Forth Replacement Crossing

Main Crossing (Bridge)
Scheme Assessment Report
Development of Options

Report on Scheme Development
January to August 2008

Contents

Bibliography and Workstream Timeline

As highlighted below, this is the fourth of a series of reports which cover the project development work carried out during 2008, following completion of the Forth Replacement Crossing Study during 2007.

Ref	Report Title and Work Period	Report synopsis
1.	*Forth Replacement Crossing Study Report 5: Final Report* *Work pre-June 2007.*	Report on work undertaken by Jacobs and Faber Maunsell to June 2007 to assess the options for a replacement crossing which recommended that a cable stayed bridge in 'Corridor D' – a crossing point immediately upstream of the Forth Road Bridge - be taken forward as the best overall performing option.
2.	*Forth Replacement Crossing Route Corridor Options Review:* *Work carried out by Jacobs Arup, January to May 2008.*	Report to assess 9 mainline connecting road corridors: three in the Northern Study Area and six in the Southern Study Area. It recommended that two of the northern and two of the southern corridor options be taken forward for further assessment.
3.	*Forth Replacement Crossing DMRB Stage 2 Corridor Report:* *Work carried out by Jacobs Arup, May to August 2008.*	Report on the assessment of the shortlisted corridor options and a supplementary assessment of a variant version of a connecting road corridor in the Southern Study Area. The report recommended that work continue to identify in detail the optimum road improvement within Corridor Option 1 North and Corridor Option 1 South.
4.	*Forth Replacement Crossing, Main Crossing (Bridge) Scheme Assessment Report, Development of Options:* *Work carried out by Jacobs Arup, January to August 2008.*	Report on the assessment of options for the outline design of the replacement crossing.
5.	*Forth Road Bridge – Feasibility of Multi-Modal Corridor:* *Work carried out by Jacobs Arup, August to October 2008.*	Report on the feasibility of utilising the existing Forth Road Bridge for non motorised and public transport/light road traffic, including for a potential future guided bus/tram/ light rail facility. The report concluded that this would be a feasible option.
6.	*Forth Road Bridge - Audit of Feasibility of Future Multi-Modal Use - Summary Report* *Work carried out by Faber Maunsell to November 2008*	Independent summary of review on the Jacobs-Arup assessment of the feasibility of utilising the existing Forth Road Bridge for non motorised and public transport/light road traffic, including for a potential future guided bus/tram/ light rail facility. The report concluded that the Forth Road Bridge could, in principle, be adapted for future LRT
7.	*Forth Replacement Crossing, Main Crossing (Bridge) Scheme Assessment Report, Development of D2M Alternatives:* *Work carried out by Jacobs Arup, October to November 2008.*	Report on the assessment of options for a narrower replacement crossing to carry a dual carriageway road with hard shoulders.
8.	*Forth Replacement Crossing, Scheme Definition Report.* *Work carried out by Jacobs Arup, July to November 2008*	The final report on the project planning work carried out during 2008 which provides recommendations of the road connections and the incorporation of the Forth Road Bridge as an integral element of the proposals for use by pedestrians, cyclists, public transport and any future multi-modal facility.

1 Introduction

The existing Forth Road Bridge forms a key link in Scotland's transport network. The crossing currently carries some 66,000 vehicles per day which includes over 70 percent of travellers across the three Forth bridges (Kincardine, Forth Road Bridge and Forth Rail Bridge).

In 2007, the Employer – the Scottish Ministers – announced that a Replacement Forth Crossing would be promoted by the Scottish Government. Previous work undertaken by Transport Scotland included consideration of alternative corridors and structures for the Replacement Forth Crossing and on 19 December 2007, the Scottish Ministers announced that the Replacement Forth Crossing would cross the Firth of Forth immediately upstream of the existing Forth Road Bridge and would be a cable-stayed bridge.

The Jacobs-Arup commission is for the management and delivery of the Replacement Forth Crossing Project inclusive of all roads and other infrastructure associated with such a structure.

This report outlines the development and assessment of Scheme Options for the Main Crossing. The preparation of this report has been carried out in association with:

- Dissing + Weitling
- Flint & Neill
- Professor Niels Gimsing

1.1 FRCS Reference Design

A reference design was developed during the Forth Replacement Crossing Study which is documented in Report 4: Appraisal Report, Appendix C - Bridge at Corridor D. Two options were considered during the FRCS, a 1300 m main span suspension bridge and a cable stayed bridge with two main spans, each of 650 m. The cable stayed bridge option was carried forward as the reference design for the Forth Replacement Crossing.

Drawings illustrating the Reference Design are included in Appendix A.

1.1.1 General Arrangement

The bridge has three towers with the central tower located on Beamer Rock. The 650 m southern main span places the south tower clear of the Forth Deep Water Navigation Channel in approximately 20m depth of water. A symmetrical arrangement results in a 650 m northern main span placing the northern tower well clear of the Rosyth Navigation Channel and in approximately 10m depth of water. 325 m side spans are adopted equal to half of the main span length. Each side span includes one additional anchor pier to provide additional stiffness.

A 635m long southern approach viaduct connects the cable stayed bridge to the south abutment via a nine span structure. The northern approach viaduct is significantly shorter at approximately 115 m long and is a two span structure.

The deck superstructure is a single cell orthotropic steel box girder with the stay cables provided in a fan arrangement and anchored along both edges of the bridge deck.

Each tower consists of a pyramid with four concrete legs extending above deck level, joined together in the zone where the stay cables are anchored. The spacing between the legs of the central tower in the longitudinal direction is greater than for the flanking towers. The shape has been developed in response to the requirement to provide additional stiffness for a double main span cable stay bridge.

The foundations shown for the main bridge towers are large caissons. For the central tower the caisson is founded on Beamer Rock which is to be levelled at the start of construction. For the flanking towers the caissons are founded at approximately 40m below water level on the sandstones and mudstones below the soft alluvial and glacial deposits in the Firth of Forth.

1.2 Development of Scheme Options

A Concept Design workshop was held from the 11th to 15th February 2008 in Transport Scotland's offices to develop a short list of design concepts that would be taken forward for further development. The workshop included a site visit and a number of technical briefings (geotechnical / environmental / alignment / structural) to provide the background data necessary for concept development. Architectural visualisations were developed illustrating, in a series of photomontages, the basic concept of the design options to be carried forward.

Concept Design Options

Subsequent to the design workshops, further analysis and investigation has been carried out to develop the short listed design concepts into a number of Scheme Options. The options can broadly be considered in terms of:

- **Functional Cross Section** – What the bridge is required to carry and how that will be arranged on the deck in terms of location of traffic lanes etc.
- **Deck Type** – The construction material and structural arrangement of the deck.
- **Tower Form** – The appearance of the towers which will be the major aesthetic impact of the bridge.
- **Approach Bridge Type** – The construction material and structural arrangement of the approach bridge.
- **Foundation Type** – The construction form of the foundations.

2 Description of Scheme Options

2.1 Functional Cross Section

The functional cross section of the bridge is required to incorporate the following items:

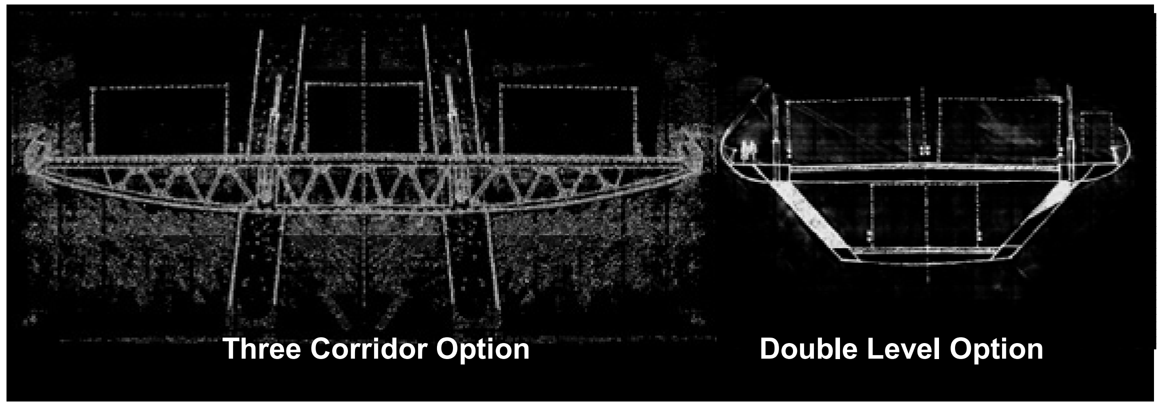
- Dual two lane motorway with 3.3m hard shoulders (70 mph design speed)
- Footway / cycleway with provision for maintenance vehicle access
- Multi-modal (public transport) corridor (70 mph design speed)
- Vehicle restraint systems and pedestrian / cyclist parapets
- Windshields (designed to be difficult to climb)
- Highway lighting

At the concept stage, a number of alternative configurations for the functional cross section were considered and assessed against a range of criteria including:

- Road connectivity
- Multi-modal connectivity
- Operational considerations
- Tower aesthetics
- Foundation costs

Two configurations were selected for further development

- Three Corridor Option
- Double Level Option



Functional Cross Section Options

The two options are illustrated in Drawings FRC/C/076/S/021 and FRC/C/076/D/031 contained in Appendix B. For the Three Corridor option the bridge deck is a single level with the multi-modal corridor located in the centre of the bridge. Stay cables and tower legs are located in a structural zone between the multi-modal corridor and the main motorway carriageways. For the double deck option the multi-modal corridor is located at a lower level with stay cables anchored close to the edge of the deck. In both arrangements the footways / cycleways are located at the edges of the deck.

2.2 General Arrangements

The general arrangement of each option has been developed to be largely consistent with the FRCS Reference Design with the bridge having two cable stayed main spans, each of approximately 650 m and three towers with the central tower founded on Beamer Rock. The general arrangements are illustrated in Drawings FRC/C/076/S/001 and FRC/C/076/D/011 contained in Appendix B.

A fundamental consideration for the general arrangement of the bridge is the provision of two cable stayed main spans. This arrangement requires special consideration of how to stabilise the central tower. Appendix D describes the studies carried out during the conceptual design development to select two different options for central tower stability corresponding to the Functional Cross Section options:

Functional Cross Section	Central Tower Stability
Three Corridor	Achieved by crossing cables in the centre of the main spans
Double Level Deck	Use the combined deck and tower stiffness to achieve sufficient static and dynamic global stiffness.

2.3 Deck Type

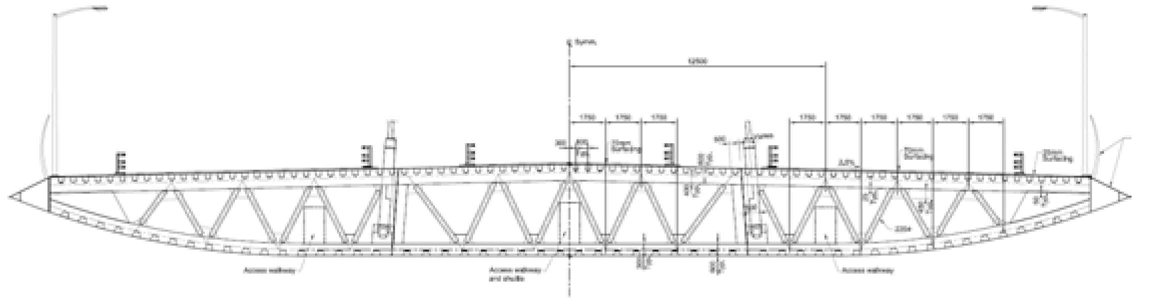
2.3.1 Three Corridor option

A key driver for the Three Corridor option is the requirement for the deck to be torsionally stiff due to the significant span of the bridge combined with the stay cables being anchored relatively close to the centre of the deck. The only feasible deck in this case is a box girder. Two alternatives are considered which are illustrated in Drawing FRC/C/076/S/101 in Appendix B:

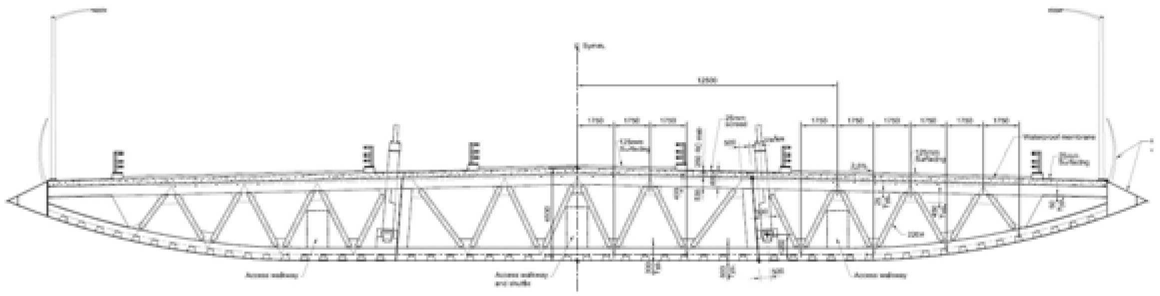
- Steel orthotropic box girder
- Steel-concrete composite box girder

Considering past practice it is known that composite decks can be more economical for short to medium span cable stay bridges but become uneconomical at long spans. The current longest composite span is the recently completed Qingzhou cable-stayed bridge over the Ming River, Fuzhou, China with a span of 605 m. The 650 m span proposed would therefore be a world record and is at the upper end of the boundary between medium span and long span by this definition. It is anticipated that the relative economy of the deck types will be marginal and will be determined by market conditions, material costs at the time of construction and the preferred working arrangements of the tendering contractors.

