STONECUTTERS AND FORTH BRIDGE –
HOLISTIC APPROACH TO DESIGN OF LONG SPAN CABLE STAYED BRIDGES

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ABSTRACT

This paper gives an over-view of the design and construction of the visually unique Stonecutters Bridge Hong Kong, currently the second longest cable-stayed span in the world, and the progression from it to the design of the equally unique 3-tower cable-stayed Forth Replacement Crossing Bridge in Scotland.

The concept design for Stonecutters Bridge was acquired through an international design competition in 2000, whereas the concept design for the Forth Bridge was acquired by selecting a design team in 2008 via a dialogue procedure and then getting the design team to develop alternative concepts from which the chosen concept was selected and developed.

For each of the bridges the design has had to consider extreme events of wind, seismic and ship impact.

1. STONECUTTERS BRIDGE

Stonecutters Bridge carries a dual 3-lane expressway and spans the Rambler Channel at the entrance to Hong Kong container terminals, providing high level clearance and linking container terminal 8 on Stonecutters Island on the east side to the new container terminal 9 on Tsing Yi Island on the west.

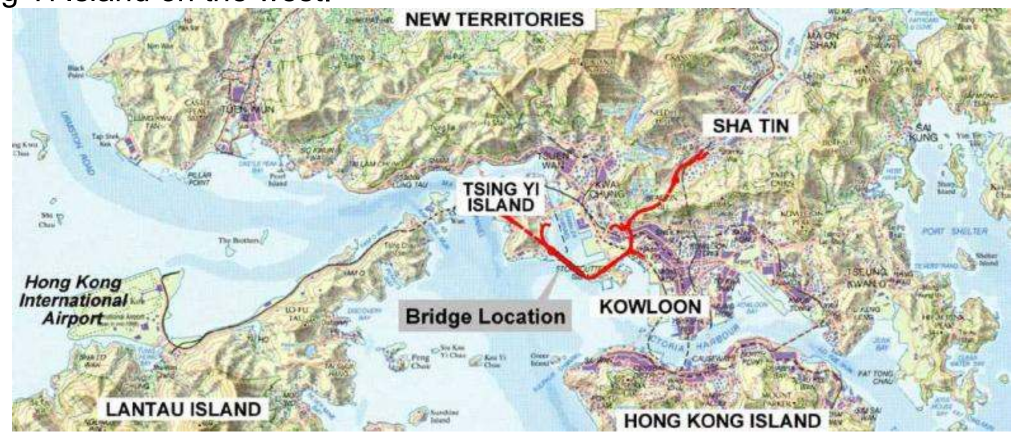

Figure 1 : Location of Stonecutters Bridge

1.1 International Design Competition And Reference Scheme

The international design competition was carried out in two stages. In the first stage 27 designs were received, and from these 5 designs were selected for Stage 2, as shown in Figure 2. The designers were asked to develop their designs, and the winning proposal selected by the technical and aesthetic evaluation committees was for a 2-tower monopole cable-stay bridge with a main span of 1018m. The designers were Halcrow Group Ltd, Flint & Neill Partnership, Shanghai Municipal Engineering Design Institute, and Dissing + Weitling Architects. This design became the Reference Scheme.

Halcrow Group Ltd.
Flint & Neill Prt. SMEDI
Dising + Weitling Arch.

Scott Wilson HK Ltd.
Leonhardt Andra Prt.
Prof. Laage

T.Y. Lin Int.
Gensler Arch.

HNTB
Wolf Architecture

Consult. KORTES Ltd.
Finroad Ltd.

Figure 2 – The Stage Two finalists and leading team members

Following the conclusion of the design competition, the client the Highways Department of Hong Kong, selected Ove Arup & Partners Hong Kong Ltd as the design and construction supervision consultant in a shortlisted tender based on the established Hong Kong Government technical/fee selection process.

1.2 Bridge Description

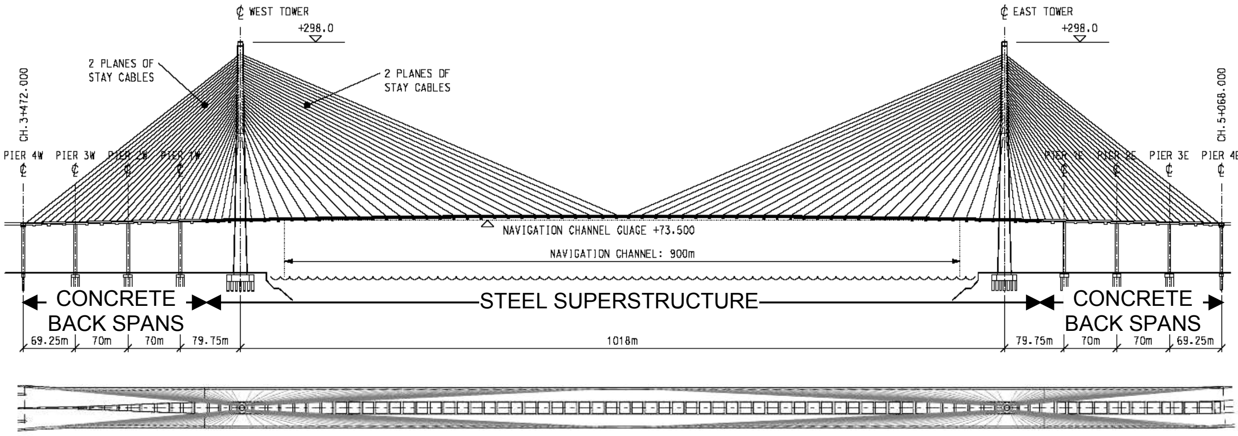

Figure 3 - Elevation and Plan

Stonecutters Bridge is cable-stayed with an orthotropic steel main span of 1018m, and a total length of 1596m (Figure 3). There are four prestressed concrete back spans on each side. The tapered mono-towers are in concrete up to level +175m and steel-concrete composite from level +175m to level +293m with the outer steel skin being duplex stainless steel. 5m tall glazing structures top the towers off to level +298m. The 2 planes of stay cables take a modified fan arrangement, anchored at the outer edges of the deck at 18m intervals in the main span and 10m intervals in the back spans.

