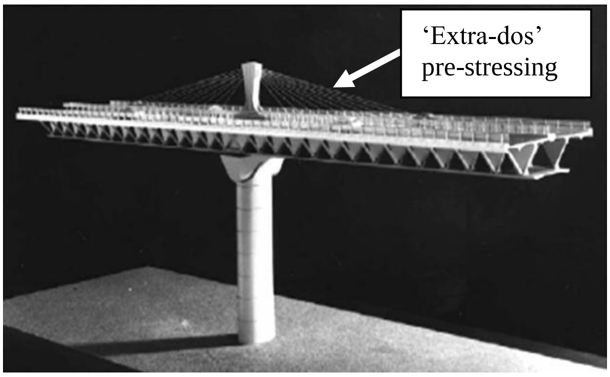


Figure 1: Proposed Viaduct for Arrêt Darré [2]

Since the external pre-stressing arranged by Mathivat was akin to the ‘extra-dos’, which is the upper curve of an arch, this new form of PC bridge was referred to as the ‘Extra-dosed’ type. Extra-dosed PC bridges are a hybrid form of bridge incorporating the structural features of PC girder bridges and those of cable-stayed bridges. While in a cable stayed bridge the vertical load is taken exclusively by the stay cables, in an extra-dosed bridge only a proportion of the vertical load is taken by the external cables (cable stays), while the girder itself takes a significant proportion of the vertical load resulting in larger girder depths than for cable stayed bridges of the same span.

The cable stays of an extra-dosed bridge essentially act as external pre-stressing but with a higher effective eccentricity than for conventional external pre-stressing which lie within the confines of the girder structure, resulting in a reduction of girder size compared to girder bridges of the same span. Due to the cable stays acting as external pre-stressing supporting only a proportion of the live load, the cable stays (external pre-stressing) can be stressed to higher stresses than those allowed in cable-stayed bridges [3] as the cables will be less severely loaded for fatigue considerations. In summary, the structural concept of extra-dosed bridges can be described as a PC box girder bridge with external pre-stressing through stay cables which also carry a portion of the vertical load.

3. General design outline

The proposed extra-dosed bridge is a 3-span structure with a 180m main span and two 100m long side spans. The main span length was determined by the design constraint of the need to avoid locating piers within the river limits. The side span lengths were constrained by the need to avoid locating piers on existing roads and the need to keep sufficient head-room over the said roads. An acceptable ratio of main span to side span length was also required in order to minimise out-of-plane forces on the pylon structure. Hence a main span to side span ratio of 1.8 was chosen. The bridge spans from P19 at station 800m to P22 at station 1180m, with pylons P20 and P21 located at stations 900m and 1080m respectively. This notation will be used throughout this paper. The layout of the proposed bridge with respect to the existing roads and bridge is shown in Figure 2.

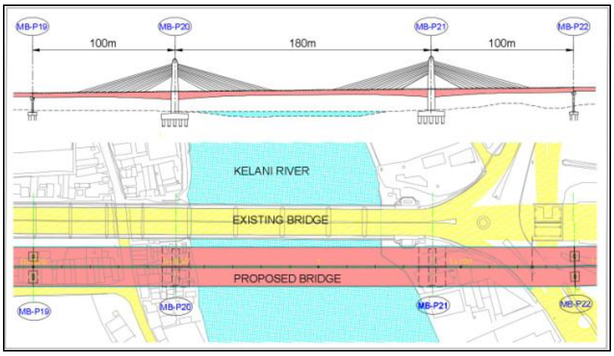


Figure 2: Layout of bridge – plan and elevation

A three cell box girder was chosen as the cross section for the main girder of the bridge. This cross section was chosen based on its high torsional rigidity as well as due to the wide nature of the deck which was designed to support 6 lanes of traffic. The cross sections of the girder at the pylon locations and at mid-span are given in Figure 3.

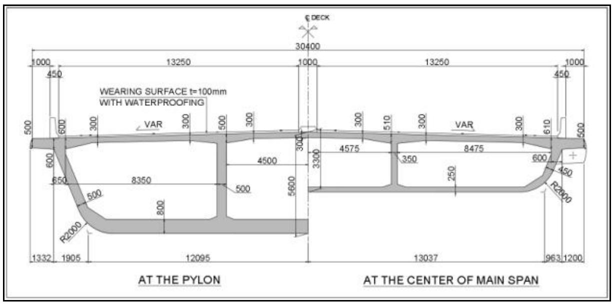


Figure 3: Cross section of main girder

The cross section heights are 5.6m at the pylon locations and 3.3m at mid span and side span ends. As per published literature [3] for extra-dosed bridges, the girder height is usually in the order of L/35~L/45 at the pylon and L/50~L/60 at mid-span, where L is the main span length. For a 180m span this translates into a height of 4~5.1m at the pylon and 3~3.6m at mid-span. A slightly larger value of girder height was chosen for the proposed bridge in order to minimise the size of the stay cables that would be required. In Table 1 typical extra-dosed bridge girder heights are compared to typical values of cable-stayed bridges and PC box girder bridges for the same span.