Since the two deck options will be aesthetically the same the aim of the design is that both types be developed. Provided that during the design development neither deck type is proven to be unfeasible it could be considered to tender the project on both deck types, allowing the tendering contractors to select the one for which they are able to provide the most competitive price for. If this strategy is followed then the tower design should be developed to have the same external shape for both deck options although the wall thickness and other internal details may vary.



Orthotropic Box Girder Section



Composite Box Girder Section

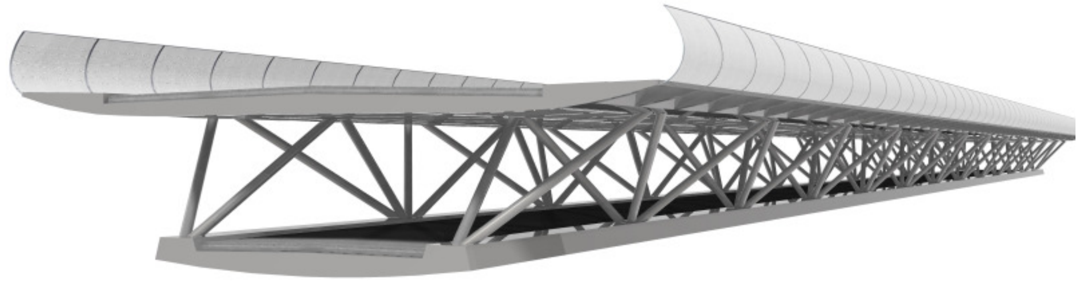
2.3.2 Double Level option

The double level options utilise a deep stiffening truss that assists with stabilising the central tower. The key driver for the overall behaviour of the suspended decks is the relative stiffness of the tower and the deck. The deck must be stiff enough to accommodate the deformations at mid-span under asymmetric live loading and reduce bending effects in the tower such that the tower can be kept relatively slender and elegant. Three alternatives are considered which are illustrated in Drawings FRC/C/076/D/111 to FRC/C/076/D/113 in Appendix B:

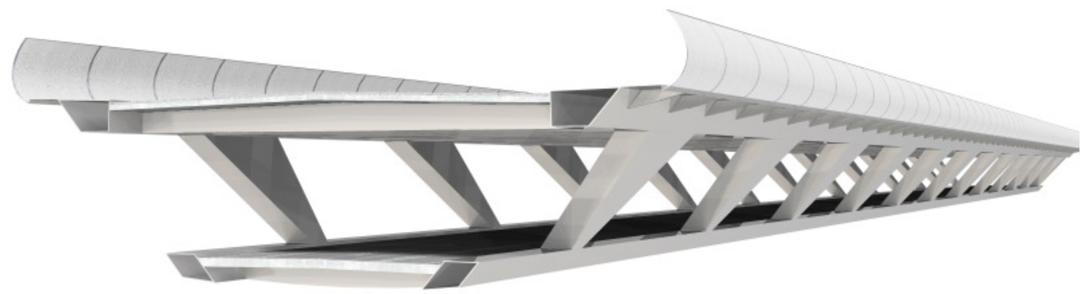
- 2 Plane Warren Truss
- 4 Plane Warren Truss
- 2 Plane Vierendeel Truss

The logical truss arrangement is a Warren truss which is more elegant than a Pratt truss. The shear forces are also reversible in most parts of the deck which means that structurally the Pratt truss is not particularly relevant since its defining feature is that the bracing arrangements relate to the direction of the shear force. Providing two planes of trusses, one beneath each plane of cables, creates a torsionally stiff and robust structure. However an alternative with four truss planes is also considered in order to triangulate the transverse span to reduce the cross beam depth and also reduce the section size of the bracing members.

The Vierendeel truss alternative is proposed in order to create a visually less complex structure since the bracing members of the Warren truss are inclined in two different directions and result in possible visual interference effects when viewed from certain angles. However, it is well established that Vierendeel trusses are less efficient than triangulated trusses and the feasibility of this proposal has been carefully studied.



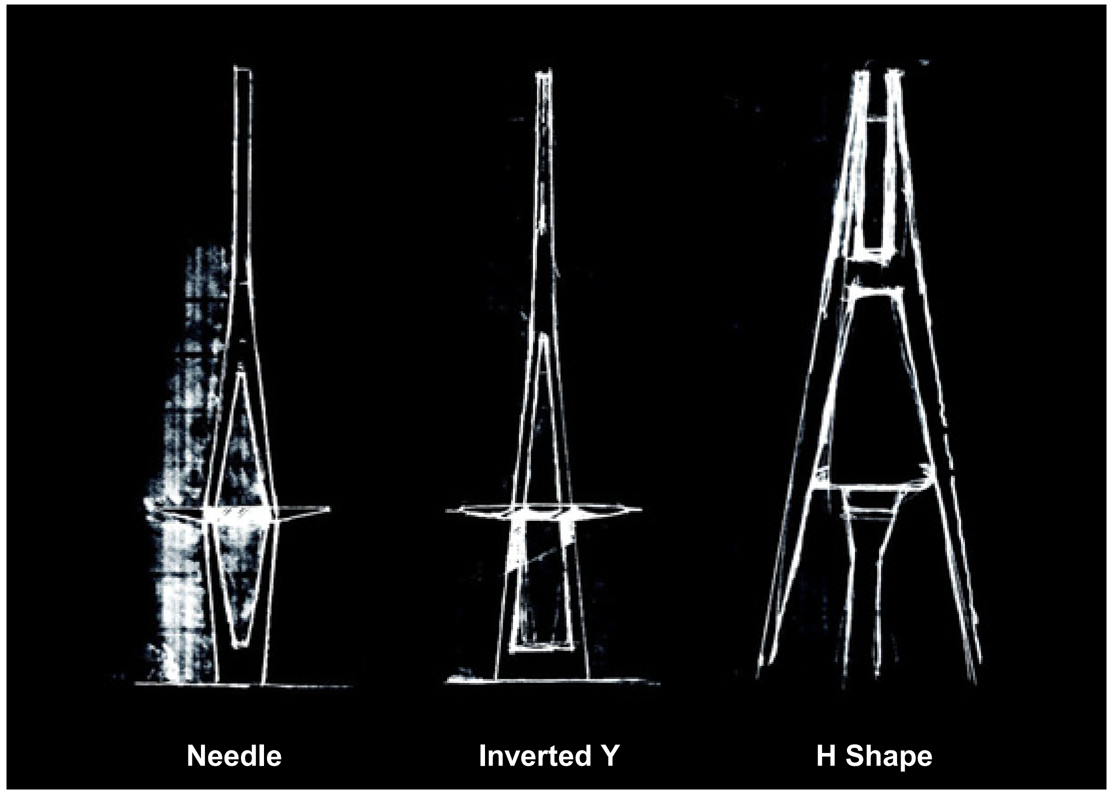
4 Plane Warren Truss



2 Plane Vierendeel Truss

2.4 Tower Forms

Three alternative tower forms have been developed, in each case the tower is a reinforced concrete hollow structure with a fabricated steel anchor box to house the upper stay anchors. Provision is made within the towers for access during construction and for inspection and maintenance.



Tower Forms

Each tower form is to be considered with a particular functional cross section as tabulated below:

Tower Form	Functional Cross Section
Needle	Three Corridor
Inverted Y	
H Shape	Double Level

2.4.1 Needle

The concept of the Needle tower is to have a single vertical element centrally located but with a hole punched through it to allow the multi-modal corridor to pass. The aesthetic development of this option has therefore focussed on developing the tower to be a single object rather than a collection of legs, base and anchor box.

2.4.2 Inverted Y

Above deck level, the Inverted Y tower is somewhat similar to the Needle tower with split legs located towards the middle of the deck with the multi-modal corridor passing between the legs and the main motorway carriageways outside. However, the aesthetic concept and practical considerations on foundation size lead to slightly different proportions above deck compared to the Needle tower.

2.4.3 H Shape

In concept, the H Shape has two legs which are inclined towards each other and connected in the region of the stay anchorages without any cross beam below deck level so that the deck floats through the tower. The aesthetic development of the concept has studied the inclination of the legs to achieve reasonable aesthetic proportions and foundation size as well as the number and composition of the cross beams. Due to the relative complexity of the truss deck form associated with this option, the tower itself is developed to be simple in form.

2.5 Approach Bridge Type

The Reference Design shows the southern approach viaduct to be 635 m long, consisting of 9 spans with a maximum span of 80 m. The northern approach viaduct is 115 m long and is a two span structure.

The key issues for the further development of the approach viaducts are visual continuity with the cable stayed bridge and long spans to reduce the numbers of piers to be constructed in the environmentally sensitive channel and inter-tidal zone.

Three types of approach bridge are being considered:

Approach Bridge Type	Functional Cross Section
Composite Box Girder	Three Corridor
Concrete Box Girder	
Truss	Double Level

For the Three Corridor Option visual continuity is best provided by a composite approach bridge with a single wide box girder of the same basic cross section as the cable stayed bridge deck. However, this may be a relatively costly solution and since the approach bridge is a significant structure in its own right a concrete alternative has been developed with three box girders.

For the Double Level option the only solution that can provide visual continuity is to continue a truss of similar form to the cable stayed bridge deck into the approach viaducts. This has the advantage of offering long spans and thus reducing the number of piers.

2.6 Foundations

The ground conditions have yet to be determined at all foundation locations and, together with water depths, will vary along the crossing alignment. These, as well as constructability, will be key drivers for the selection of foundations schemes. The side span and approach span pier locations will depend on the structural form adopted for the deck, whilst the foundation geometry at the main towers will depend on the tower form selected. Consequently there are a range of conditions and foundation solutions that may be appropriate which will require further investigation and development in the next stage.

The most challenging foundations will be those for the main towers. The south tower foundation will be in the deepest water and, with 650 m main spans, will be located where the river bed level is about -22 mOD. The ground conditions are currently uncertain but it is anticipated that there will be a substantial thickness of soft alluvium overlying variable glacial deposits before reaching bedrock. This foundation will also have the most onerous ship impact loading. The FRCS Reference Design shows caisson foundations taken to rockhead at the main towers. However, based on the information currently available these would be difficult to construct at the south tower.

Preliminary studies have shown the alternative of a piled foundation to be feasible. This would comprise a group of large diameter piles socketed into the rock. It is envisaged that a precast pile cap could be used and that techniques developed in the offshore industry could be adopted for forming connections between the piles and pile cap underwater.

A similar piled foundation with precast cap may also be suitable for the north tower but would require some initial dredging to install the pile cap. Alternatively in situ pile cap construction within a temporary cofferdam could be considered.

The central tower foundation will be located on Beamer Rock and it is expected that a spread foundation will be suitable for this location. The overall sizing of the foundation will depend on the form of tower selected.

The side and approach spans will require foundations in varying depth of water or on land with rockhead at varying level below bed level. Either piled or pad foundations may be appropriate depending on the conditions at each location.

3 Key Issues and Assumptions

3.1 Key Issues

The scheme options have been assessed with respect to the key issues and criteria which govern the overall design of the bridge. The major design objectives are to provide an elegant, unique and instantly recognisable structure which is durable and straightforward to maintain. With the procurement programme being a key concern for delivery of the project, constructability is also very important as this will lead to a reduced construction period as well as reduced costs.

Aerodynamic stability is an issue whose importance for any long span structure has been well established. Whilst a programme of wind tunnel testing is required to fully investigate these phenomena, the use of correlations established from previous tests as well as computational modelling has been used to provide preliminary guidance on performance.

An unusual structural feature of the Forth Replacement Crossing is the provision of double main spans which are inherently less stiff than a traditional system where the pair of towers which flank the single main span is anchored back to ground by the stay cables in each side span. In this case the central tower has no back stays and may deflect significantly under asymmetric loading of the bridge resulting in relatively large deck deflections as well. This bridge will include the world's longest multiple cable stayed spans and developing an understanding of the flexibility associated with this structural system as well as appropriate mitigation measures has been influential in the structural design. This is described in more detail in Appendix D.

The bridge crosses a navigable waterway and maintaining safe navigation clearance at all times governs the vertical alignment of the bridge. Furthermore the potential consequences to the bridge due to errant ships impacting the towers and piers are critical to the design of the lower sections of the towers and the foundations. The possibility of subsequent explosions and pool fires if the ships contain hazardous flammable materials is also being studied.

3.2 Consideration of longer main spans

The general arrangement of two main spans each of 650 m, centred around Beamer rock, means that the location of the south tower is in relatively deep water. Bed level is approximately -22 mOD, and bedrock is tentatively assumed at approximately -55 mOD although the tower is beyond the extent of previous borehole investigations so this level is uncertain. More accurate levels will be determined as part of the ongoing marine ground investigation. The depth to rock is expected to lead to a substantial piled foundation at the south tower.