The deck is a twin box-girder, with the two longitudinal girders connected by cross girders. The piers in the back spans are monolithically connected to the deck. The three intermediate piers are single column piers, while the end piers at the adjoining viaducts are twin column portal structures. Laterally the bridge deck is restrained by vertical bearings on the towers and by the back span piers. In the longitudinal direction dynamic movements are restrained by hydraulic buffers at the towers. The ground is reclaimed on both sides, and comprises a highly variable thickness of superficial deposits overlying bedrock typically at level –50m to –90m.

1.3 Detailed Design

The bridge was the first cable-stayed bridge in the world with a span over 1km for which detailed design was completed. The exposure of the site to typhoon winds created particular challenges, as did the busy harbour, which imposes severe restrictions on the construction operations. The bridge will carry traffic with a very high content (around 42%) of heavy goods vehicle.

1.3.1 Design for Extreme Events

a) Wind Loading

The wind dominated the design. The bridge is a large highly flexible structure and required a complete wind model for dynamic calculations. Wind turbulence intensity measurements were made near the bridge site to measure the site specific wind conditions. This helped to calibrate and supplement the results from a 1:1500 scale wind tunnel model of the surrounding terrain. Together these studies provided an understanding of the turbulent wind climate resulting from the nearby hills. The measured wind parameters were used to modify the design wind climate presented in the Design Memorandum.

Further wind tunnel studies included a deck section model at 1:80 scale and a high Reynolds Number deck section model at 1:20 scale to check for aerodynamic instability for wind speeds at deck level of up to 95m/sec. Also a 1:100 scale free-standing tower model was tested, and a 1:200 scale full bridge aeroelastic model to confirm the overall behaviour.

Wind buffeting calculations which allow the assessment of the actions on a flexible structure arising from the interaction between gusty winds and the dynamics of the structure were carried out in 2 separate pieces of software to ensure full confidence in the results from this complex analysis.

b) Ship Impact Simulations

The tower foundations are located approximately 10m behind the seawalls on both sides of the Rambler Channel. Given the close proximity, account was taken in the design for impact loading induced by a ship collision with the seawall. A series of centrifuge tests were carried out to model the effect of a 155,000 tonnes container ship impacting the seawall at a speed of 6 knots. The results of the test including pressure measurements aided calibration of a dynamic 3D finite element model, allowing the force exerted by the vessel impact at the front face of the tower foundations to be determined.

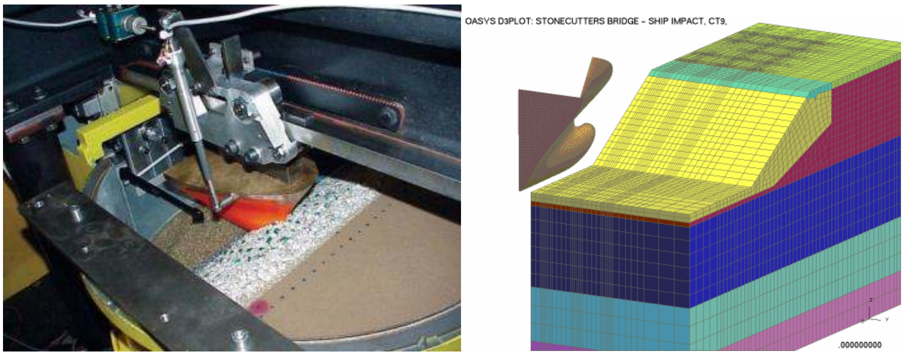

Figure 4 - Ship Impact Model Test and Numerical Simulation

c) Seismic Studies

A study of risk levels established three limit states, with earthquake return periods of 120 years for serviceability, 2400 years for ultimate and 6000 years for SILS (Structural Integrity Limit State). The bridge should behave elastically during frequently occurring or minor earthquakes (SLS) without the need for any repair. During a moderate earthquake (ULS) certain elements may undergo large deformations in the post elastic range without substantial reduction in strength, and damage level shall be minimal with repair carried out without the need for bridge closure. The deformation and damage during a severe earthquake (SILS) shall not be such as to endanger emergency traffic or cause loss of structural integrity but might require closure of the bridge for repair. The design earthquake ground motion is represented by site-specific design response spectra (with 5% damping) determined by a Probabilistic Seismic Hazard Assessment (PSHA) for the three return periods.

The PSHA combined the seismic source zoning, earthquake recurrence and the attenuation relationships to produce "hazard curves" showing levels of ground motion and associated annual frequencies of being exceeded. Summation of these from all possible magnitude ranges demonstrated the overall frequency of exceedance for each ground motion level.

1.3.2 Other Design Considerations

The expected fatigue loading in steel deck plate is intense due to the predicted numbers of heavy goods vehicles. The bridge is located in a sub-tropical climate with summer time temperatures frequently above 30°C. The reduction in stiffness of asphalt surfacing at high temperatures means that the benefit of the surfacing in acting compositely with the deck plate to reduce local stresses will be limited. To cope with this loading, without beneficial composite action with the surfacing, the orthotropic steel deck has been designed with an 18mm thick deck plate and 325mm deep, 9mm thick trough stiffeners.

The construction sequence needed to be taken into account in the design analysis. The concrete back spans were to be constructed in advance of the cantilevering of the main span deck. Full support was to be provided using falsework prior to installation of the stay cables, since without the stay cables the spans are not self-supporting. The back spans provide stability and resistance to the buffeting wind loads on the main span cantilever.