Table 1: Girder heights for three bridge types

Type of bridge	At pylon	At mid-span
Extra-dosed bridge	L/35 ~ L/45	L/50~L/60
Cable stayed bridge	L/80 ~ L/100 (constant)	
Box girder bridge	L/8 ~ L/16	L/35 ~ L/40

The girder height varies parabolically from 5.6m at the pylon location to 3.3m, 61m either side of the pylon centreline. The girder height is constant from Station 800-839m, for the middle 58m of the main span and also from station 1141-1180m. The top slab is 300mm thick throughout the length of the bridge while the bottom slab thickness and web thickness varies along the length of the bridge as shown in Figure 4.

The girder is supported at the pylon locations and at the end piers on 4 pot bearings each which are located near or directly beneath the web walls. The bearings, which provide no rotational restraint, are fixed in translation in the direction transverse to the bridge axis at all piers, and are free in the longitudinal direction at all piers except at P21.

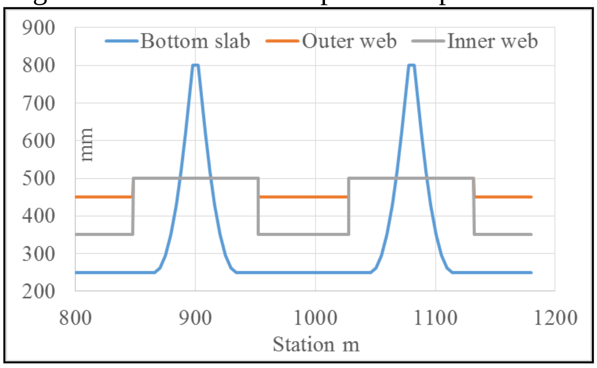


Figure 4: Thickness variation of slabs and webs

Providing longitudinal fixity only at a single pier is not usual in long-span bridge design. This layout was adopted since the design longitudinal load-effects due to wind, temperature and seismic loading in Sri Lanka were relatively minor. The girder is also supported by a system of stay cables emanating from two U-shaped pylons with a twin tower configuration. The twin towers are approximately 20m high above the top surface of the box girder and are inclined 5^0 to the vertical for aesthetic reasons. Each tower supports two planes of stay cables composed of 12 stay cables each. Hence 24 stays emanate out from each pylon. The design resulted in the shortest six cables in each plane being 27 tendon cables while the longest six were 37 tendon cables. The layout of the pylons and stay cables are shown in Figures 5 and 6. The twin towers are rigidly connected to the pylon pier while the connection between the girder and pylon pier is through pot bearings as described.

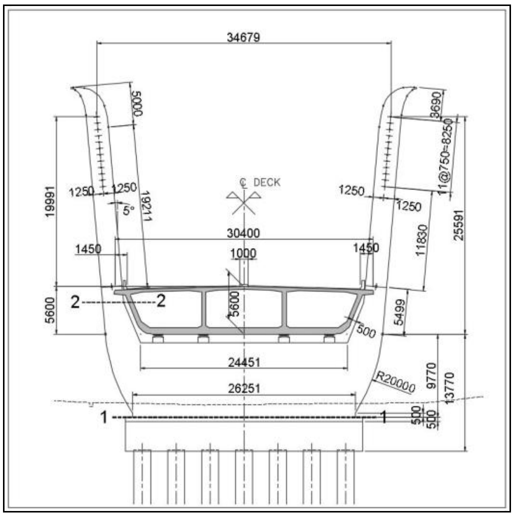


Figure 5: Pylon layout

General design guidance [3] states that for an extra-dosed bridge the tower height above the girder level is of the order of L/8~L/15 which for a 180m span gives a tower height of 12~22.5m. . Hence the tower height of 20m that was chosen falls within the general design guidance. For comparison, a cable–stayed bridge tower would be approximately 36~60m high for the same span. A double plane stay cable arrangement as described was chosen given the need to incorporate a 30.4m wide deck and due to the increase in torsional stiffness a double plane stay arrangement offers. A fan-type arrangement of stay cables was chosen out of the types commonly used (Figure 7).

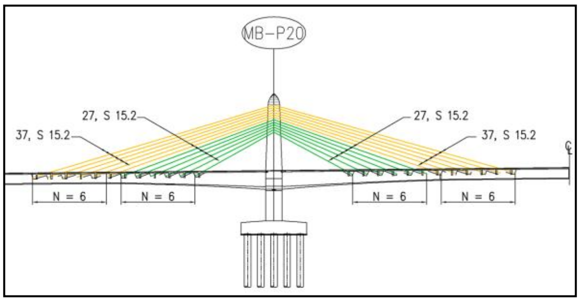


Figure 6: Stay cable layout (P20/P21)

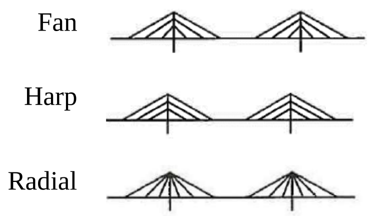


Figure 7: Types of stay cable arrangement

The fan type, which is a hybrid arrangement in between the radial and harp types, utilises cable stays more efficiently than the other types while keeping the sectional forces in the pylon at an acceptable level especially compared to those resulting from the radial type arrangement. The stay cables are located at 4.5m intervals along the suspended length of the girder and spaced at 0.75m intervals at the towers. At the tower a saddle type anchoring system (Figure 8) was chosen since it results in a smaller tower width and smaller spacing of stay cables at the towers than alternative anchorage systems. The 4.5m interval along the girder corresponds to the segment length considered for the girder construction.