Studies have been carried out to investigate whether it is feasible to locate the south tower further south and increase the main span lengths. The potential benefits are a saving in the foundation costs which could arise from two improvements to the foundation conditions:

- Bedrock level being not as deep (to be assessed by marine ground investigation)
- Reduction in design ship impact force due to greater distance from the typical vessel transit paths (to be assessed by ship impact investigation)

There would also be further benefits arising from a reduction in the number of approach span piers in the water. However, the superstructure costs would be increased. An assessment of the overall merit is required once the investigations that determine the potential benefits are completed. It is not obvious whether longer spans will prove to be of benefit and therefore the assessment of scheme options at this stage has been carried out on the basis of 650 m spans. However the general arrangement of potential longer span bridge configurations is illustrated in Drawings FRC/C/076/S/002 and FRC/C/076/D/013 contained in Appendix B.

3.3 Stay Cables

Two different types of stay cable system are suitable for large cable-stayed bridges: parallel wire cables or multi-strand cables. Alternative cable types of locked coil strand or spiral strand are not appropriate due to their poor fatigue performance, low stiffness and lesser ultimate tensile strength (in typically manufactured cable sizes).

Parallel wire cables have a very compact cross section and are factory manufactured to the specific lengths required. Galvanised wires are arranged into the required pattern, and a polyethylene sheath is extruded onto the outer surface. The cables are wound onto reel and transported to the bridge site, where substantial lifting equipment is required to handle them. Very large jacks are needed to stress the cables. Cable length adjustment can be made with either shim plates, or a large nut on the threaded portion of the cable socket, depending on the system adopted.

Multi-strand cables are assembled on site. After the cable sheath is placed between the tower and deck anchorages, individual strands (each consisting of 7 galvanised wires within a polyethylene sheath) are fed through and secured using wedges at each end. The diameter of the cables is larger than for parallel wire cables of the same capacity, as each strand has its own corrosion protection sheath, and spare space is required within the outer sheath to allow strand installation. Stressing of individual strands can take place using small stressing equipment to adjust the lengths, and care must be exercised to ensure an even force distribution between all strands. Any de-stressing must be done using a large stressing jack to adjust a nut on the anchor so as to avoid disturbing the wedges holding the individual strands.

Cable replacement for parallel wire cables involves removing the entire cable, and replacing it with another one. Large lifting and stressing equipment is required. For multi-strand cables it is possible to withdraw, inspect and replace individual strands by reversing the assembly method. Although still a major operation, it can be performed using relatively small equipment and without major disruption to operation of the bridge. In practice it may be that once the cables have reached the end of their design life removal of the entire cables may be required which would involve similar procedures as for the parallel wire cables. Nevertheless, the ability to inspect individual "witness" strands at periodic intervals is a definite advantage.

The compact nature of the parallel wire cables enables equipment to be clamped onto the cables at any location along its length. If either cross–ties to link stay cables together, or external damping devices are required to limit unforeseen vibrations, or if street lighting equipment is to be suspended from the stay cables this can be an advantage. For multi-strand cables, provision for these types of equipment must be planned in advance of installing the stay cables and a special fixing point formed in the outer sheath.

At this stage, multi-strand cables appear favourable, due to long term inspection and replacement considerations. As the cable diameters, and therefore wind loading, are larger for this system, designing the structure accordingly does not preclude the use of parallel wire cables if they prove more advantageous. For example it could be considered to tender the project allowing either stay type in order to obtain the most competitive price.

A maximum cable size of 127 strands has been assumed, as although some cable manufacturers include larger cable sizes in their literature, experience and suitable equipment for fatigue testing and installation is extremely limited. There are a number of manufacturers that have a stay system with this size as their limit. If larger sizes are demanded there may be a restriction to competition.

3.4 Ship Impact

The bridge crosses a navigable waterway with approximately 5,500 significant vessel transits per year in the Forth Deep Water Navigation Channel travelling to and from Grangemouth and other upstream ports. Vessels up to 39,000 DWT pass under the bridge but the number of passes of such large vessels is very low. Over half of the vessel traffic is less than 6,000 DWT and only 1% of the traffic is larger than 20,000 DWT. The Rosyth Navigation Channel also passes below the northern main span of the bridge but the volume of shipping using this channel is an order of magnitude lower than the Forth Deep Water Navigation Channel and the subsequent risk from ship impacts is also very low.

The importance of ship impact loads for the design of the foundations was recognised during the Setting Forth studies which recommended a design ship impact load of 130 MN based on a 33,000 DWT ice strengthened tanker travelling at 12 knots. A force of this magnitude would govern the design of the foundations and would require significantly more piles than are needed to resist the ordinary in-service loads of self-weight, traffic and wind.

Considering the very low volumes of large ships it is possible that a statistical analysis could conclude that the probability of a large vessel striking one of the towers or piers at full speed is extremely low and therefore acceptable such that the design ship impact scenario could involve a smaller, more typical, vessel and/or travelling at a lower speed. The American design standard AASHTO provides a detailed and prescriptive methodology for carrying out such a statistical analysis which would result in a design impact load of approximately one third that recommended by the Setting Forth studies.

However, some of the target criteria, correlations and formulae used by the AASHTO method are superseded by guidance in the Eurocode and recent research. On the other hand, the Eurocode does not provide a prescriptive methodology for the statistical analysis. A project specific statistical methodology is currently being developed which would be compliant with Eurocode but may include some of the statistical components of the AASHTO methodology where they are believed to be relevant.

An important component of the statistical analysis is the probability that a ship will lose control in the vicinity of the bridge. Loss of control can be contributed to both human error and mechanical failure and the probability of these incidents occurring can be significantly reduced by piloted and tug-assisted vessels. Discussions have been held with Forth Ports which indicate that high rates of pilotage and tug-assistance are expected for the larger vessels which should be included in the statistical analysis.

From the preliminary results of the statistical study it is believed that the design ship impact force will be lower than that estimated by the Setting Forth studies and can be accommodated by moderate strengthening of the foundations compared to those required to resist in-service loads and the assessment of scheme options has been carried out on that basis. Furthermore, slender tower elements near the waterline have been considered unacceptable due to excessive vulnerability to large ships travelling in ballast at high states of tide.

3.5 Other Issues

3.5.1 Surfacing Thickness

The road surfacing system adopted will depend on the structural nature of the deck. Generally thinner surfacing is used for steel structures compared to concrete or composite structures because of the significant weight saving and hence reduction in structural quantities. In the past very thin surfacing systems have been adopted in the UK with a 38 mm mastic asphalt system being used on a number of steel bridges but this has in some cases resulted in poor ride quality and difficulties in maintaining the system. If an orthotropic steel box girder is adopted, a surfacing thickness of approximately 70 mm will be suitable on top of the stiffened steel deck plate which is consistent with European practice for steel bridges. This thickness will result in reasonable ride quality and allow the upper wearing course to be replaced without disturbance to the lower base course. 70 mm of surfacing also allows a 2 mm reduction in the deck plate thickness compared to thinner surfacing due to composite action in reducing fatigue stresses. An assessment will be made of the most suitable material to use considering either Gussasphalt, mastic asphalt or epoxy asphalt systems on top of the waterproofing layer.

For a concrete deck slab, as would be adopted for a composite deck solution or a truss solution, the weight penalty associated with thicker surfacing is proportionally less and a standard 125 mm surfacing layer has been assumed in this assessment (hot rolled asphalt or stone masic asphalt weairing course with appropriate base layer). This may result in a slightly better ride quality and more standard maintenance and replacement procedures.

3.5.2 Vehicle Restraint Systems and Parapets

Along the edges of each carriageway, vehicle restraint systems will be provided in accordance with the relevant standards. A zone immediately behind each barrier will be kept free of any structural components, so that in case of an accident which leads to deformation of the barrier, the risk of a vehicle striking the structure is extremely small. Nevertheless, vehicle impacts on the structure will be considered in the design. The barriers systems adopted will have been proven to comply with the relevant standards and appropriate limits of deformation.

The barriers will provide segregation between the traffic and the pedestrians / cyclists. At the outer edges of the walkways, pedestrian parapets will provided. These could be combined with the wind screens to make use of the same supporting structure.

3.5.3 Windshields

Due to the critical function of the bridge as a key link in Scotland’s transport network, it is important that it remains operational at all times for traffic use. The exact criteria for the maximum wind speeds across the carriageway will be defined as part of a study of other major bridge crossings, and research into the effects of gust wind on road and light rail vehicles.. The criteria will need to be met under all wind conditions when traffic can still use other parts of the network such as the approach roads leading to the bridge. Wind screens will be provided on the bridge to achieve this.

The three corridor cross section will require windshields at each edge of the deck, but as the section is very wide, it may be necessary either for these to be very high, or to have additional wind screens surrounding the multi-modal corridor in order to provide suitable wind shielding across the full width of the deck. For the double level cross section, windshields will be required on the edges of both the top deck and probably also the bottom deck.

Additional windshields may be required along short lengths close to the towers, where sudden changes in cross wind can occur due to the shielding nature of the tower structure.

The windshields along the edges of the deck will be designed to be difficult to climb over.

Possible layout of anti-climb windshields

4 Assessment of Functional Cross Section Options

4.1 General considerations

The bridge design must allow for the multi-modal corridor to be used for trams or LRT systems in the future. However, as there are currently no details for the installation of such a system the bridge design must also provide for the corridor to be used in the interim.

Initial concept development assumed that the interim use of the corridor could be either as High Occupancy Vehicle (HOV) or bus lanes. It is now proposed that only HOV use of the multi-modal corridor should be considered. The hardshoulders are widened to 3.3m compared to the 2.75m required for a standard D2UM cross section in order to allow buses to use the hardshoulders during peak hours.

Whilst for bus usage an unsegregated highway cross section on the multi-modal corridor was assumed this is not considered adequately safe for HOV usage. Therefore the HOV lanes will be separated by a central reserve incorporating a VRS.

Locating the footway / cycleway adjacent to and on the same level as the main carriageways was seen to be advantageous since it allows the footway to be used as a refuge for passengers of broken down vehicles as well as providing greater security for users of the footway since they are clearly visible to motorists and the risk of assault is reduced.

4.2 Assessment of options

At the concept design stage the functional cross sections were assessed against a range of criteria comprising:

- Road connectivity
- Multi-modal (Bus/HOV) connectivity
- Multi-modal (Tram/LRT) connectivity
- Operational access to multi-modal (Tram/LRT)
- Traffic crossovers
- Tower aesthetics
- Foundation costs

Both of the selected functional cross sections provide good operational access to the multi-modal corridor with both tracks of the Tram/LRT located adjacent to each other and with sufficient space for maintenance walkways adjacent to the tracks.

The Double Level option has the ability to provide traffic crossovers (contra-flows) between the main carriageways on the bridge which is an advantage but this is not considered a particularly important criterion since the length of the structure is not excessive (crossovers could be arranged beyond the end of the bridge) and wide hard shoulders are proposed which could allow resurfacing without the need for crossovers to allow contra flows.

At the concept stage, tower aesthetics and foundation costs were expected to be slightly better for the Three Corridor option than the Double Level option. This remains true and is further discussed in Sections 6 and 8 of this report.

The key issues for further assessment of the functional cross sections is connectivity, meaning how the motorway corridors and multi-modal corridors connect to the network at either end of the bridge.

4.3 Three Corridor Option Connectivity

Preliminary studies have been carried out regarding the connectivity for the three corridor option. As noted in Section 4.1 above the studies assume that the multi-modal corridor will be used for HOV lanes initially.

4.3.1 Southern Approach

At the southern end of the bridge the main highway alignment is D3M as it approaches the bridge. The central reservation is widened to allow future operation of a tram or LRT system within the centre. For the initial case of HOV usage of the multi-modal corridor, HOV's will have been streamed into the off side lane on approach to the bridge. On the southern approach viaduct of the Main Crossing the HOV lanes transition from the off side lane of the motorway into the multi-modal corridor reducing the main cross section to D2M by the time it reaches the cable stayed bridge.

If a tram or LRT system is installed in the future then it will run between the carriageways. The approach cross section will still be D3M and will taper to D2M over the length of the southern approach viaduct. The LRT will cross either above or below the southbound carriageway of the motorway with all grade separations occurring off the main crossing.

A schematic alignment of the southern approach multi-modal arrangements is included in Appendix E.

4.3.2 Northern Approach

At the northern end of the bridge the basic concept is similar although the details are slightly more complex due to the interchange immediately behind the northern abutment.

The approach viaducts to the north of St Maragaret's Hill will provide a D3M cross section on separated structures with sufficient space between the structures to construct a future viaduct to carry the tram or LRT. The gradient of the future viaduct would differ from the road in order to bring the LRT either over or below the northbound carriageway of the motorway.

Lane transitions either streaming the HOV lanes or else tapering the cross section down to D2M will take place on the northern approach spans of the Main Crossing as well as on the adjacent viaducts. The detailing of the viaducts will need to allow for the future change in use but this is anticipated to be relatively straightforward.

A schematic alignment of the northern approach multi-modal arrangements is included in Appendix E.

4.4 Double Level Option Connectivity

4.4.1 Southern Approach

As for the Three Corridor Option, the highway alignment is D3M as it approaches the bridge with HOV's streamed into the off side lane. This lane is separated from the main carriageway and then drops downwards within a structure between the carriageways in order to be at the lower deck level at the Main Crossing abutment. The width of the structure will have to include for sightlines on the curve.

Provision will be made within this structure to bring a future tram or LRT system beneath the southbound carriageway, into the structure and therefore onto the lower deck of the Main Crossing.