1.4 Construction

1.4.1 Concrete Backspans

a) Pier Shafts and Cross Heads

The intermediate pier shafts are between 60 and 65m tall, with hollow box sections tapering from 12.5m to 10m wide in the transverse direction, and having a constant thickness of 4m in the longitudinal direction. Walls are either 600mm or 1m thick. They were constructed with 60MPa concrete using a hydraulic climbing form system. The end portal shafts were constructed by similar techniques.

At each intermediate pier, the monolithic cross head was formed by in-situ cantilever construction. A temporary works truss cantilevering from the pier shaft provides the support in the temporary condition before the concrete has gained the required strength.

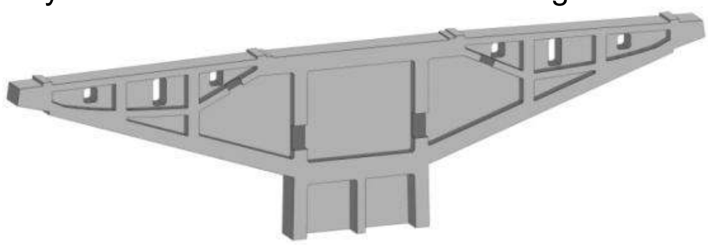

Figure 5 – Cutaway Section of Typical Pier Cross-Head

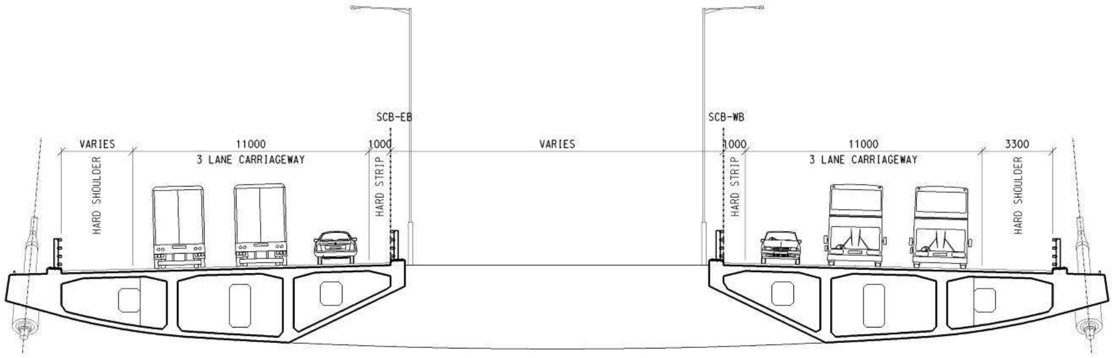

Figure 6 - Concrete Deck – East Back Spans

b) Concrete Deck

There are 3 cross girders in each back-span which were cast first as independent units. After the first stage of transverse prestress was applied, the two longitudinal deck bays between these cross girders were cast. After the remaining transverse prestress was applied, the final deck pours stitched the span concrete to the pier cross heads. Once a continuous deck was formed, the longitudinal prestress which is a combination of internal and external tendons of varying lengths was applied, with stressing taking place at the ends of the deck where there is adequate access.

This sequence allowed independent components of the deck to be constructed and adjusted to the correct geometry prior to forming an increasingly complex non-determinate structure.

Figure 7 – Back Spans under Construction

1.4.2 Towers

a) Lower Towers

The concrete lower towers have a tapering shape reducing from an elongated circular section 24m by 18m at the base to 14m diameter at deck level and 10.9m diameter at +175m. The wall thickness is a constant 2m up to deck level, and then tapered to 1.4m at +175m.

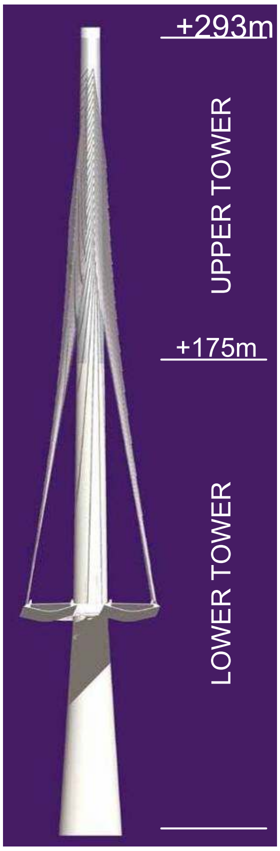

Figure 9 - Tower Elevation

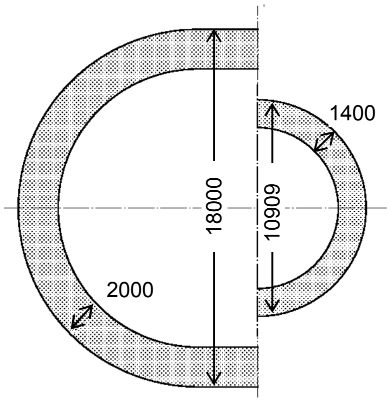

Figure 8 – Lower Tower Section at base and at +175m

Figure 10 - Lower Tower Construction

Figure 11 - Upper Tower

The complex shape was formed using a climbing formwork system (Figure 10). 10 individual panels carried the plywood shutters. Strips were cut off the edges to reduce the perimeter length for each pour. The high quality plywood had to be durable enough for the repeated pours, but also flexible enough to be bent into the ever decreasing radius shape.

The climbing operation to raise the form in preparation for the next pour was controlled by 10 pairs of screw jacks, supported on the top of the previous construction joint. A cycle time of 7 days was achieved for the typically 4m high pours, with concrete finishing works done from trailing platforms hanging below the main working platforms.

b) Upper Towers

The structure of the composite upper towers is considerably more complex (Figure 11). The circular section has a constant taper from 10.9m diameter at +175m to 7.16m diameter at +293mPD. The outer skin is a 20mm thick structural stainless steel shell. This is composite with a concrete wall, which tapers from 1400mm to become a constant 820mm thick. The lowest 3 sets of stay cables anchor in corbels on the inside face of the concrete wall, whereas the remaining 25 sets anchor within a steel box section forming the core of the tower.
In each tower, 32 stainless steel skin sections make up the outer shell and 25 carbon steel anchor box sections stretch from +195m to +280m. The geometry of the steelwork was carefully controlled in the fabrication process by trial assembly to ensure that when placed on site it fitted into place. Steelwork was lifted into place by the tower crane with site connections being bolted.