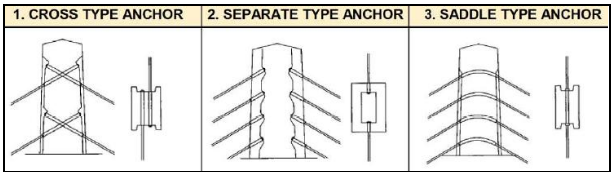


Figure 8: Anchorage systems at pylons

A double tube saddle type tower anchorage system (Figure 9) which allows for the replacement of stay cables was chosen.

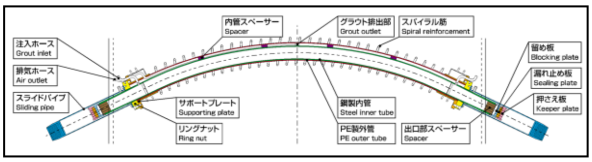


Figure 9: Double-tube saddle anchorage (typical) [4]

Usually, the suspended length of the girder, which is the length supported by stay cables, is of the order of 0.2L. However for this bridge the suspended length was increased to 0.28L, taking into account the deck size as well as to keep the stay cable size to a minimum (Figure 10)

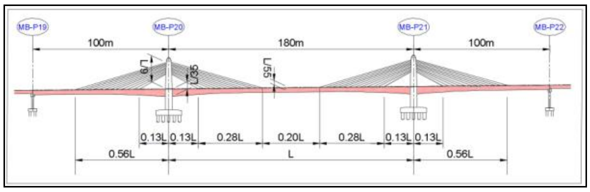


Figure 10: Stay cable layout along the bridge

At the girder level each stay cable is anchored to the girder through anchorages (Figure 11) located on the sides of the bridge deck.

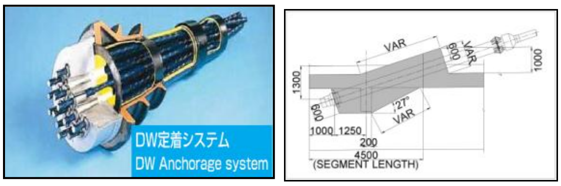


Figure 11: Stay cable anchorage (typical) [4]

The segments of the bridge which contain stay cable anchorages also consist of 400mm thick full width cross beams which are 1750mm high as seen in Figure 12. The structural effect of the cross beams is to improve the load-distribution within the girder cross section of the stay cable forces and to improve the transverse resistance of the girder.

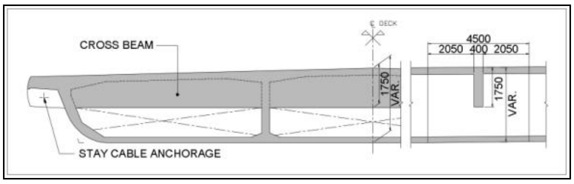


Figure 12: Cross beam layout

Taking into account its excellent corrosion resistance as well as relative ease of construction, epoxy coated and filled (ECF) tendons (Figure 13) will be used for the stay cables of the proposed bridge. In addition to the epoxy coating, the tendon also has a polyethylene (PE) covering and the stay cable itself has a protective PE pipe in which all the tendons are enclosed. ECF tendons also offer superior fretting fatigue resistance compared to other alternatives which is advantageous since the tendons will be susceptible to fretting fatigue due to the saddle type anchorage used at the towers.

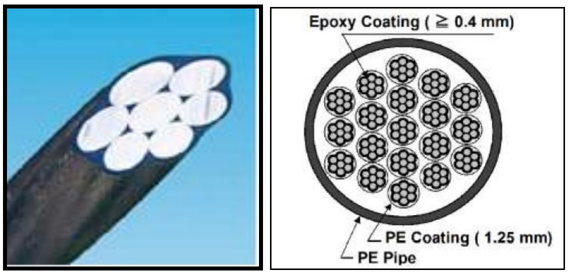


Figure 13: Typical ECF tendon [4] and Stay cable

The sub-surface soil profile at the locations of the proposed piers consist of a thick alluvium layer composed of layers of peat, clay and sand overlaying the bedrock layer. The rock layer consisted of highly to moderately weathered gneiss and was located approximately 25~30m below mean sea level. The allowable bearing capacity for the design of piles socketed in rock was recommended to be 3000kPa together with an ultimate socket friction of ~200kPa. The decision to locate the fixed bearing condition in P20 was made since fixing the girder at P20 resulted in larger lateral forces at P21 (governed by creep and shrinkage effects) and since the ground conditions at P21 were more favourable than at P20.