If this is implemented then the D3M section needs to be tapered down to D2M. This cannot be safely done in the tight radius curve behind the south abutment so must take place on the southern approach of the Main Crossing. Therefore the D3M section needs to be carried through to the southern abutment which will require the structure containing the trams / LRT to be contained within a box section rather than being in an open channel. It is anticipated that the roof of the box will be provided when the structure is first built. This means that the HOV lanes need to achieve the lower level alignment a short distance behind the southern abutment. The future conversion would involve construction of a wall closing the end of the box and then filling in of the open channel ramps leading up to the box.

The widening of the Main Crossing itself is achieved by providing structural cantilevers to widen the upper deck.

A schematic alignment of the southern approach multi-modal arrangements is included in Appendix E.

4.4.2 Northern Approach

At the northern end of the bridge the basic concept is similar although the details are more complex due to the interchange immediately behind the northern abutment.

The HOV lanes are brought to a low enough alignment sufficiently far behind the north abutment to allow the main carriageways to converge before reaching the cable stayed bridge.

Conversion of the viaduct structures will be required in the future in order to modify the functional use. The detailing of the viaducts will need to allow for this future conversion which may not be straightforward

A schematic alignment of the northern approach multi-modal arrangements is included in Appendix E.

4.5 Comparison

Connectivity of both of the Functional Cross Section options appears feasible. However the Three Corridor Option is more favourable than the Double Level Option for a number of reasons.

4.5.1 Functionality

For the Phase 1 usage of the multi-modal corridor as HOV lanes the connectivity of the Three Corridor Option is significantly better at the southern end of the bridge since the streaming of HOV's off the main carriageway onto the multi-modal lanes is on a straight section rather than a curve and takes place approximately 800m further away from the adjacent interchange than for the Double Level Option. This allows safer lane transitions and better connectivity for both northbound and southbound traffic.

Furthermore, the grade separation required for the Double Level Option is likely to result in poor driver perception of the road layout since the lane transitions will involve curved ramp structures and minimum sight lines. In contrast the lane transitions for the Three Corridor Option will be on straight level sections of road with open sight lines.

For the Phase 2 usage with the multi-modal corridor for trams or LRT the connectivity between the two functional cross sections is almost identical. The Three Corridor option is slightly better at the southern end because the tapering of the section from D3M to D2M is further north of the tight radius bend beyond the south abutment.

4.5.2 Cost and disruption

The initial cost of the transition structures required for the Double Level Option will be higher due to the grade separation requirement which results in additional structures. Furthermore the structures for the northern approaches will have to be detailed for future conversion which is likely to require indirect load paths and greater materials.

The cost of construction works associated with converting to the Phase 2 usage as well as the disruption during the conversion is anticipated to be higher for the Double Level Option. In particular for the northern approaches the conversion works will require the demolition / removal of the viaducts which stream the HOV lanes into the multi-modal corridor and the reconstruction of a new viaduct in between the main carriageways.

5 Assessment of Deck Type Options

5.1 Three corridor options

5.1.1 General

The deck options for the three corridor functional cross section are illustrated in Drawing FRC/C/076/S/101. The stay cables are anchored in the structural zone reserved for the tower legs, in other words in the 'shadow' of the tower legs. This means that the stay cables provide less torsional support to the deck than if they were connected in a more traditional manner close to the deck edge.

To compensate for the reduced torsional support a box girder deck is required. This provides the necessary torsional stiffness to achieve aerodynamic stability and prevent unacceptable twisting under eccentric traffic loading.

Bridges with stay cables anchored along the centreline of the bridge are reasonably common. When that is the case, no torsional support is provided by the stays themselves and the torsional behaviour of the deck is governed only by its own torsional strength and stiffness. The Tsurumi Tsubasa Bridge with a centre span of 510m is believed to have the longest cable stay span in the world of this type.

Although the span of the Forth Replacement Crossing will be longer than the Tsurumi Tsubasa Bridge, the stay cables are not anchored exactly on the centreline and studies have been carried out which show that the torsional stiffness of the bridge is provided by both the stays and the deck stiffness in approximately equal measures.

Nevertheless, the torsional behaviour of the bridge is important and the issues which have been considered are:

- Twist of the bridge under out of balance live load (serviceability issue)
- Torsional shear stresses in the deck
- Transfer of torsional shear from steel to concrete (composite option)
- Aerodynamic stability

5.1.2 Orthotropic Deck

(a) Deck structure

The deck is a relatively traditional orthotropic box girder, 4.5m deep at the bridge centre-line. The fatigue sensitive top flange plate of typically 14 mm is stiffened at 300 mm centres by longitudinal trough stiffeners spanning between diaphragms which are required to maintain the shape of the box and provide the transverse framing. The bottom flange plate typically varies between 10 to 12 mm and is governed by transverse flexure under Special Vehicle loading (equivalent to HB 45 loading in BD 37/01).

A braced diaphragm is proposed in preference to a full plate diaphragm. Plated diaphragms typically contribute about 20 % of the weight of the deck structure, although this percentage increases as the box gets deeper. For the 4.5m deep box a braced truss diaphragm will have the advantages of weight saving and better internal access within the deck. It is considered to be more economic, despite the additional support arrangements required during the assembly process.

A continuous internal web is provided at the stay anchorages in order to efficiently transfer the stay forces into the section and also to mobilise intermediate diaphragms between stay locations in transverse bending.

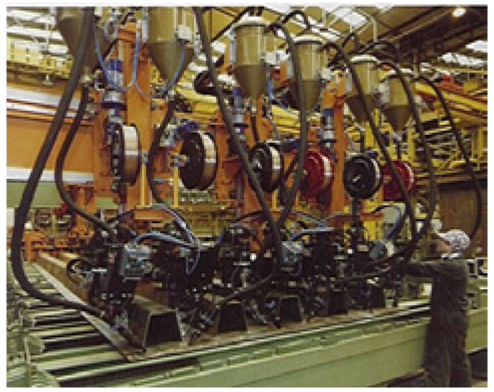
(b) Construction modules

An erection unit length of 22 m is proposed based on a number of issues, one of which is the maximum stay cable spacing achievable within the limit of 127 strands per cable noted in Section 3.3 above. The advantages of maximising the erection unit length are:

- Maximum offsite work in factory controlled environment.
- Maximum work off critical path
- Minimum work at height (Health & Safety benefit)
- Minimum programme
- Maximum speed when overall project is almost at its most cashflow negative

These advantages are believed to justify the additional cost of temporary works and scale of equipment needed in assembly and load-out and erection associated with a larger unit. Gantries and floating plant are available that can handle the envisaged 450 t segment weight.

It is also preferable to maximise the diaphragm spacing, within practical limits, in order to reduce the manual fabrication associated with transverse material. A large part of deck panel fabrication is associated with adding the transverse plating (combs) and this work tends to involve non-automated methods.



Automated Panel Fabrication

Manual Fitting of Transverse 'Combs'

Typically in UK and Europe, the spacing of diaphragms that provide transverse support to orthotropic steel decks is in the range of 3.5 to 4.5m. In the US, the practice is to extend the span of the orthotropic deck to about 6.1m, which results in a heavier trough and deck plate but significant reduction in workmanship. The reduction in work content can compensate the cost of marginally more material. Two reasonable options are available within the 22 m module, either 4.4 m or 5.5 m. The main concern with greater diaphragm spacings is the fatigue of the orthotropic deck plate and a detailed fatigue assessment will be carried out to determine the preferred diaphragm spacing. The design work to date is based on 4.4 m.

(c) Articulation

The deck articulation is as follows:

- Central Tower – monolithic connection
- Flanking Towers – floating system with lateral restraint bearings
- Side Span Piers – guided bearings
- End Piers – guided bearings

The monolithic connection of the central tower provides a maintenance free torsional connection without concerns related to uplift that would be associated with providing bearings. Longitudinal loads due to wind and braking will also be transferred between deck and tower through the monolithic connection. The monolithic connection has to transfer significant loads and will be designed with a combination of prestressing bars and shear connectors as well as local strengthening of the deck. The detailing of this connection needs to allow construction of the tower to proceed above the connection in advance of mobilisation of the deck erection phase of construction. Additional costs would arise if the main span deck erection plant and labour force were to be mobilised for the erection of this segment alone as the remainder of deck erection would not to commence until a number of months after this point. In the event that the units are assembled overseas there would also be either a special delivery of this piece alone or storage costs for the remainder of a full delivery.

At the flanking towers a monolithic connection would result in excessive thermal restraint forces and a floating system has been adopted with no vertical load transfer between deck and tower. Transverse wind loads are transmitted through lateral restraint bearings.

For the guided bearings at the side span piers uplift is a concern when the main span is loaded. This is a common issue for cable stayed bridges with various solutions available. A number of options have been studied:

(i) Monolithic connection

Building the deck into the pier generates significant sway forces in the pier as the ends of the bridge move with thermal expansion and contraction. With conventional reinforced concrete construction the structural demand is excessive. Use of a steel pier which is more flexible is possible, but the additional cost of this solution is unlikely to be justifiable.

(ii) Uplift bearings

Although bearings can be designed to resist both uplift and downward forces, these bearings are both expensive and prone to noise and wear as there is inevitably some free movement between support of upward and downward load. The magnitude of load in this instance is also outside the range of conventional bearings. Uplift bearings are therefore not considered practical for these reasons.

(iii) Non-structural counterweight

It is possible to place non-structural counterweight within the deck such that no uplift would occur. This option is relatively simple but is not the most economical solution.

(iv) Vertical tie down cables

Vertical tie down cables are a practical solution and would utilise the same technology as the stay cables. The cables would be tensioned to ensure that the bearings remained under compression under all serviceability loading conditions. Under ultimate limit state loads lift off of the bearings could occur but the magnitude of the lift off would be limited by the strain in the tie down cables and the lateral guides of the bearings would remain engaged. The height of the piers provides sufficient length for sway of the tie downs under thermal expansion and contraction of the deck. A cost comparison shows the tie-down to be approximately 25% of the cost of the non-structural counterweight.

(v) Composite deck counterweight

The alternative solution is to change the construction material of the last 150 m of the side spans to composite. The additional weight of the concrete slab would ensure that the bearings remained under compression under all serviceability loading conditions. Under ultimate limit state loads lift off of the bearings could occur but the magnitude of the lift off would be limited by separate restraint brackets and the lateral guides of the bearings would remain engaged. Although in isolation this concrete is more expensive than the tie-down option, the saving resulting from the reduced area of orthotropic steel deck would more than compensate for this.

Both vertical tie-down cables and the composite deck counterweight solutions are practical. The composite deck counterweight option is expected to be more economical since it results in a reduction in the overall steel quantities and is therefore the option taken forward.

Movement joints are provisionally located at piers S2 and N2 at the end of the stay cable fan. This minimises the overall length of the cable stayed bridge structure. However, if the approach bridge were to be composite then the movement joint at N2 could be eliminated and the two spans of northern approach could be continuous with the cable stayed bridge structure. The movement joint at S2 could also potentially be eliminated.

(d) Static serviceability

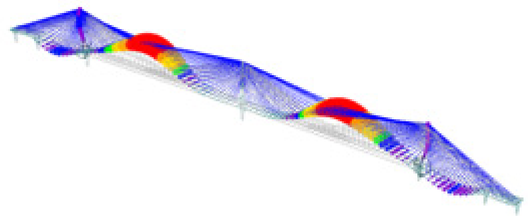
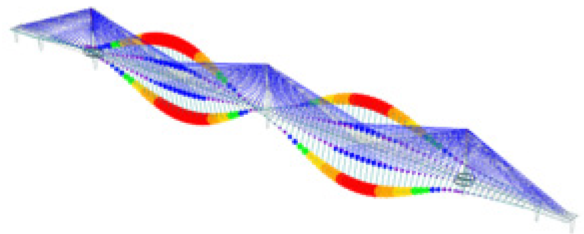
The static serviceability has been assessed to determine the maximum deflections and twists that could occur in the bridge deck due to traffic load. The twist is the change in transverse gradient of the bridge at mid span. Characteristic and frequent values are tabulated below. The quoted return periods are nominal and are for reference only.

Load Condition	Characteristic Loading (1,000 year return period)	Frequent Traffic Loading (1 week return period)
Maximum Deflection (one span only loaded)	3,250 mm	1,300 mm
Maximum Twist	2.6%	1.1%

The maximum vertical deflection only occurs when one of the main spans is fully loaded and the other is fully unloaded. The chance of this occurring is very low and the maximum deflection of 3,250 mm is expected to have negligible chance of occurring during the design life of the bridge. The maximum twists given are more realistic since commuter traffic could realistically result in one motorway carriageway being fully loaded whilst the other carriageway has little or no load on it.

Serviceability criteria for twist are rarely given in design standards or even project specific design criteria. The criteria for this project need to be established. However, by making reference to the Messina Bridge design criteria as well as allowable cant values for LRT systems a maximum characteristic twist of 5% is proposed in the draft design criteria for this project. By comparison with this criterion the bridge behaviour is acceptable.

(e) Aerodynamic stability

First Symmetric Vertical Mode 0.290 Hz	
First Torsional Mode 0.405 Hz	

Orthotropic deck – key dynamic modes

The ratio of the modal frequencies is 1.7 which is significantly higher than the provisional target ratio of 1.2 required to avoid coupled flutter vibrations.