The East and West Towers were structurally completed in November and December 2008 respectively, followed by the installation of tower top glazing structure and the maintenance unit.

1.4.3 Steel Deck

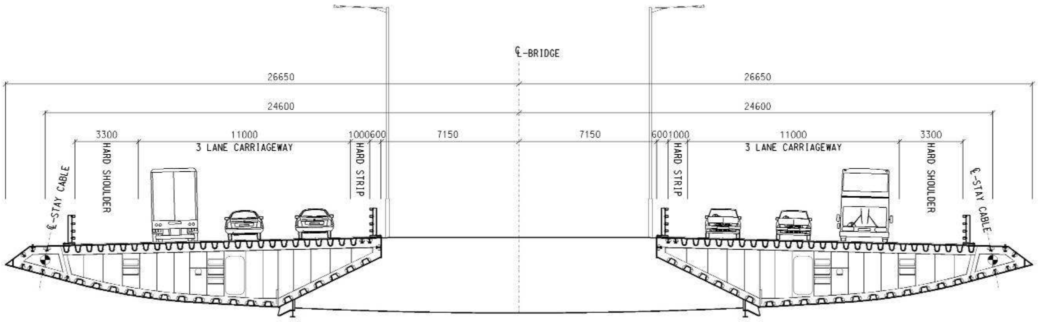

Figure 12 : Steel Deck

c) Fabrication and Assembly

Steel deck panels were fabricated in Shanghaiguan in North Eastern China and assembled into deck segments in Shatian, Guangdong province, Southern China. Match fabrication to ensure a consistent cross section shape and correct segment alignment was crucial to ensure site welding the segments together in Hong Kong proceeded without problems.

d) Heavy Lift

The 88m length of steel deck around each tower is above land and was erected using a heavy lift scheme (Figure 13). In a 4000T lift, the two longitudinal girders were strand jacked simultaneously 75m into their final positions. Due to the tapering tower shape, the two decks were 12m further apart at ground level than in their final positions, so had to be slid transversely once at high level. A 2m longitudinal slide was also necessary to place the decks onto a temporary interface truss before lifting and welding the connecting cross girders, and casting the 2m section to stitch the steel and concrete decks together.

Figure 13 - Heavy Lift

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Figure 14 – Main Span Closure

e) Main Span Erection

Main span deck segments were erected by cantilevering out from each tower (Figure 14). Each 18m long, 53m wide segment comprises the twin deck with connecting cross girder and weighed around 500T. One of the main project constraints was the need to maintain the flow of shipping unhindered by the construction of the bridge. Simulations of shipping movements and measurements of the currents were made. A dynamic positioning barge delivered each segment and used GPS to accurately maintain the position prior to lifting. A rapid lifting speed was key, so the lifting frames at deck level were equipped with high capacity winches which raised each segment 75m into place in around 40 minutes.

Due to the different support conditions there was a geometric mismatch between the lifted segment and the deck cantilever tip, which had considerable transverse sagging. A temporary bowstring prestress arrangement was installed on the lifted segment to manipulate the shape accordingly. Once in place, welding to the previous segment and installation of the stay cables followed. An 8-day target for each cycle was set, meaning that a segment was lifted on either side every 4 working days.

2. FORTH REPLACEMENT CROSSING BRIDGE

The Forth Replacement Crossing, carrying a dual 3-lane motorway, will be built across the Firth of Forth in Scotland to maintain and enhance a vital transport link. The wide estuary will be crossed by a cable stayed bridge with 3 towers and a pair of 650 m main spans. In the centre of each main span the stay cables will overlap to stabilise the central tower, a unique design feature for a bridge of this scale.

The scheme design of the crossing has been carried out by Arup, working as part of the Jacobs Arup joint venture, in accordance with the Eurocodes and project specific design criteria. The structure will provide a fitting 21st century icon, to stand alongside the existing cantilever rail bridge from the 19th century and road suspension bridge from the 20th century, both Grade A listed bridges with historical significance. Figure 15

Figure 15 – Visualisation: Three centuries of engineering in the Firth of Forth

The Firth of Forth is a dramatic estuary which separates the Scottish capital of Edinburgh from the Kingdom of Fife to the north. The downstream crossings of the Forth at Queensferry are a pair of historic bridges, the iconic cantilever rail bridge constructed in the 1880's and the Forth Road Bridge , Britain's first long span suspension bridge, which was opened in 1964.

The replacement bridge will be slightly to the west of the existing bridges, making use of a natural granite outcrop in the middle of the Forth to allow the wide estuary with two navigation channels to be crossed by a cable stayed bridge with a pair of 650 m main spans, with an approach viaduct to the south. The scheme was selected as the one with the minimum impact on the environmentally constrained area.

2.1 Design Development

Three tower cable stay-bridges result in instability of the central tower when alternative spans are loaded with traffic. A number of solutions are possible to overcome this problem as shown in Fig 16. The solutions are :

- Introduce anchor piers
- Have horizontal stabilizing cables connecting the top of the towers
- Have stabilizing cables connecting the top of the towers to the deck/tower interface of the adjacent towers
- Have overlapping cables in the centre of the spans

Another solution is to have stiff towers . The solution of over-lapping cables was first proposed by Prof. Niels Gimsing. This method allows slender towers to be used, and at the outset of the design it was proposed to use overlapping cables as shown in the general arrangement of the bridge in Fig 19. The two cable-stay spans are each of 650m and cross over the navigation channels.