Table 2: Construction sequence (time in months)

Construction activity	Time
Pile cap, pylon pier and pier head	13
Girder segments without stay cables and part construction of towers	+3.5
Girder segments with stay cables and completion of tower construction	+7.5
Completion of cantilevers	+ 1
Construction of side spans	+ 3
Construction of closure segment at mid span	+1.5
Parapet construction and surfacing	+2

The construction of the proposed extra-dosed bridge will be carried out using the balanced cantilever method with two cantilevers on either side being constructed from each pylon. Table 2 outlines the general planned sequence of construction and approximate timelines. An assumed construction schedule was considered for the structural analysis which is described in the next section.

4. Structural modelling and analysis

The structural modelling for the design of the extra-dosed bridge was done using the CsiBridge2015 analysis software. For the consideration of global effects, a three-dimensional finite element (FE) model consisting of 1-D elements was used. The box girder, pylons, piers and cables were modelled using 1-D frame elements with equivalent stiffness properties. A screen-shot of the finite element model is given in Figure 14.

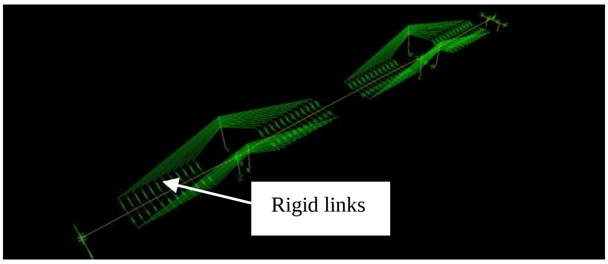


Figure 14: FE analysis model (tendons not shown)

The elements were modelled along the locations of their centroids and the connections between the stay cables and girder were made through rigid links as shown. Since the girder was modelled using frame elements the cross beams were not explicitly modelled. The effects of the cross beams were considered by the use of rigid links as described above. The stay cable anchorage points considered in the model corresponded to their locations in the actual structure. The pylon support foundations were modelled using coupled translational and rotational springs and was updated throughout the analysis to reflect the actual foundation configuration designed. The pot bearings supporting the main girder were modelled using springs with very high translational stiffness with releases specified as appropriate. Hence the connection between the girder elements and the pylon elements in the model was through these spring elements. The 'pier table' of the pylon was modelled by constraining the joints corresponding to the bottom of the pot bearings and the bottom of the towers to act as a rigid body. The stay cables were rigidly connected to the towers at the pylons.
Initially the analysis was done without including the internal pre-stressing tendons within the model. This was done in order to obtain the load-effects of the girder to estimate the required number of internal pre-stressing tendons. The number of internal tendons were then estimated, with an allowance of approximately 2MPa for secondary effects of pre-stressing for the girder. The pre-stressing tendon layout thus designed was then explicitly modelled as elements in the FE model (Figure 15).

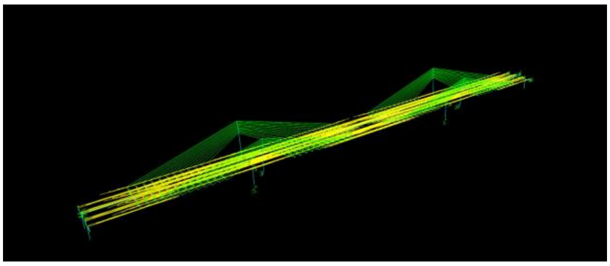


Figure 15: FE model with tendons (in yellow)

The jacking stress for the tendons was specified to be $0.72f_{pu}$. All pre-stress losses were calculated through the software using the following loss parameters. Jacking from both ends was assumed for all internal tendons.

- Friction coefficient 0.3 /rad
- Wobble coefficient 0.004 rad/m
- Wedge draw in 5 mm

The following main loads were considered in the analysis;

1. Dead load and super-dead loads
2. Live loading due to HA and HB loads
3. Wind loading
4. Temperature loading
5. Creep and shrinkage
6. Differential settlement of piers (10mm)
7. Cable and tendon pre-stressing effects
8. Secondary live loading
9. Frictional restraint effects

All loads were considered in accordance with BS5400:2 [5] with traffic loading being taken from BS5400:2(1978). 45 units of HB loading were considered for the analysis. In addition to the aforementioned loads the following special loading conditions were also considered.

1. Sudden loss / replacement of any one stay
2. Replacement of any one bearing

The sudden loss of any one stay was modelled by removing the cable element from the model and re-running the analysis, with equal and opposite forces applied to the girder and tower locations to which the cable was connected to, equal in value to the force in the particular cable at the ULS obtained from the original model (with all load factors set to 1.0). A 1.8 impact factor was applied to take into account dynamic effects. A similar approach was used for the stay and bearing replacement conditions (without the impact factor).