Historic wind tunnel tests carried out for the Setting Forth and Second Severn studies indicate a reduced torsional galloping (flutter) velocity of at least 4.5 for an aerodynamic box-girder section with 3.0m high wind screens. The reduced velocity is a non-dimensional aerodynamic parameter with the following definition:

$$\frac{U_c}{b \times f}$$

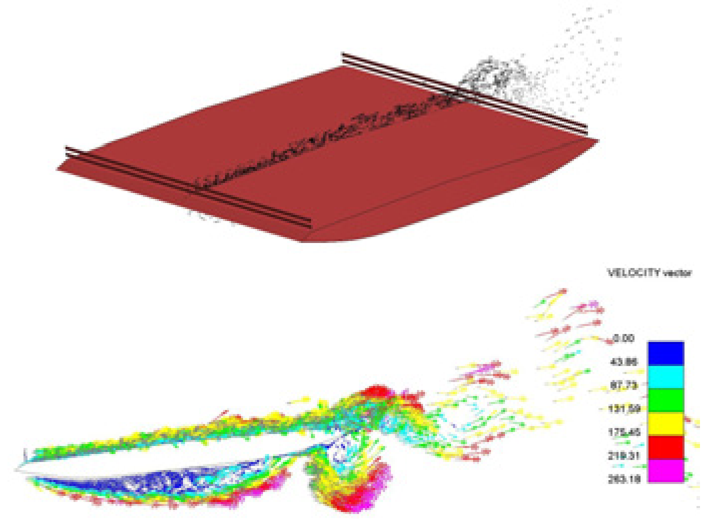
Where:

U_C is the critical wind speed for the onset of torsional galloping (m/s)
b is the width of the deck (m)
f is the fundamental torsional frequency of the deck (Hz)

This non-dimensional parameter would indicate a critical wind speed of at least 90 m/s compared to a target of approximately 60 m/s. Forced displacement discrete vortex method simulations have been carried out to calculate flutter derivatives to study the behaviour of the proposed section which is significantly wider than the referenced wind tunnel tests.

The simulations indicate that the critical wind speed of the proposed cross section should be somewhat higher due to the increased width and the use of the reduced velocity of 4.5 should be safe at this stage and the section is expected to be stable against torsional galloping (flutter). However, sectional model wind tunnel tests are required to confirm this.

Vortex shedding stability cannot readily be studied with numerical simulations and must also be investigated in the wind tunnel. However this low wind speed phenomenon is a serviceability issue not a safety issue and can normally be solved by minor modifications to the section or the addition of guide vanes etc.



ArupDVM discrete vortex method simulations

(f) Edge Detail

The corner unit of aerofoil box sections often accounts for a significant proportion of the work content in fabrication and assembly. The outer 2m of the proposed section does not contribute significantly to the structural performance of the section. It does however provide the nosing detail to control the aerodynamic performance of the section. There are arguments therefore to simplify the construction of the box by ‘squaring-off’ the section at the back of footway and creating the nosing in a non-structural material. This would have the advantage of being able to integrate the wind shielding, the nosing and a rail to support an inspection gantry in a single component. This would also reduce the perimeter that needs to be match fitted between box units in the assembly yard, as well as reducing the assembly width.

On the other hand, the additional workmanship associated with the corner units will be less than typical for this particular bridge section as the fabrication complexity associated with the stay cable anchorages is not integrated with the complexity associated with the edge detail. It is intended to consider both options in more detail in the next phase of design. If a non-structural edge unit is to be proposed, a detail that is equally maintainable as the structural option needs to be developed.

Non-Structural Edge Detail, Pont de Normandie

5.1.3 Composite Deck

(a) Deck structure

The deck for the composite option is similar in general principles to that of the orthotropic option, except that the orthotropic deck plate is replaced by a conventional reinforced concrete slab.

The bottom flange plate typically varies between 12 to 14 mm with trough stiffeners at around 900 mm centres spanning between the cross frame diaphragms. A continuous internal web is provided at the stay anchorages in order to efficiently transfer the stay forces into the section and also to mobilise intermediate diaphragms between stay locations. Two further web elements are located at the edges of the deck to form the edge of the concrete slab.

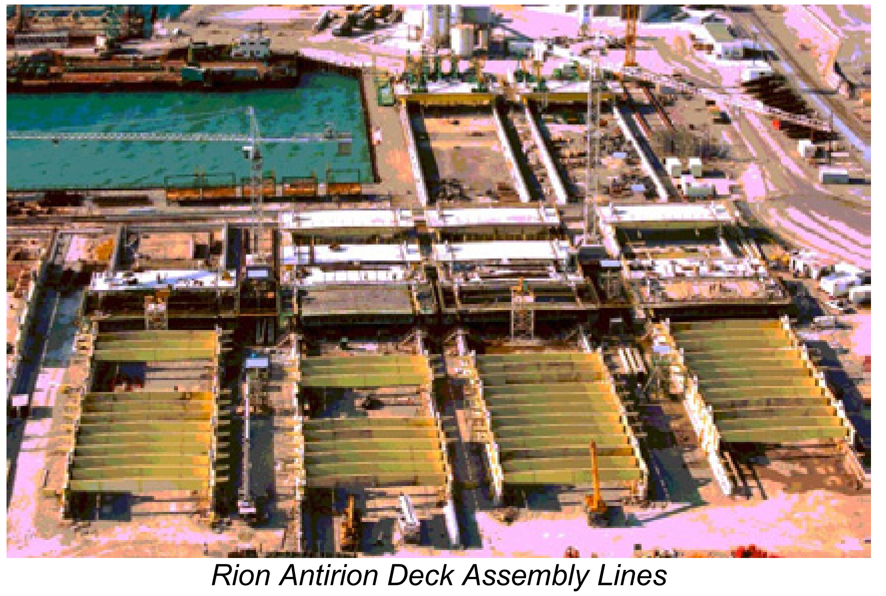
The slab is connected to the webs of the box girder and the flanges of the cross beams by conventional headed shear studs. Transverse prestressing of the slab is proposed to improve the in-service behaviour. This will be anchored on the outer webs.

Three options are being considered for the forming of the slab:

- Cast directly onto the box with folding reusable formwork
- Cast directly onto the box using permanent formwork
- Full depth precast slabs shear connected to the steelwork and stitched together

The advantage of reusable formwork is that given the number of re-uses of the forms, it would ordinarily be expected to be the most economic option. However, for the box girder type deck this will be influenced by the required complexity of the form to withdraw it between the transverse bracing members after the slab has been cast. The further development of the layout of the transverse bracing system should consider how the folding forms might be withdrawn.

Given the anticipated complexity of the forms, the relative economy will also be dependent on the number of forms required to keep pace with erection. The width of the deck and the requirement for it to be a box structure will probably result in the steelwork assembly being on critical path. Depending upon where the boxes are assembled and how much space is available it may be that only a limited number of forms are required to keep pace with the steelwork assembly.



Rion Antirion Deck Assembly Lines

Considering the complexity of the re-usable forms, permanent formwork has been considered as an alternative. This has the advantage of minimum initial plant cost. However, there would also be additional costs associated with this formwork which are believed to be likely to outweigh the savings in initial plant costs. Furthermore there would be a weight penalty associated with increased slab thickness required for either precast plank systems such as Omnia or GRP formwork. The knock on costs associated with this would further reduce the economy of the system.

The full depth precast slab option is attractive as installation of the slabs would be faster than the in-situ options and would avoid the cost of the folding forms. This appears to be a valid alternative to re-usable formwork for this deck type worthy of further investigation.

(b) Construction modules

As for the orthotropic deck, the maximum segment length that can be achieved within the 127 strand limit for the stay cables is proposed (refer Section 3.3). However, the deck will be heavier than the orthotropic option and current studies indicate that the cable stay spacing should be limited to 14 m centres. This leads to typical units which weigh approximately 600 t including both the steel and the concrete. Consideration of the possibility of erecting double length units is discussed in Section 9.1.4 (e).

For the transverse system, the same arguments apply as for the orthotropic deck and a braced diaphragm is proposed, in preference to a full plate diaphragm, for overall economy and better internal access. In addition, a plated diaphragm would prevent the use of folding forms for casting the deck as it would obstruct their extraction after casting. It is preferable to maximise the diaphragm spacing, within practical limits for the maximum span of the concrete deck slab achievable with the minimum thickness required for constructability.

Study work carried out to date indicates that the optimum spacing would be 4.0 m which is attainable with a 265 mm slab if considered independent of the erection unit length. However, the number of diaphragms and the diaphragm position in each 14 m unit should be ideally the same so that the stay anchorage is located at the same point in each unit. This allows common formwork to be used in each unit and any strong points for lifting or attachment of gantries will be in the same position in each unit. Consequently 4 diaphragms at 3.5 m spacing is the optimum regular pattern for the 14 m segment length. For this spacing the concrete slab thickness can be reduced to 250 mm. With the diaphragm spacing below 4.0 m potentially the erection unit length could be slightly increased by further reducing the thickness of the slab outside of the trafficked areas in order to reduce the dead load. This will be studied but the benefit is likely to be marginal.

An alternating pattern is also feasible with diaphragms at 4.0 m centres, with each alternate unit having 3 or 4 diaphragms respectively. The box projection beyond the diaphragm at the end of each unit would be adjusted to create the 14 m length and the stay cable anchorages would fall either at the diaphragm or mid-way between diaphragms alternately. The anchorage detailing would have to accommodate this. This option will be studied as an alternative to the 3.5 m spacing.

(c) Articulation

The requirements for articulation for the composite deck are similar to those presented for the orthotropic deck, and the same arrangements are adopted.

To solve the uplift at the side span piers it is unlikely that adding counterweight, either as non-structural weight, or an increase to the deck slab thickness, will be economically viable for the composite option as there will not be a consequential saving in the deck slab cost. Vertical tie-down cables are, therefore, the current preferred option for the composite deck scheme.

Movement joints are provisionally located at piers S2 and N2 at the end of the stay cable fan which minimises the overall length of the cable stayed bridge structure. However, if the approach bridge were to be composite then the movement joint at N2 would be eliminated and the two spans of northern approach would be continuous with the cable stayed bridge structure. The movement joint at S2 would also probably be eliminated.

(d) Static serviceability

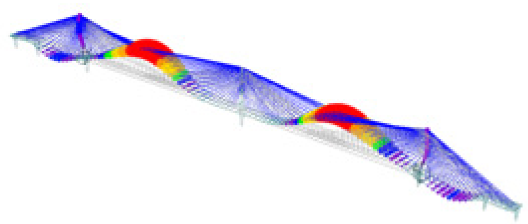
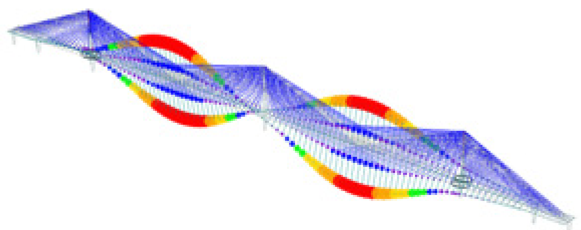
The static serviceability has been assessed to determine the maximum deflections and twists that could occur in the bridge deck due to traffic load. Characteristic and frequent values are tabulated below. The quoted return periods are nominal and are for reference only.

Load Condition	Characteristic Loading (1,000 year return period)	Frequent Traffic Loading (1 week return period)
Maximum Deflection (one span only loaded)	2,200 mm	880 mm
Maximum Twist	2.2%	0.9%

The vertical deflections are approximately 30% less than for the orthotropic deck and the twists are approximately 15% less. The increased stiffness is due to the greater dead to live load ratio for the deck which results in increased stay cable quantities and therefore a stiffer stay system.

(e) Aerodynamic stability

The behaviour of the composite decks is similar to that of the orthotropic deck (see discussion in previous section). The natural frequencies are, of course, altered by the differing mass / stiffness of the systems. The comparable frequencies are as follows:

First Symmetric Vertical Mode 0.263 Hz	
First Torsional Mode 0.369 Hz	

Composite deck – key dynamic modes

The ratio of the modal frequencies is 1.4 which is higher than the target ratio of 1.2 required to avoid coupled flutter vibrations.

A critical wind speed of at least 80 m/s is predicted, compared to a target of approximately 60 m/s.

(f) Edge Detail

The issues relating to the edge detail discussed for the orthotropic steel deck in Section 5.1.2(f) apply equally to this composite option. There is an additional consideration that a vertical web plate is proposed for the composite deck to form the edge of the slab and be an end plate for the transverse prestressing in the slab. This tends to favour the use of a non-structural edge detail.

5.1.4 Comparison

Orthotropic Deck	Composite Deck
✓ Better aerodynamic performance ✓ Lower stay cable material and construction costs ✓ Lower tower construction costs / programme ✓ Potentially lower foundation construction costs / programme ✗ Increased inspection requirements for fatigue critical welds	✓ Better performance for static serviceability ✓ Lower deck fabrication costs ✓ Potentially lower cantilever construction cycle time ✗ Increased number of cantilever construction cycles ✗ Increased inspection requirements for stay cables (due to increased numbers of cables)

Each deck type – orthotropic and composite – has both advantages and disadvantages. In terms of construction cost and schedule it is difficult to differentiate between the two decks, particularly when future commodity price fluctuations could occur. Whilst the composite deck is cheaper to fabricate it requires additional stay and tower quantities to support it. Similarly, whilst the cycle time for each cantilever construction stage could potentially be reduced for the composite deck, the number of cycles is higher due to the need for a reduced stay spacing to limit the stay size.