A number of solutions were developed based on various types of towers and type of decks as shown in Fig 17. These designs were developed in sufficient detail and their costs evaluated. Based on cost and aesthetic considerations the mono-pole tower solution was chosen for further development as the Specimen Design.

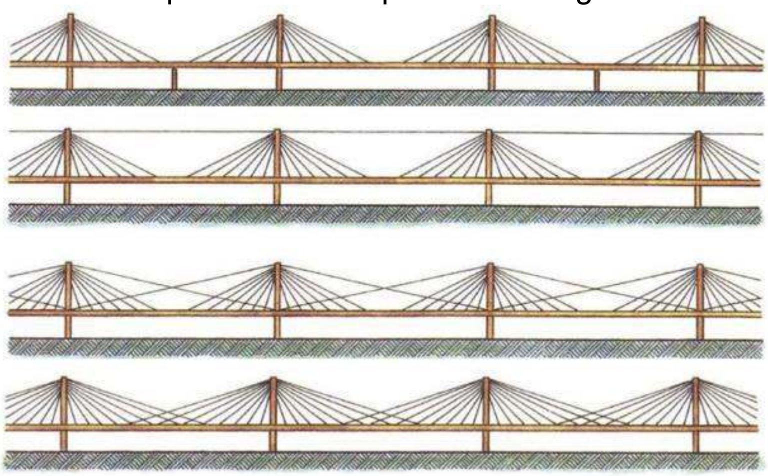

Figure 16 – Methods for Stabilising Towers

Mono-Tower   H-Shape   Diamond   A-Frame

Figure 17 – Tower Forms

2.2 Specimen Design

The Specimen Design of the crossing is a scheme design incorporating a high level of detail . Transport Scotland, the client, wanted to have this specimen design for several purposes: in order to verify the feasibility of the bridge arrangement, to define the overall form and geometry of the crossing, to inform the environmental assessment and the Bill of Parliament, to enable a detailed cost build up to be calculated, and to be a specimen design as a starting point for the tendering contractors from which to prepare their design proposals.

The total length of the bridge is 2,638 m. Although the crossing is divided into a cable stayed bridge and a southern approach viaduct, the structure is continuous from abutment to abutment with no intermediate expansion joints. Longitudinal fixity is provided by a monolithic connection at the Central Tower located on Beamer Rock with transverse support provided at all towers and piers.

The towers are vertical reinforced concrete elements located in the centre of the deck with two planes of stay cables anchored centrally in the “shadow” of the tower between the carriageways. The stay cables overlap in the centre of the main spans. The deck itself is a streamlined box girder and stay cables are multi-strand type.

The key design requirements for the approach viaduct are long spans to minimise environmental impact, and visual continuity with the cable stayed bridge. The aesthetic requirements are achieved by a pair of constant depth box girders supported on V-shaped piers. The transverse separation of the carriageways is constant, and this also suits the road geometry on either side of the main crossing.

During preparation of the Specimen Design it became clear that there was no clear advantage to distinguish between all-steel orthotropic and steel-concrete composite construction for the cable stayed bridge deck box. Therefore both options have been worked up as design solutions, and the contract permits either to be adopted. The heavier composite deck variant has stay cables spaced at typically 16.2 m whereas for the orthotropic variant a typical spacing of 25 m is adopted.

Similarly for the approach viaduct the choice between composite and prestressed concrete construction for the twin boxes was not driven by significant cost difference, so designs for both variants were completed and the option left open. For the composite option, incremental launch construction is assumed, whilst for prestressed concrete option a construction sequence using in-situ balanced cantilevering is assumed.