As the bridge will be constructed using the balanced cantilever method a staged analysis was done in order to realistically model dead load effects and effects due to creep and shrinkage. An assumed construction schedule was used for the staged analysis. A 15 day cycle was considered for the construction of girder segments without stay cable anchorages and an 18 day cycle was considered for segments with stay cable anchorages. For each stage, the respective girder segments were added after which the dead load and internal pre-stressing were applied and stay cable pre-stressing applied thereafter where appropriate. During construction of the cantilevers the springs modelling the pot-bearings at the pylons were temporarily assigned to provide full restraint. In reality too, a temporary fixing arrangement will be constructed at the pylon locations to facilitate balanced cantilever construction. When adding the respective segments in the analysis model, segments on either side of the pylon were added at the same time, mimicking the proposed actual construction sequence. Once the cantilever construction was completed, the side-spans were added to the model after which the rotational restraints temporarily assigned to the pylon bearing springs were released. The closure segment at mid-span was then added and the final translational releases were assigned to the bearing-springs, prior to stressing the bottom tendons of the closure segment. The super-dead loads were then added and the effects of long term creep and shrinkage were assessed through time-lapse load-stages which calculated effects up-to 30 years ($T=\infty$) after completion of the bridge ($T=0$). The creep and shrinkage calculation was done through the software which followed the procedure specified in the CEB-FIP 1990 model code [6]. The creep and shrinkage effects were considered not only for the long term but throughout the construction period. Figure 16 shows a screen-shot of the stage at which the cantilevers emanating out of P20 have been completed.

The effects of all other loads were calculated using the staged analysis model (and associated stiffness) at T=0. The HA and HB live load effects were calculated through influence line analysis using the in-built function of the analysis software. For the wind loading a basic wind speed of 33.5 ms^{-1} was considered [7] while for temperature loading effects, a uniform temperature difference of +/- 7^0C was considered with an installation temperature of 32^0C [8]. A temperature difference +/- 8^0C was considered between steel and concrete elements of the bridge. The re-distribution of load effects due to the change of support fixities was calculated through the software itself. During the staged analysis, the loading from the form traveller was considered as a point load of 160T while a construction live load of 14.6 kN/m on one cantilever and half the load on the other was also considered. The main material parameters considered in the analysis are tabulated in Table 3.

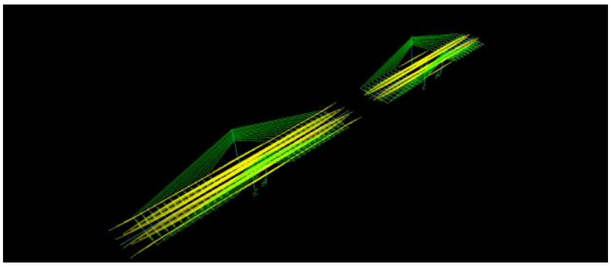


Figure 16: Model at completion of P20 cantilevers

Table 3: Main material parameters considered

Parameter	Value
E (Young's Modulus) of girder (1.15 x 34 – taking into account effect of rebar and tendons) - G50 concrete	39.1 GPa
E of tower - G50 concrete	34 GPa
E of pier (pylon piers included) – G40	31 GPa
E of pre-stressing tendons/cable stays	200 GPa
Shrinkage start date as per [6]	3 days
UTS of tendons f_{pu}	1850 MPa
Relative humidity	70%
Shrinkage coefficient as per [6] β_{sc}	5
Relaxation class as per [6]	2

The full sectional stiffness was considered for the girder elements in the analysis while the sectional stiffness of the pylons and piers were reduced by 50% to account for the fact that these will be cracked at SLS. The same analysis model was used for SLS and ULS, in line with limit state theory. For the stay cables no 'apparent modulus' effects [9] were considered, since even for the longest cable, the change in modulus was negligible.

Initially the analysis was run with all stay cables considered as 27 tendon cables. However it was ascertained that the cable capacity was not sufficient to meet the design criteria upon which the longest six stay cables emanating from each tower was changed to 37 tendon cables. The stay cable pre-stress was applied through the software at each relevant analysis stage as a 'target-force' load-case in which the software increased the strain of the cable until it achieved the specified force. The amount of stay cable pre-stress was initially determined considering the remaining allowable force increase in the cables after the resulting SLS loads in the cables without pre-stress were deducted. Since staged analysis is a type of non-linear analysis, the maximum amount of pre-stress was finalised through iteration.

Creep and shrinkage loss of internal pre-stress was accounted for in the analysis itself as the tendons were modelled explicitly and deformed compatibly with the elements they were embedded to.

5. Detailed design of box girder

Using the load-effects from the global analysis, the SLS and ULS design of the main box girder for longitudinal effects was carried out. The steps described in sections 5.1 to 5.3 were followed in the design. In the longitudinal direction, the main box girder was designed as a Class 2 pre-stressed concrete member as per BS5400-4 [5]. The internal pre-stressing layouts that were designed for the top and bottom slabs of the main box girder are shown in Figures 17a-c. The arrangement is symmetric about the centreline of the girder cross section.