The most significant technical difference between the two decks lies in the serviceability performance. The heavier weight of the composite deck results in increased stay cable quantities which gives a stiffer structural system. This stiffness reduces the deflections of the deck under traffic loads. However, the increased mass of the deck results in a lower torsional frequency which increases the risk of aerodynamic instability of the composite deck.

Despite these differences, investigations to date indicate that the serviceability performance of both deck types is likely to be adequate. It is recommended that both types be progressed with a view to allowing each tendering contractor to select the deck type for which they can provide the most competitive price.

5.2 Double deck options

5.2.1 General

Accommodating the multi-modal corridor on a separate level underneath the roadway enables a narrower deck to be used than for the three corridor layout. This enables the legs of the towers to straddle the deck without the proportions of the tower becoming too wide compared to the height. The stay cables can then be anchored close to the edges of the deck in a traditional manner.

The deck required for this layout is relatively deep, providing sufficient bending stiffness to the overall structure to deal with the issues arising from asymmetric live loading as described in Section 3.1 and Appendix D which arises due to three tower arrangement.

Both orthotropic steel decks and concrete decks were considered. A steel deck is lighter which would result in reduced stay and tower quantities. However, steel orthotropic decks are best utilised in box girder construction so as to enclose the exposed surface of the deck stiffeners which would otherwise be difficult to maintain. This is not readily achievable with truss option and therefore a composite deck option is preferred.

(a) Truss Arrangement

Three forms of truss arrangement have been considered which generate differing bending moment demands on the chord and bracing members. The options under consideration, which are illustrated in Drawings FRC/C/076/D/111 to 113 are:

- 2 Plane Warren Truss
- 4 Plane Warren Truss
- 2 Plane Vierendeel Truss

Both the two-plane Warren truss and the Vierendeel truss require deep cross beams for the transverse bending effects whereas the four plane Warren truss is partially triangulated transversely.

The longitudinal pitch of the truss is related to the pitch of the stay cable anchorages. The maximum pitch achievable within the 127 strand stay limit has been adopted in order to maximise the size of the erection unit.

The demand on the stays increases considerably towards the mid span of the bridge. Whilst for the Warren truss options varying the pitch of the truss to match the varying stay cable demand would be visually unacceptable this is not the case for the Vierendeel truss where the pitch can be adjusted, or the width of the “posts” varied to suit the demands on the girder, creating an interesting visual appearance with the structural form reflecting its behaviour. Thus the pitch or “transparency” of the structure can be increased away from the high demand mid span regions.

(b) Chord Options

Tubular and fabricated box chord options have been considered for the trusses. There is availability of large diameter tube within the UK although this is somewhat dependent on the wall thickness required. There are also periods of time when the oil business tends to absorb the majority of large diameter tube supply when demand in that industry is high. The advantages of tubular chord members is that they are easier to paint than box sections and the fabrication cost of the basic member tends to be lower.

The advantages of the box chord option are; the ratio of vertical and plan bending capacities can be matched to the bending demand of the section; jointing of box sections is simpler most particularly with regard to bolted erection splices; it is easier to use thick plates in areas of high structural demand; and, it is easier to form the joints onto the other sections.

The box chord option has been taken forward on the balance of the above views.

(c) Slab Options

Similar considerations discussed above for the single level deck apply to the truss options. The open nature of the truss section will allow relatively conventional formwork to be used for the slab. The forms will be able to be re-used without complex folding form arrangements. The advantages of permanent formwork are minimal as a result and would not warrant the extra cost.

Truss assembly will be relatively simple as there are no full length longitudinal connections to create. If may therefore be possible, with sufficient assembly beds to assemble at the same rate as erection. The slab construction may therefore be on the critical path and hence the speed of full depth precast slab erection may be of more advantage for this option.

Both a conventionally formed and a full depth precast slab option will be considered in the next phase of design.

(d) Construction Unit Size

Again the desire has been to maximise the unit size for erection to limit the work at the erection front. A 16 m stay spacing, truss pitch and erection unit size would be ideal but the weight of the structure would result in excessive stay sizes. As a result the typical stay spacing has been reduced to 12 m. The combined steel and concrete weight of a 12m length of deck including the concrete slabs is approximately 450 t. This is easily manageable as an erection unit but will involve relatively frequent erection splices.

The section depth will permit greater lengths of cantilever and erection of double length units is considered favourable as discussed in Section 9.1.4(e).

5.2.2 Two Plane Warren Truss

(a) Deck Structure

The two-plane Warren truss is fully triangulated longitudinally. This results in longitudinal bending and axial effects only generating small secondary bending effects in the chord and bracing members. In the transverse direction there the structure is not triangulated so that distortional loads on the deck and the bending of the upper and lower deck crossbeams generates transverse bending in the bracing members. The initial design has shown these bending moments to be manageable. There will be a cost penalty associated with the required framing around the joints to deal with this bending but this will be compensated for with the halving of the number of bracing members and associated connections in comparison with the four plane option.

(b) Construction Modules

Both the upper and lower decks are traditional ladder type composite construction consisting of a reinforced concrete deck slab supported on transverse spanning cross girders. However, in order to minimise maintenance the cross girders will be fabricated hollow sections rather than traditional open I-girders.

The ability to erect the deck in double length units leads to a higher priority for optimising the cross girder spacing than for the box girder solutions where the erection unit length must first be optimised and the diaphragm spacing must follow. Study work carried out to date indicates that a spacing of 4.0 m would be attainable with a 250 mm slab although local thickening may be required in the region of the towers. The reduction in slab thickness compared to the composite box girder option for the same transverse structure spacing is achievable because of the flange width and torsional stiffness of the crossbeam,

The deck structure is based on a 12 m truss and stay module which is the maximum multiple of 4.0 m spaces that can be accommodated within the limit of 127 strands per stay. It is anticipated that decks will be erected in either 12 m or 24 m long modules.

Future studies will investigate whether a 14 m truss and stay module could be achieved.

(c) Articulation

The deck articulation for all of the truss options is as follows:

- Central tower – floating system with longitudinal and transverse restraint
- Flanking Towers – floating system with transverse restraint
- Side Span Piers –guided bearings
- End Piers –guided bearings

Where transverse restraint is provided at the towers it is envisaged that bearings will be provided to permit the requisite sliding and rotational movement.

The main bridge will be made structurally continuous with the approach spans. Expansion joints will be provided at the abutments only.

Longitudinal restraint is provided to the deck by the stay cables. However, this delivers loads due to longitudinal wind and braking forces direct to the top of the tower which is already significantly loaded in longitudinal bending due to the double main span cable system. It is likely therefore that additional longitudinal restraint will be required and the most practical way to provide this is in the form of buffers.

Two options are available with the first being hydraulically linked buffers at one of the towers which would completely restrain longitudinal movement but would allow plan rotation to avoid an undesirable plan moment connection between tower and deck. The most obvious location for this would be at the central tower but it is possible that providing the restraint at the south tower could be more favourable in reducing the design moments in the critical central tower.

The alternative option would be to provide hydraulically independent buffers at one or both of the abutments which would only restrain short term displacements due to the braking and the dynamic component of wind load. Long term displacements due to sustained wind and thermal expansion / contraction would be unrestrained and the stay cable system would act to transfer sustained wind to the towers and keep the bridge centralised under thermal strains. Locating the buffers at the abutments is advantageous in terms of ease of access for maintenance.

It is possible that these options could be combined with buffers at the tower and at the abutments to minimise the loads on the tower.

Current design work has been based on hydraulically linked buffers at the central tower providing complete longitudinal restraint at this location. The alternatives described above will be studied at the next stage.

As for all of the deck options, uplift is experienced at the side span piers. In common with all options which include concrete deck slabs for the main span, tie down cables are expected to be the most economical way to resist uplift.

(d) Static Serviceability

The static serviceability has been assessed to determine the maximum deflections and twists that could occur in the bridge deck due to traffic load. Characteristic and frequent values are tabulated below. The quoted return periods are nominal and are for reference only.

Load Condition	Characteristic Loading (1,000 year return period)	Frequent Traffic Loading (1 week return period)
Maximum Deflection (one span only loaded)	2,250 mm	900 mm
Maximum Twist	0.6 %	0.2 %

The vertical deck deflections are similar to those found for the composite box girder deck type. Since the dead to live load ratio is similar for those two structural options this indicates that the stiff truss girder provides equivalent global stiffness to the crossing stays adopted for the slender box girder.

As would be expected, the maximum twist of the deck is very much less than for the box girder solutions since the overall deck width is less and the stay cables are anchored close to the deck edge.

(e) Aerodynamic Stability

Divergent aerodynamic instability (flutter and galloping) is not expected to be a problem for any of the truss options in view of the flow of air between the decks and the high bending stiffness of the deck. Nevertheless, this will need to be demonstrated through wind tunnel testing. Wind tunnel testing will also be required to verify non-divergent vortex shedding behaviour of the trusses.

Similarly wind tunnel testing of the towers will be required to verify their aerodynamic stability and their sensitivity to wake buffeting effects, both in service and during erection when they are in a free-standing condition.

5.2.3 Four Plane Warren Truss

(a) Deck Structure

The four plane truss is triangulated longitudinally and partially triangulated transversely. This solution involves the minimum bending at the connection nodes. Each bracing connection will therefore be simpler than those in the other truss options. There are twice as many bracing members and top chord members and the lower chord node is formed with four intersecting bracing members, although these have been separated into two planes for simplicity of detailing. The cost comparison will assess the relative benefit of increased structural efficiency compared with increased numbers of members and connections.

Because of the support provided by the inner plane of the truss and inner top chords, the top deck crossbeams are 3-span beams and are therefore lighter and shallower than in the Two Plane Warren Truss. This has a small effect on overall girder depth and vertical alignment.

As for the Two Plane option the deck structure is of traditional ladder type composite construction with a reinforced concrete deck slab supported on transverse spanning fabricated hollow box section cross girders.

(b) Construction Modules

The construction modules are the same as for the Two Plane Warren Truss. Refer to Section 5.2.2(b).

(c) Articulation

The proposed articulation arrangements are the same as for the Two Plane Warren Truss. Refer to Section 5.2.2(c)

(d) Static Serviceability

The static serviceability has been assessed to determine the maximum deflections and twists that could occur in the bridge deck due to traffic load. Characteristic and frequent values are tabulated below. The quoted return periods are nominal and are for reference only.

Load Condition	Characteristic Loading (1,000 year return period)	Frequent Traffic Loading (1 week return period)
Maximum Deflection (one span only loaded)	2,200 mm	880 mm
Maximum Twist	0.8 %	0.3 %

The vertical deflection is similar to the Two Plane Warren Truss whereas the twist is slightly higher as one would expect.

(e) Aerodynamic Stability

The aerodynamic performance is similar to the Two Plane Warren Truss. Refer to Section 5.2.2(e)

5.2.4 Vierendeel Truss

(a) Deck Structure

In the longitudinal direction the structural system comprises two planar Vierendeel trusses inclined outwards. The truss comprises an upper and lower chord with vertical bracing members which appear tapered when viewed in elevation. While this is an interesting and unique solution, with some advantages from an aesthetic and originality point of view, the design development has revealed some disadvantages, as discussed below.

The upper and lower chords are required to span longitudinally between web members. However, unlike the fully triangulated Warren truss options, the upper and lower chords of the Vierendeel truss must also carry global shear forces, resulting in significant additional flexural bending effects within the chords. As a consequence the upper and lower decks require significantly heavier truss chords when compared to the Warren Truss option.

These local moments are at their peak at the joints between the members. The result of this is that there is highest structural demand in the areas that are most complicated to fabricate. Several options for the connection details were developed. However, these resulted in either excessively thick plate or large numbers of butt welds in thick plate. It was concluded that these moment connections will inevitably have a very high fabrication content and thus be disproportionally expensive.

Furthermore, the material demand in the chords changes more frequently than would be economic to actually reflect in the structure by changing plate thicknesses. As a consequence of this the Vierendeel truss tends to be relatively inefficient in terms of material quantities when compared to the Warren truss options.

The result of the above is that the steelwork tonnage is greater than for the other options and there will be additional fabrication cost per tonne associated with the increased complexity. The lack of direct load path for the compressive load to the lower deck structure may also dictate a complex phased construction sequence to allow the upper deck to compress without generating secondary moments in the bracings.

As for the Two Plane option the deck structure is of traditional ladder type composite construction with a reinforced concrete deck slab supported on transverse spanning fabricated hollow box section cross girders.

(b) Construction Modules

As for the Warren truss options, the cross girder spacing typically at 4.0 m centres with a 250 mm slab. However, the deck structure and stay spacing is based on a combination of 12 m and 16 m modules to keep the stay size within the limit of 127 strands. This results in an opening up of the structure towards the towers where the longer 16 m module can be adopted due to the reduced demand on the stays.

(c) Articulation

The proposed articulation arrangements are the same as for the Two Plane Warren Truss. Refer to Section 5.2.2(c)

(d) Static Serviceability

The static serviceability has been assessed to determine the maximum deflections and twists that could occur in the bridge deck due to traffic load. Characteristic and frequent values are tabulated below. The quoted return periods are nominal and are for reference only.