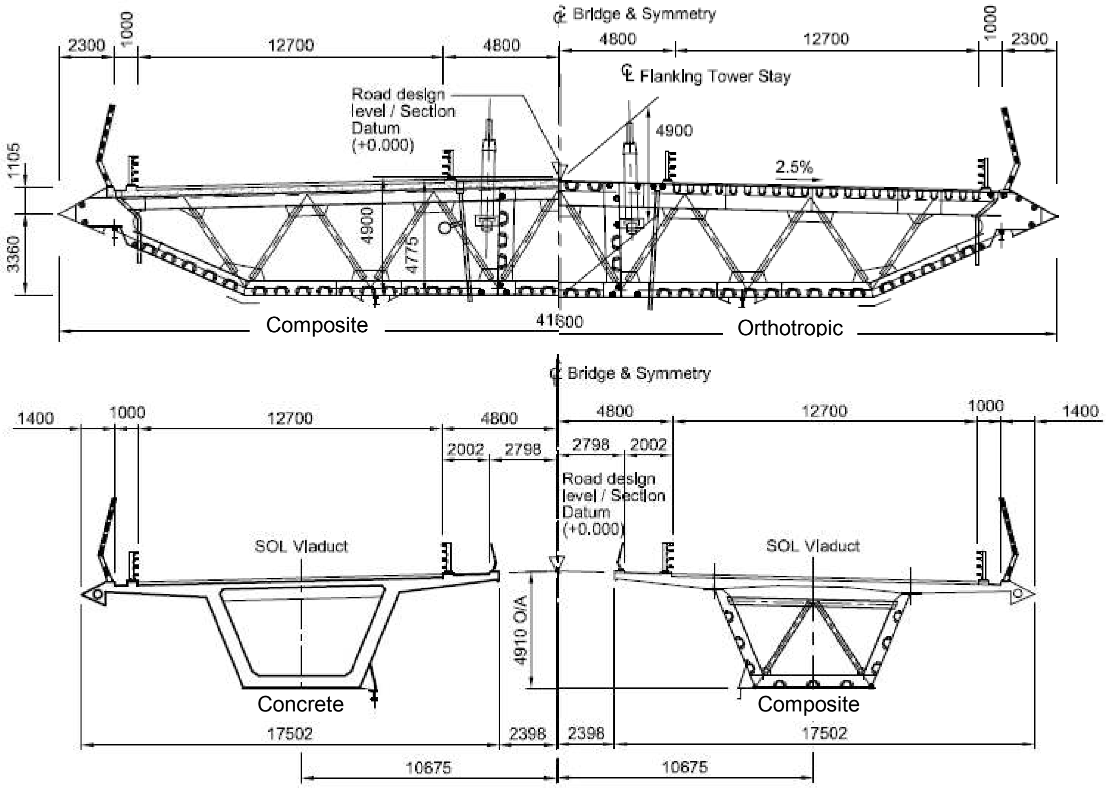

Figure 18 - Deck sections, showing all variants

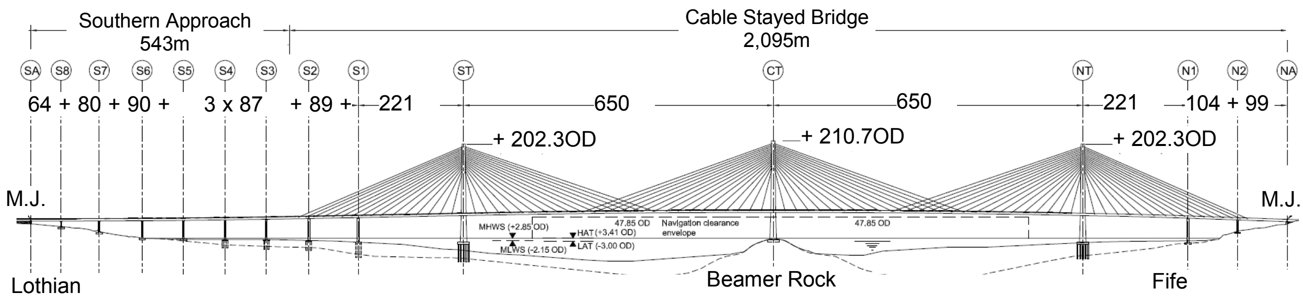

Figure 19 – General Arrangement (orthotropic deck variant)

2.3 Basis Of Design

The Forth Replacement Crossing is one of the first major bridges in the UK to be designed to the Eurocode, implemented in April 2010 as the basis of design for bridges and other structures. Work on the Specimen Design commenced in early 2008 when not all of the UK National Annexes and other implementation documents were available. The design criteria to be used for the structural design are set out in a project specific Design Basis document which has been updated and simplified as more national documents have been published. The final version forms part of the contract, providing additional rules and criteria appropriate to the bridge as well as clarifying how some of the Eurocode rules should be interpreted.

Aspects such as the site specific wind climate and the rules for ship impact criteria have been defined. Historic wind data from measurements on the existing Forth Road Bridge was analysed along with models to account for the local terrain to define the minimum design wind speed and turbulence characteristics. The design mean hourly wind speed at deck level was taken as 31.9 m/s. The studies undertaken on ship impact used a quantitative risk assessment approach based primarily on Eurocode Part 1-7, taking into account the complex navigational conditions in the vicinity of the bridge, with bends in the navigation channels and significant obstructions, not least of which is the existing Forth Rail Bridge

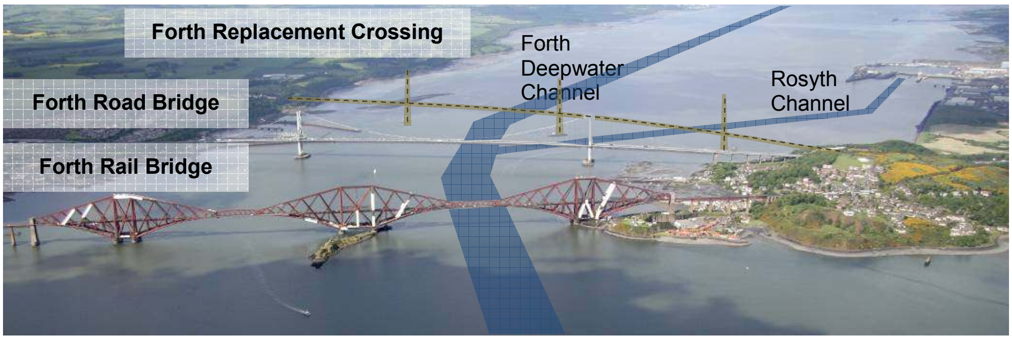

Figure 20 – Navigation Conditions

Due to the marine environment, design for durability requires careful consideration, especially given the unexpected deterioration of the existing road bridge. The choices left open to the contractor for aspects of the concrete mix designs, and the steelwork corrosion protection system are therefore more restricted than in some design and build contracts. Low grade concretes are not permitted, and the minimum cover to reinforcement is specified. Stainless steel reinforcement will be used in outer layers of bars at the base of the towers within the splash zone. The outer surface of steel deck components will be protected with a paint system comprising a zinc-rich epoxy primer, two layers of MIO epoxy, and a polyurethane top coat. The inside of the deck boxes which house the stay cable anchorages will be dehumidified so that future touch up and repainting operations within a confined space will not be required. The upper towers will similarly incorporate dehumidification to protect the steel anchor boxes and stay cable anchorages.

2.4 Analysis And Behaviour

The overall structural analysis was carried out using 3D global computer models. Additional local and semi-local analysis models were established to examine more closely the distribution of stresses and to aid in calibration of the behaviour of the global models.

2.4.1 Global models

Separate models were assembled for each of the main Specimen Design variants to account for the variation in geometry and properties associated with each scheme. The global models encompass the entire length of the crossing from South abutment to North abutment.