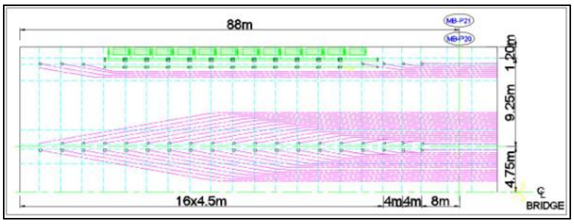


Figure 17a: Top slab pre-stressing (for P20/P21 cantilever spans) – 88 x 15ø15.2mm tendons

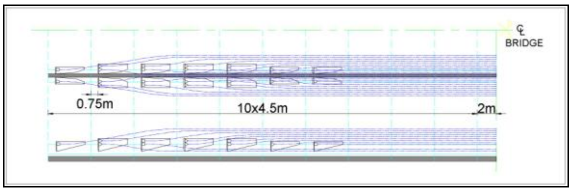


Figure 17b: Bottom slab pre-stressing (mid-span) 66 x 15ø15.2mm tendons

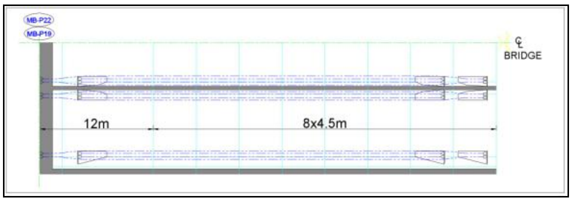


Figure 17c: Bottom slab pre-stressing (side-spans) – 24 x 15ø15.2mm tendons

5.1 Stress check for completed bridge

The extreme fibre stresses of the main girder cross section due to the critical load combinations were calculated for the bridge at and after completion. The resulting stresses were then checked with the relevant stress limitations, which as per BS5400-4 Section 6.3.2 [5] were 2.55 MPa in tension and 20 MPa in compression for grade 50 concrete. The calculated extreme fibre stresses along the bridge are shown in Figure 18. When calculating the stresses, for contributions from the axial forces applied on the girder by the stay cables and internal tendons, a distribution angle of 33^0 was considered [10], since the axial forces are not immediately effective across the whole cross section (Figure 19). This resulted in an effective distribution length behind the anchorage of approximately 7.5m. In the calculation of stresses the contribution from the aforesaid axial forces was only considered effective after this length.

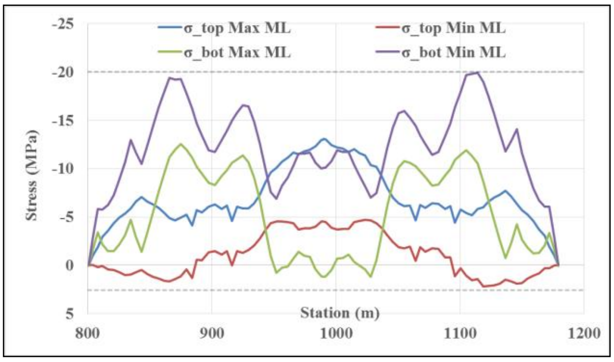


Figure 18: Extreme SLS fibre stresses (T = 0 to ∞) (Tension positive, sagging moment positive)

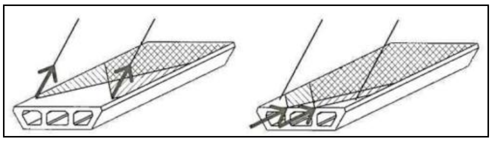


Figure 19: Distribution of applied axial forces

5.2 Stress check during construction

Similar to 5.1, stresses were also calculated for load-effects during construction. It was confirmed that the maximum and minimum stresses during construction were also within the required limits.

5.3 Ultimate capacity checks

In addition to the SLS design, the ULS moment, shear and torsion capacities were also checked. For the longitudinal moment capacity in order to obtain the required capacity above the applied ULS moment it was necessary to design and consider the capacity contributions from the reinforcement of the top and bottom slabs. The longitudinal reinforcement thus designed is tabulated in Table 4. The moment capacity was calculated taking into account the co-existing axial force in the section. Figure 20 shows the variation of maximum and minimum ULS longitudinal moments and the calculated ULS capacities.

Table 4: Slab rebar (top and bottom surfaces)

Slab	Rebar (c/c in mm)	Length along bridge
Top slab	H12@150 c/c	Full length
Bottom slab	H12@150 c/c	P20/21 to P20/21 +/- 12m, P20/21 +/- 54m to +/-90m and side span ends
	H25@150 c/c	P20/21 +/- 12m to +/- 36m
	H20@150 c/c	P20/21 +/- 36m to +/- 54m

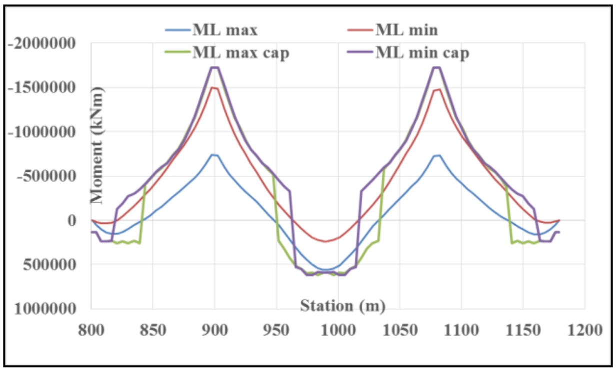


Figure 20: ULS moments and capacities (Sagging moments positive)

The ULS shear and torsion effects were also assessed. The distribution of shear between the outer and inner webs was obtained through an additional finite element model which modelled each web and associated top and bottom slabs as separate elements along with the cross beams. For the outer and inner webs maximum distribution ratios of 0.37 and 0.20 were obtained. These ratios together with the obtained load-effects from the main analysis model was used for ULS shear design of the girder.