Load Condition	Characteristic Loading (1,000 year return period)	Frequent Traffic Loading (1 week return period)
Maximum Deflection (one span only loaded)	4,600 mm	1,850 mm
Maximum Twist	0.6 %	0.2 %

Static vertical deflection of the Vierendeel Truss is more than double that of the Warren Truss options. This greater flexibility is attributable to flexural bending effects in the upper and lower decks as they are subjected to global shear effects. The Vierendeel Truss is also 40% more flexible than the Orthotropic Deck making it the most flexible of all the solutions considered.

The maximum vertical deflection only occurs when one of the main spans is fully loaded and the other is fully unloaded. Although the chance of this occurring is very low and the maximum deflection of 4,600 mm is expected to have negligible chance of occurring during the design life of the bridge the poor static serviceability performance under more common load patches is a cause for concern and would require careful study before this deck type could be adopted.

The large vertical deflections of the Vierendeel Truss could be reduced by adopting crossing stay cables as proposed for the box girder decks. However, this poses some additional fabrication complexities for a truss type deck and requires additional deck width.

(e) Aerodynamic Stability

The aerodynamic performance is similar to the Two Plane Warren Truss. Refer to Section 5.2.2(e)

5.2.5 Comparison

The technical and aesthetic differences between the different truss options are too significant to consider offering multiple options to the tendering contractor. It is recommended that only one of the truss options is eventually carried forward.

However, at this stage it is too early to select between the Two Plane and Four Plane Warren Truss options. Both of them provide interesting aesthetic possibilities. Whilst the number of members is of course less for the two plane option, provided that the member size can be kept sufficiently slim the four plane option may provide a very interesting and transparent structure. The reduction in distortional effects as well as the load share for longitudinal effects should allow this transparency and may result in a more structurally efficient deck overall. Further design development is required to provide sufficient quantities for detailed cost comparison and to provide confidence in member sizes for aesthetic comparison. Once the structures are defined in more detail in this way an informed selection can take place.

On the other hand, sufficient investigation has been carried out to indicate that the Vierendeel Truss is currently the least favourable option from the point of view of structural performance and construction economy. This option was worthy of consideration since it offered a unique structural form which potentially could have been very clean visually and relatively easy to maintain with the minimum number of surfaces for repainting. However, the structural demands have led to heavy vertical members which have to a large extent negated the initial aesthetic benefits of this option. The steel quantities are estimated to be approximately double the other truss options and the fabrication complexity will be high with significant structural demand on the connections. Furthermore the structure will be significantly more flexible than any of the other options considered which raises question marks over whether it would perform adequately in-service.

Considering all of these points it is recommended that the Vierendeel truss is not progressed further.

6 Assessment of Tower Options

6.1 General

The tower options have been developed considering aesthetics, structural capacity and the space requirements for stay cable anchorages as well as inspection and maintenance facilities.

The towers are hollow reinforced concrete structures with the stay cables connected to a fabricated steel anchor box embedded in the upper part of the tower. The use of an anchor box maximises off-site fabrication allowing rapid construction progress and accurate geometry controlled in factory cinditions.

Structural demands on the central tower are particularly significant due to the double main span arrangement. At this stage of design the structural sizing of the tower is based upon grade C50/60 concrete and grade 500B 40mm diameter bars which is considered standard practice in the UK. However, concrete grades up to C70/85 are permitted by the relevant UK National Annexes to the Eurocodes and 50mm diameter bars are reasonably common within international practice. Some tendering contractors may opt for higher strength concrete and/or larger diameter bars which will allow reduction in concrete quantities and/or reinforcement densities. However, a more competitive tender should be possible if the tower dimensions allow for “standard” grades and diameters.

For aesthetic reasons the external dimensions of the flanking towers are kept the same as for the governing central tower although thinner wall sizes and/or lower reinforcement densities will be possible. The towers extend 145m above deck level and for all three towers the external geometry of each upper tower is identical. Due to the vertical profile of the bridge the height of the lower tower below deck varies. The vertical alignment of the bridge has been made symmetrical over the two main spans so that the height of the two flanking towers is identical but there is a difference between these and the central tower. The geometry of the lower tower is therefore slightly different but the variation is achieved in a subtle manner to suit the bridge profile and the aesthetics of the towers is not compromised.

The space required for stay anchorages as well as inspection and maintenance access facilities govern the dimensions required at the tower top, particularly in the transverse direction. For each tower a rack and pinion inspection and maintenance lift will be provided from deck level to the tower top. For the H-Tower the lift will also descend to the base level. Sizing is based on a minimum car size of 0.78 x 1.30 m which is sufficient for 5 persons. Intermediate lift stops together with the required access platforms will be provided to give access to the stay anchors which are spread out over a vertical height of up to 60m. In addition, emergency ladders must be accommodated to provide access/escape in the event of mechanical failure of the lift. The stay anchor box itself is sized to allow stressing of the stays at the tower top and provision is made for bringing large tension adjustment jacks to the anchor head,

The tower foundations are submerged below low-tide level to allow the towers to emerge uninterrupted from the water at all states of tide and avoid a bulky waterline object interfering with the aesthetic form. A consequence of this is that at high tide the tower legs could potentially be exposed to relatively significant ship impact forces. A connecting element is required between the tower legs to provide the necessary robustness.

Aesthetic studies of the towers and development of the tower forms has been carried out continuously from the concept to scheme stage. The aesthetic development and approach to the aesthetic design has been described in the Jacobs-Arup report “Approach to Aesthetic Design and Procurement” May 2008. The initial development focussed on overall conceptual forms based on rectangular cross sections with later refinement studying different cross sections to enhance the forms.

6.2 Three Corridor Options

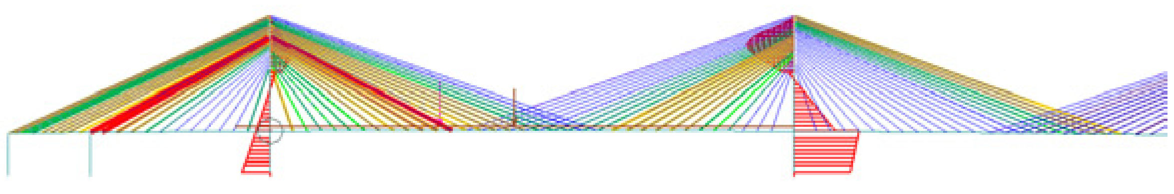
6.2.1 General

(a) Applicability

The towers developed for the Three Corridor Option are applicable for either the Orthotropic or the Composite Deck. As described in Section 2.3.1 above, the external dimensions of the tower are kept the same for both deck types in order that the visual appearance of the bridge will not be affected by the deck type selection. However, the wall thickness and reinforcement ratios will differ depending on the deck type since the structural demands are not the same.

(b) Structural behaviour

The high structural demand in the central tower occurs both in the upper tower approximately 105m above deck level and in the lower tower immediately below deck. In both cases the critical loading is traffic load on one main span only as illustrated below:



Stay forces and tower moments for traffic on one span

In the span being loaded the shorter, steeper stays deliver load to the central tower whereas in the opposite main span it is the longer, shallower stays which transfer the load to the crossing stay region. The result is a large bending moment in the upper tower due to the vertical spread of the stay cable anchorages.

The relative flexibility of the structure results in the central tower being pulled towards the loaded span which results in moment in the tower. This moment is increased due to framing action with the deck provided by the monolithic joint. The result is a large bending moment immediately below deck level.

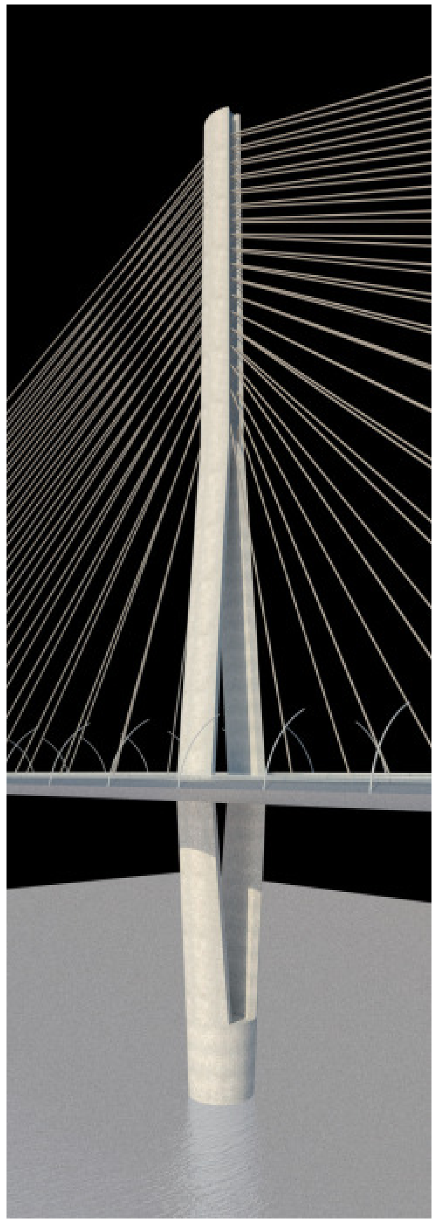
The behaviour is similar for both deck types but the axial forces due to permanent loads in the tower are higher for the Composite Deck whereas the longitudinal flexural moments due to live load described above are slightly higher for the Orthotropic Deck. Transverse flexural forces due to wind are similar for both deck types. The increased axial load for the Composite Deck is dominant and this deck type results in higher structural quantities in the tower.

(c) Stay anchorages

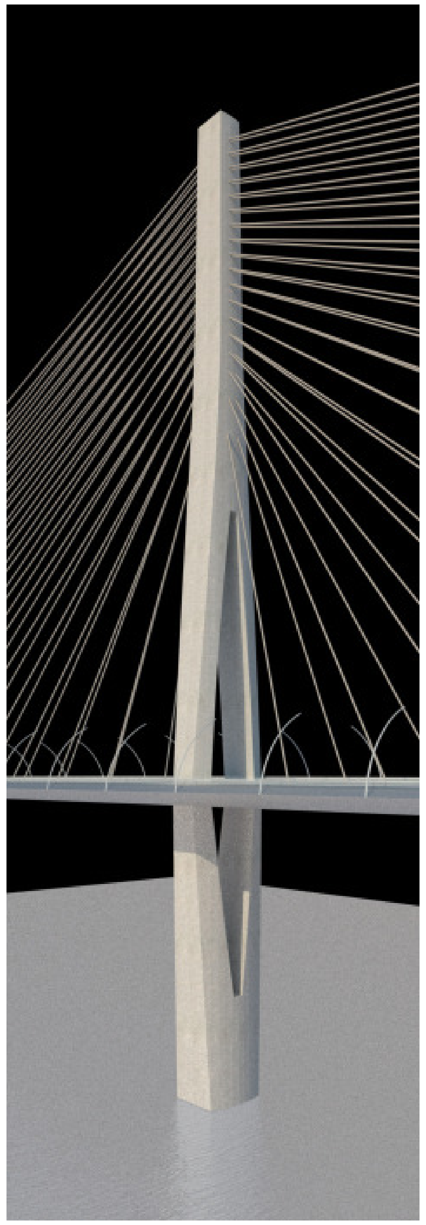
As described above, the vertical distribution of the stay anchorages is important in determining the bending moment in the upper tower. It is proposed that the stays be anchored in a fabricated steel anchor box structure which will act compositely with the upper tower. This is a common arrangement which is adopted on Pont de Normandie and Stonecutters Bridge amongst many others. An exercise was carried out to determine the preferred anchor box height considering a balance between ease of fabrication and maintenance of the anchor box versus reduction in structural demands on the tower. The result was a height difference of 60m between the highest and lowest stay anchor points.

6.2.2 Needle

A great many variations on the Needle Tower were considered during the conceptual design development which resulted in the development of an N1 concept and an N2 concept. The N1 concept is that the tower is developed from an initially circular form, modified to suit the structural and practical requirements. The N2 concept is that the tower is developed from an initially rectangular form, modified to provide improved aesthetics. Drawings FRC/C/076/S/201 and FRC/C/076/S/202 in Appendix B show current versions of the N1 and N2 concepts.



N1 Tower

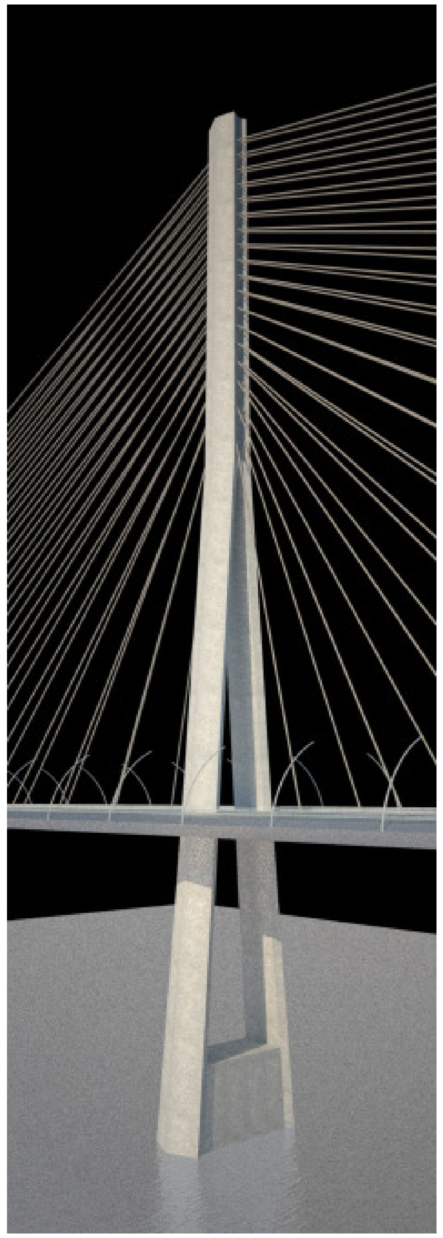


N2 Tower

The N2 tower is recommended because it better achieves the aesthetic concept of a single vertical element centrally located with a hole punched through. Whilst the lower part of the N1 tower is attractive in that a circular shape can be achieved it is difficult visually to resolve that shape in the upper tower without an appearance of two distinct legs separated by the anchor box.