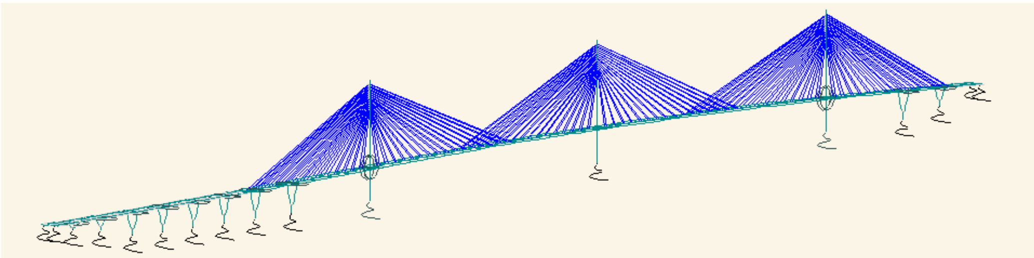

Figure 21 - Global Analysis Model (orthotropic deck variant)

Where a range of values exist for the most appropriate definition of structural properties, sensitivity analyses have been carried out adopting different values. This is the case for the foundation stiffnesses due to variability in ground conditions, and the stiffness of the towers due to potential cracking in the reinforced concrete.

The bridge is subject to movements under different loading conditions. The reference condition is defined as the completed bridge, subject to a uniform temperature of 10°C. It is in this condition that the geometry of the bridge and the road alignment are defined. Therefore the structure has virtually no resulting deformation when loaded with permanent actions and those time dependent effects which have occurred up to the end of construction

2.4.2 Stay Cable tuning

The stay cable forces were determined at the reference condition using an iterative tuning procedure with the objective to minimise the flexural moments in the deck and tower as well as deflections. With a conventional stay cable arrangement, the tuning procedure is relatively determinate given these objectives, as each pair of main span stays carries the vertical component of load for a deck segment, and each side span stay pair balances the horizontal force at the tower. For a bridge with overlapping stay cables, a degree of indeterminacy is introduced in the crossing zones since each support point on the girder is now provided by four rather than two stay cables. The Specimen Design solutions have targeted an approximately equal sharing of the vertical component of load by the stay cables meeting at the common point. Additional fine tuning was carried out subsequently to refine these overall assumptions

2.4.3 Design Effects

The design envelopes of most live load effects were determined using the 3D global model. Particular load cases were investigated in more detail, for example those which maximised the bending effects in the towers were re-analysed including p-delta effects to find the second order moments caused by deflection.

Wind effects were calculated with a buffeting analysis to capture the interaction of the gusty wind and the dynamics of the structure, and this proved to be one of the dominant actions in design.

The effects of ship impact were the subject of detailed investigations, including non-linear analyses calculating the plastic hinge rotations of the piles under the “collapse prevention” criteria.

2.4.4 Semi-local models

FE models with 2D plate elements representing sections of the deck were used to study the effects in the generalised span regions and at the connection of the deck to the Central Tower.

No alterations to the global model were found necessary for overall behaviour, but the effects of shear lag in the span regions result in a small increase in peak stress in the top plate above the stay cable webs. At the Central Tower connection, the expected concentrations of stresses were quantified for critical loadcases to allow the design of this junction to be carried out.

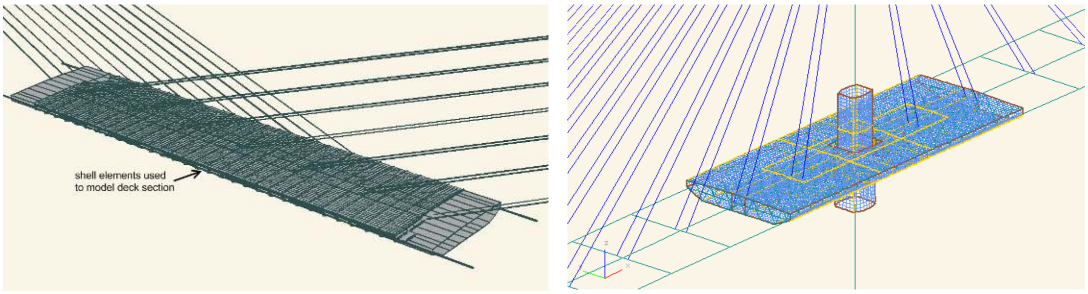

Figure 22 – Semi-local Analyses Models (orthotropic deck variant)

2.4.5 Construction Stages

The construction sequences assumed for orthotropic and composite variants differed so that alternative techniques could be studied. In reality, either of these sequences for the main spans could be adopted for either option:
- balanced cantilever erection to mid-span, resulting in 325 m long cantilevers
- balanced cantilever method to 257 m from all towers, with a heavy lift segment, 136 m long, for the main span closure.

In both cases it was assumed that the installation of the overlapping stay cables commences after the main span has been closed. A total construction programme of around 60 months is expected.

2.5 Wind Tunnel Testing

2.5.1 Preliminary Wind Tunnel Studies

As part of the option selection process, different types of deck sections were tested at 1 to 50 scale to investigate the aerodynamic stability and force coefficients. At the early stages, ladder beam decks were included in the investigation as they may have provided a cost effective solution. Mitigation measures were required to improve stability including edge fairings and partially open central vents. The risk of aerodynamic problems for these decks was reduced, but not eliminated, and they were not progressed beyond the preliminary stage once the box option had been selected.

2.5.2 Wind Shielding Study

Part of the design criteria is for enhanced reliability of the crossing remaining open in strong winds compared with the existing bridge which is subject to frequent restrictions and occasional closures. Wind shielding along the edges of the deck will be provided, but a balance is required to determine the level of protection to vehicles without increasing the forces that the structure must carry beyond reasonable levels. Performance criteria were set to select wind shields which would achieve conditions on the bridge which are no worse than the conditions that would be expected on typical approach roads around the site. A wide range of wind shield geometries were tested on a 1:40 scale model of the deck section. The wind shields selected were 3.44 m high with 6 horizontal slats, each 300 mm high. The wind shields provide the required reduction in moment for a double-deck bus.