5.4 Displacement of girder

The displaced shape of the bridge due to dead and super dead loads (including pre-stress) at the end of creep and shrinkage is shown in Figure 21. A maximum displacement of 423mm (~ L/425) was calculated from the analysis at mid-span. During construction this long term deflection needs to be taken into account in order to ensure that the road alignment of the structure achieves the design requirement in the long term.

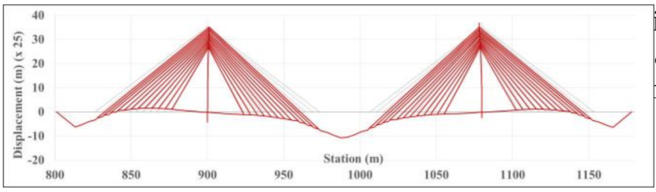


Figure 21: Long term bridge displacement

6. Detailed design of stay cables

The stay cables were designed ensuring that SLS loads in the cables did not exceed $0.6f_{pu}$. For the load-cases of sudden loss of one stay and stay replacement, a stress of $0.65f_{pu}$ was permitted while during construction a maximum stress of $0.7f_{pu}$ was considered permissible. The resulting maximum SLS cable loads for cables emanating from P20 and P21 are shown in Figure 22. Results are presented for cables of one tower of each pylon (as effects are nearly symmetric). As can be seen the maximum cable loads are less than the allowable for all cables. Since the loads in the cables vary due to the live load, fatigue of the cables was also considered. The allowable stress for fatigue is a function of the maximum allowable SLS stress [3] as shown in Figure 23.

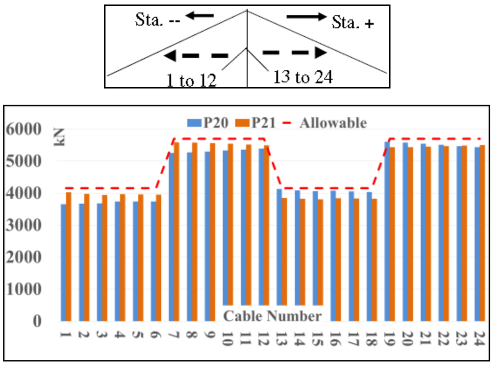


Figure 22: Maximum SLS cable loads (P20/P21)

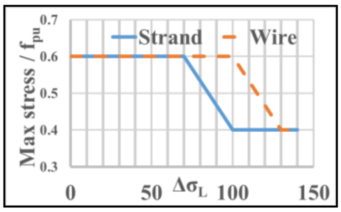


Figure 23: Allowable fatigue stress range ($\Delta\sigma_L$) [3]

For an allowable stress of $0.6f_{pu}$, the allowable stress range is 70 MPa. The calculated cable stress ranges due to live load (HA loading only) are shown in Figure 24. Using HA loading to assess fatigue stress ranges may seem overly conservative. However this method is acceptable ince additional bending stresses induced in the ables near anchorages [11] are not explicitly taken nto account in the analysis.

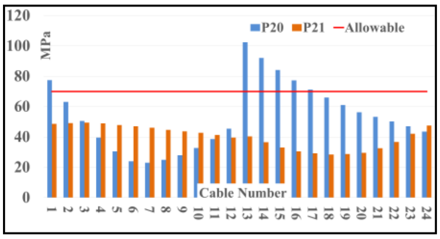


Figure 24: Stay Cable stresses due to HA loading

The shortest five cables of P20 were observed to have stress ranges above the limiting value. This was mitigated by increasing the number of tendons used for these stays. Cable vibrations due to wind/rain will be monitored during construction and damping devices will be designed and installed as required.

7. Detailed design of pylons

The towers of the pylon vary from a 2.5m x 3.5m section at the top to a 2.5m x 5m section at the level of the top of the girder, after which the section increases in width until the level of the 'pier-table', as shown in Figure 25. Below the level of the pier table the pylon 'pier' is a cellular box structure with the typical section as shown in Figure 26. The overall width of the pier varies from 32.71m at level of the bearings to 26.25m at level of the top of the pile cap.