2.5.3 Confirmatory Study

Large scale tests of the final deck and tower sections at 1:30 scale were conducted to confirm the aerodynamic stability and the force coefficients. The deck was aerodynamically stable up to and beyond the ultimate design wind speed expected at the site. Cut outs in the corners of the towers were found to greatly improve the aeroelastic behaviour of the towers.

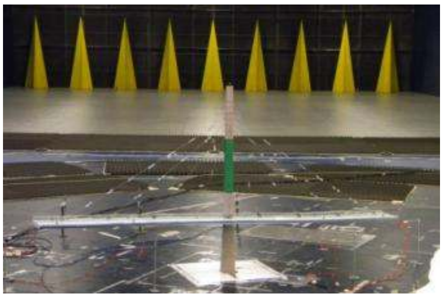

Figure 23 - Wind Tunnel Testing : deck section and balanced cantilever from central tower

2.5.4 Full Aero-elastic Study

A very large 1:170 scale model of the cable stayed bridge was used for the full aeroelastic tests. Key stages of construction were investigated as well as the completed bridge in smooth and turbulent wind flow proving aerodynamic stability up to and beyond the design criteria.

2.6 Verification and Design details

2.6.1 Stay Cables

The sizing of the stay cables is generally governed by the SLS load combination comprising permanent actions together with traffic load model 1 as the leading variable action and wind as an accompanying variable action.

Table 1: Stay Cable data

	Orthotropic Deck Option	Composite Deck Option
Number of Stay Cables	192	288
Cable sizes	56 – 123 strands	42 – 126 strands
Total cable tonnage	3640 T	5670 T

The stay cables will incorporate surface treatment to limit rain-wind induced vibrations, and additional damping has been specified to ensure vibrations from other phenomena are minimised. As both the surface treatment and the details of the damping are dependent on the preferences of the stay cable supplier, a detailed solution has not been presented as part of the Specimen Design. Minimum performance requirements specified in the contract will need to be achieved.

2.6.2 Decks

f) Steel panels

A major part of the deck section checks of both variants of the cable stayed bridge is the verification of slender steel stiffened deck plates. There is a significant transverse stress in the steel plates due to the transverse bending, which for the bottom flange reduces the overall section longitudinal capacity. BS EN 1993-1-5 gives two methods for checking plates. Generally, the effective area method was used to check the capacity of the deck section with an additional reduced stress method check of the individual sub-panels which are subject to significant transverse stress.

The top plate of the deck is typically 14 mm thick with 336 mm deep 8 mm thick trough stiffeners detailed for fatigue loading. The bottom plate varies in thickness along the bridge from 10 mm to 24 mm to account for the different load effects at different sections. Typical stiffeners are troughs 314 mm deep and 6 mm thick. Diaphragms are formed as trusses rather than plates as this was found to be more economical and provides a much more open space within the deck. For the orthotropic deck, diaphragms are spaced at 5 m, whereas for the composite deck a spacing of 4.05 m is adopted, in each case to provide 4 sub-panels between each stay cable support point.

g) Concrete slabs

Since the deck elements have significant compression forces, the effects of slenderness are important. The global axial compression can induce additional moments in the concrete slab due to initial imperfections. Further moments can be induced due to deflection of the slab under local wheel loads and long term displacements of the slab due to creep. The Eurocode gives three methods for checking the second order effects - nominal stiffness, nominal curvature and the general method. It was found that the nominal stiffness and nominal curvature methods were very conservative and there is sufficient benefit, in terms of reinforcement quantities, to perform a refined analysis using the general method. The composite option incorporates transverse prestressing in the deck slab to prevent cracking and maintain the torsional stiffness of the box.

2.6.3 Towers

The analysis and subsequent design was carried out using the general method in BS EN 1992. The effects of geometric imperfections were assessed by analysing the tower with a deformed shape equivalent to the first bucking mode. The magnitude of the deformation was equal to the maximum construction tolerance estimated to be no more than 200 mm at the top of the tower. The effect of imperfections was found to be very small compared to the static deflections due to wind load.

2.6.4 Piers

The piers to the main bridge side spans and approach viaducts are conventional reinforced concrete elements. A V-shape has been chosen to support the decks while allowing a single foundation at each pier location. Steel tie-beams are provided between the pier legs at pier head level.

2.6.5 Approach Viaducts

The overall scheme has been sized to maintain the same structural depth for the approach viaduct as for the cable stayed bridge deck, and both variants for the twin box girders have the same external shape. The composite boxes have cross frames typically every 8 m, with stiffening ring frames in between these. Trough stiffeners similar to those in the main deck are used. The concrete boxes consist of segments between 3.6 m and 4.0 m long, depending on the length of individual spans.

2.6.6 Foundations

The 3 main towers generate the highest foundation loads. There are in addition 10 side span and approach viaduct piers with smaller loads, seven of which will be within the estuary and the remainder on land. In addition to service loads, the foundations within the estuary are required to resist accidental ship impact. Ground conditions vary considerable across the site.

Beamer Rock is a dolerite outcrop which will support the Central Tower. A pad foundation with overall dimensions of 35 m by 25 m has been designed to be recessed into the rock. Construction works within the estuary can be minimised by using a cellular structure cast off site. After blasting the rock and positioning the foundation, the cells would be infilled to complete the base.

For the flanking towers, piled foundation solutions have been designed incorporating 3.4 m diameter piles. The South Tower will sit in about 22 m of water, with rockhead at around -40 mOD. At the North Tower the water depth is around 5 to 8 m, and the rockhead at typically -34 mOD.

CONCLUSIONS

The Stonecutters Bridges along with Forth Replacement Crossing show that a cost-effective and elegant design can be obtained via a design competition route or by selection of a design team through competitive dialogue

ACKNOWLEDGEMENTS

The architectural consultant for both Stonecutters Bridge and Forth Replacement Crossing Bridge is Dissing + weithing.

This paper has been published with the kind permission of the Director of the Highways Department Hong Kong and the Project Director for the Forth Replacement Crossing, Transport Scotland.