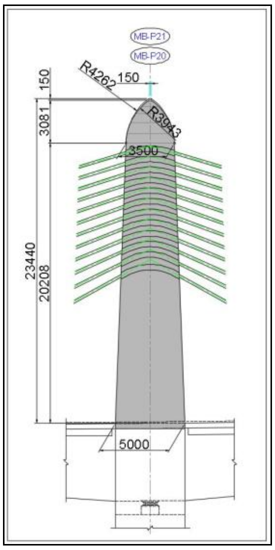


Figure 25: Pylon tower section variation

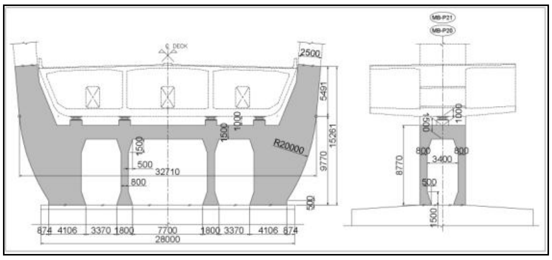


Figure 26: Pylon pier typical cross sections

The tower and pier sections were designed as bi-axially loaded reinforced concrete columns at the ULS and the crack widths were checked at the SLS. The ULS maximum axial force and sectional moments for Sections 1-1 and 2-2 as defined in Figure 5, are given in Table 5 for pylon P20, along with the designed perimeter axial reinforcement. The design of the end piers is not explicitly described in this paper as its design depends on the loadings from the approach bridge as well. However the design philosophy of the piers is the same as that of the pylons.

Table 5: ULS load-effects for pylon P20

Load-effect	Sect 1-1	Sect 2-2
Max compression kN	257606	34818
Min compression kN	185037	53357
Max moment about longitudinal axis	106814	37936
Max moment about transverse axis	501642	120504
Max shear in longitudinal dir.	14955	7365
Max shear in transverse dir.	2944	1998
Max torsion	39439	4
Designed axial rebar	H32@150 mm c/c	H32+H40 @150mm

8. Detailed design of foundations

The designed pile layout for pylons P20 and P21 is shown in Figure 27 while the corresponding pile cap section is shown in Figure 28. 2m diameter piles were considered in the design.

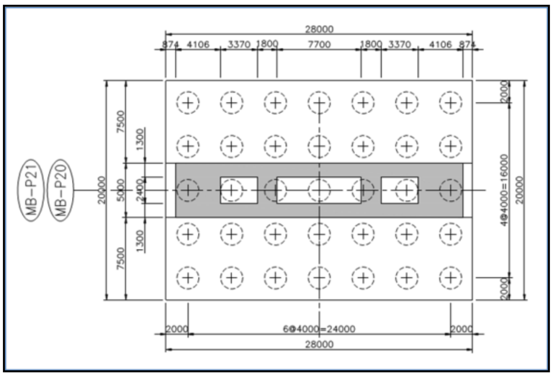


Figure 27: Pile layout of P20/P21 foundations

The design of piles was carried out by using the reactions of the pylon support springs which modelled the effect of the pile foundations in the global analysis model, and applying the said reactions as input loads to a separate finite element model of the pile system. This finite element model is shown in Figure 29.

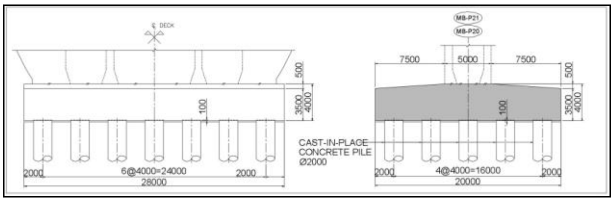


Figure 28: Pile cap dimensions P20/P21

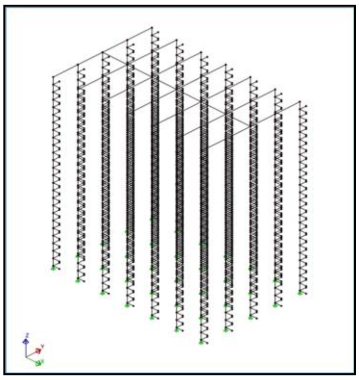


Figure 29 – FE model used for pile design

The top of the piles in the above model was joined together by rigid elements as the 4m high pile cap was considered to be rigid. The piles were supported by springs spaced at 1m intervals which modelled the varying stiffness of the soil layers. The bearing stress from the pile acting on the bearing layer was calculated using the SLS axial force at the bottom of the pile. The pile reinforcement was designed for the ULS condition by considered the pile as a bi-axially loaded reinforced concrete column. The maximum load-effects used for the design of piles in P20 and P21 are tabulated in Table 6. The pile caps of both P20 and P21 pylons are 28m long, 20m wide and 4m high. The pile cap reinforcement was designed based on the moments in the 28m x 4m sections at the face of the pylon pier. This resulted in bottom main reinforcement of 2 x 2H32@150mm c/c. Due to the arrangement of the pylons, the 20m x 4m sections were not critical for the pile cap design.

9. Conclusion and further work

This paper has presented and discussed the detailed design of the main structural elements of the proposed new extra-dosed bridge over the Kelani River, which will be the first of its kind in the country. At the time of writing this paper the detailed design work is ongoing, especially with regard to bearing design, anchorage design and transverse design of the box girder and cross-beams. It is hoped that more details of the design will be the subject of a separate paper in the future.

Table 6: Pile load-effects (kN/kNm)

Load effect	P20	P21
Max SLS axial compression	11618	12122
Max ULS axial compression	15040	15604
Min ULS axial compression	4740	3978
Max ULS moment (moment	1357 (43)	1266 (20)
about ppclr. dir in brackets)	1027 (541)	1122 (603)
Axial rebar	40H25 @ upper part	
	40H20 @ lower part	

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