There is no visible cross beam below deck. This is important aesthetically in achieving a simplicity of the deck/tower connection and achieves the further benefit that an under-deck inspection gantry may pass between the tower legs. A crossbeam is required structurally for the flanking towers although it is only required to act as a tie between the two legs so is slim enough to fit within the depth of the deck. The crossbeam could be either steel or prestressed concrete. At the central tower the monolithic connection to the deck acts as the crossbeam.

6.2.3 Inverted Y



Y1 Tower

Y2 Tower

Two cross section shapes were also considered for the Inverted Y tower, which are shown in drawings FRC/C/076/S/221 and FRC/C/076/S/222 in Appendix B. The Y2 concept is similar in form to the N2 concept with an initially rectangular form being modified by introduction of a curved surface on the front and rear faces. The Y1 concept is based around a pentagonal cross section in order to achieve greater shadow definition to the section. The Y2 concept is preferred because of the greater simplicity achieved at the base with a single object rather than two legs and a crossbeam. As with the Needle tower there is no visible cross beam below deck.

6.3 Double Level Options

6.3.1 General

(a) Applicability

The tower developed for the Double Level Option is applicable for any of the three truss type decks considered. In principle the external tower dimensions could vary slightly according the to deck type since it is not intended that multiple truss alternatives will be offered to the tendering contractor. In other words if the Double Level Option is selected as the Specimen Design then one of the candidate deck types will have been selected and the others rejected. However, in practice, the aesthetic and structural requirements for the different deck types are similar and the external dimensions of the tower are kept the same for all types. However, the wall thickness and reinforcement ratios will differ depending on the deck type since the structural demands are not the same.

(b) Structural behaviour

As described in Section 6.1 the central tower forces are governing which is illustrated by the bending moment diagrams shown below.

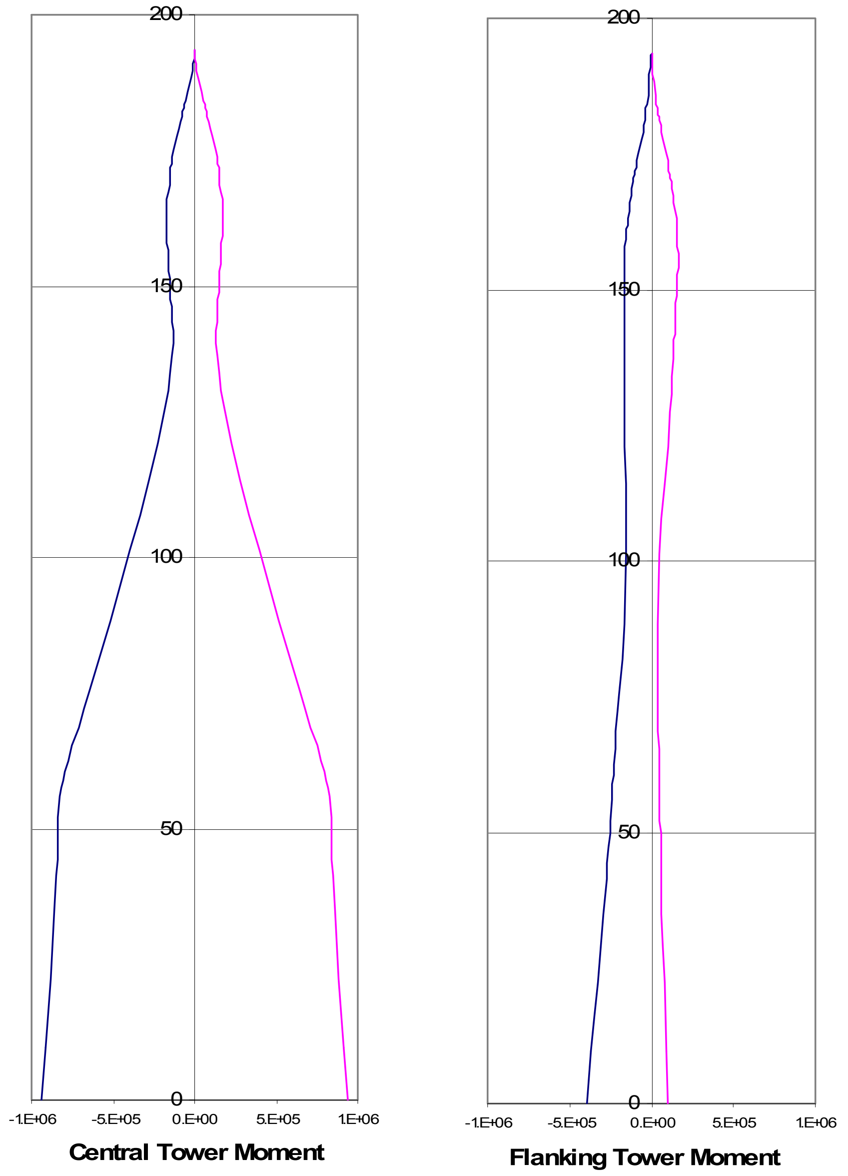
Longitudinal flexure of the tower legs in the governing central tower increases linearly from the bottom stay anchorage towards deck level. However, as a consequence of the longitudinal restraint offered to the deck by the central tower, the rate of increase in longitudinal flexure of the latter is significantly reduced below deck level. This in turn leads to a critical tower leg section not at foundation level, but at deck level where the overall cross section dimensions are smaller yet applied forces are not significantly less than at foundation level.

The towers are restrained laterally at deck level only. As a result, lateral stability of the tower legs is a further major factor driving the design.

(c) Stay anchorages

The tapered tower leg cross section was selected as a natural and elegant solution to reflect the reduction in structural demand with height. However, longitudinal bending of the tower legs over the height of the stay anchorage zone does not follow this trend and requires a significant moment to be carried by a cross section of modest size.

The design proposed incorporates a fabricated steel stay anchorage box acting compositely with the reinforced concrete tower leg whilst maintaining the provision required for access during construction and for inspection and maintenance throughout the life of the structure. As for the Three Corridor Option, the height of this anchor box is a balance between ease of fabrication and maintenance of the anchor box which would be provided by a tall box versus reduction in structural demands on the tower which would be obtained from a short box.

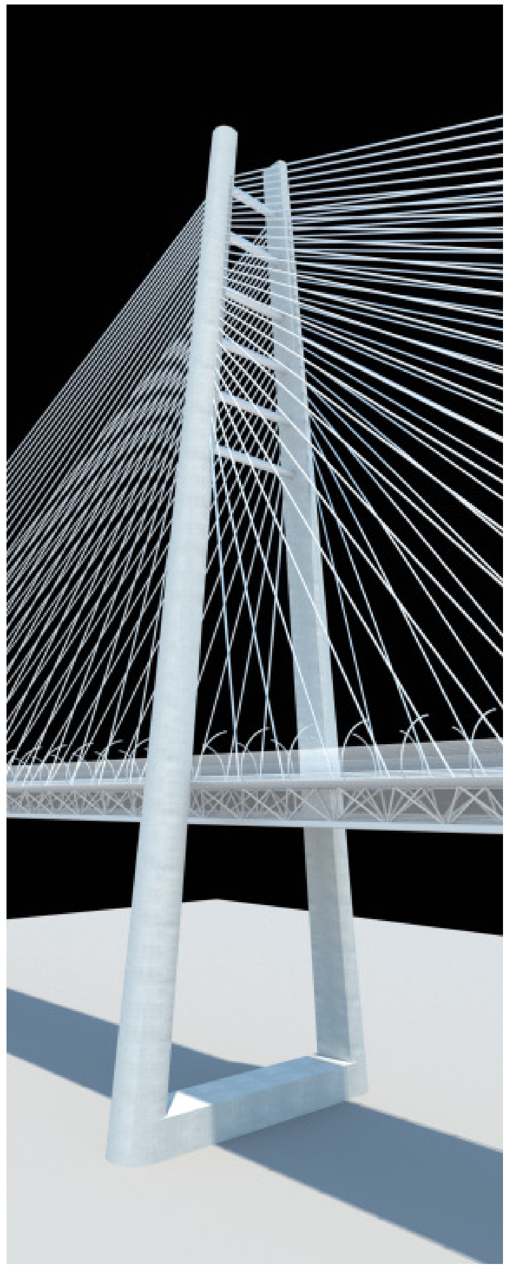


Longitudinal Bending Moments in Tower Legs due to Live Load (Warren Truss Options)

6.3.2 H-Shape

A large number of different configurations were studied for the double level towers. This included variations on “A” shaped towers and “H” shaped towers. Within these a number of variations were considered. The final preferred option is a modified “H” shaped configuration shown in Drawing FRC/C/076/D/241 in Appendix B. Each tower consists of two reinforced concrete legs inclined towards each other and connected together by a series of steel struts in the stay anchorage zone. Thus the crossbeam of the “H” has become, in effect, a series of thin crossbeams moved up towards the top of the legs over the stay anchorage zone.

The selected tower design utilises a “D”-shaped cross section with the curved side of the legs facing away from each another. The legs are based geometrically upon a simple truncated cone with a section removed from the inner face as a result of a planar cut. Currently the plane of the cut passes through the apex of the cone. Further development and refinement of the tower form may include varying the plane of the cut relative to the leg axis to modify the tower proportions.



H1 Tower

6.4 Assessment of Options

6.4.1 Technical Comparison

Comparison on structural performance of the towers is not particularly revealing since the performance of the Needle and the Inverted Y is almost identical whereas the differences in the performance of the H Shape are inextricably linked to the different deck type and articulation arrangement proposed for the Double Level Option. However, one major area of technical difference that can be compared is the footprint of the towers and the significance of that to foundations and ship impact.

The estimated minimum foundation sizes required are approximately 45m by 35m for the flanking towers and 35m by 25m for the central tower. Both the Needle and the Inverted Y tower can be accommodated within this footprint and therefore the foundations for these two options will be very similar. Because the Needle delivers a more concentrated load than the Inverted Y then a slightly thicker pilecap/footing is required but otherwise the foundations are the same. On the other hand the tower quantities for the Needle are slightly reduced compared to the Inverted Y and in particular the quantity of stainless steel reinforcement to be provided in the inter-tidal and splash zone. Therefore the net cost difference is expected to be marginal.

However the width of the H-Shape tower at foundation level is approximately 65m which is much larger than the required size for a single pilecap / footing. Thus, for the flanking towers two independent pilecaps are proposed connected together by a structural beam at waterline which also acts to strengthen the tower legs against ship impact. The overall length of the foundation is greater than for the Needle / Inverted Y options which potentially increases the vulnerability to ship impacts since impacts with the corner of the pilecap may act at a greater eccentricity to the overall pile group and produce a larger twisting effect on the foundation. This, combined with the heavier loads from the double level deck options, results in the need for greater numbers of piles. For the central tower, the greater width results in the tower straddling the high point of Beamer Rock with the pad footing foundations needing to be constructed at a lower elevation on the flanks of the rock. This will require more complex temporary works and greater quantities of rock excavation. The larger footprint also results in greater stainless steel reinforcement quantities in the tower.

6.4.2 Aesthetic Comparison

The objective of the aesthetic design of the main crossing is to build a bridge that will be elegant, unique and instantly recognisable as the Forth Replacement Crossing. At the same time it will fulfil all the functional requirements in a way that delivers value for money in whole life terms, having full regard to buildability and maintainability. The setting for the crossing is a world-famous landscape and the required standard of aesthetics is high.

It is fitting that the overall form of the new bridge heralds the 21st Century, just as the rail bridge is a memorial to19th Century engineering and the suspension bridge relates to the 20th Century.

It is also important that, in addition to being an aesthetically pleasing and iconic structure, the scale of the new bridge is sympathetic to the surrounding landscape and complementary to the form of the existing road and rail bridges. In particular, the towers must not dominate the slender towers of the existing road bridge.

The new bridge will be seen from many locations, both locally and at a distance from settlements, roads and hills in the landscape around the Forth estuary.

Historically, viewpoints at North and South Queensferry have enabled the closest and most dramatic views of the existing bridges.

However, from the north, the new bridge will be viewed most closely from Queensferry Hotel and Admiralty House (also known as St Margaret's Hope), west of North Queensferry, while the majority of North Queensferry will have more distant views beyond one or both of the existing crossings.

From the south, the new crossing would be most visible from the north-west of South Queensferry, Port Edgar marina, Linn Mill, and Inchgarvie House.

Travellers using the new bridge, the existing road bridge or sailing on the Forth close to these bridges will also be able to view the new bridge in close proximity.

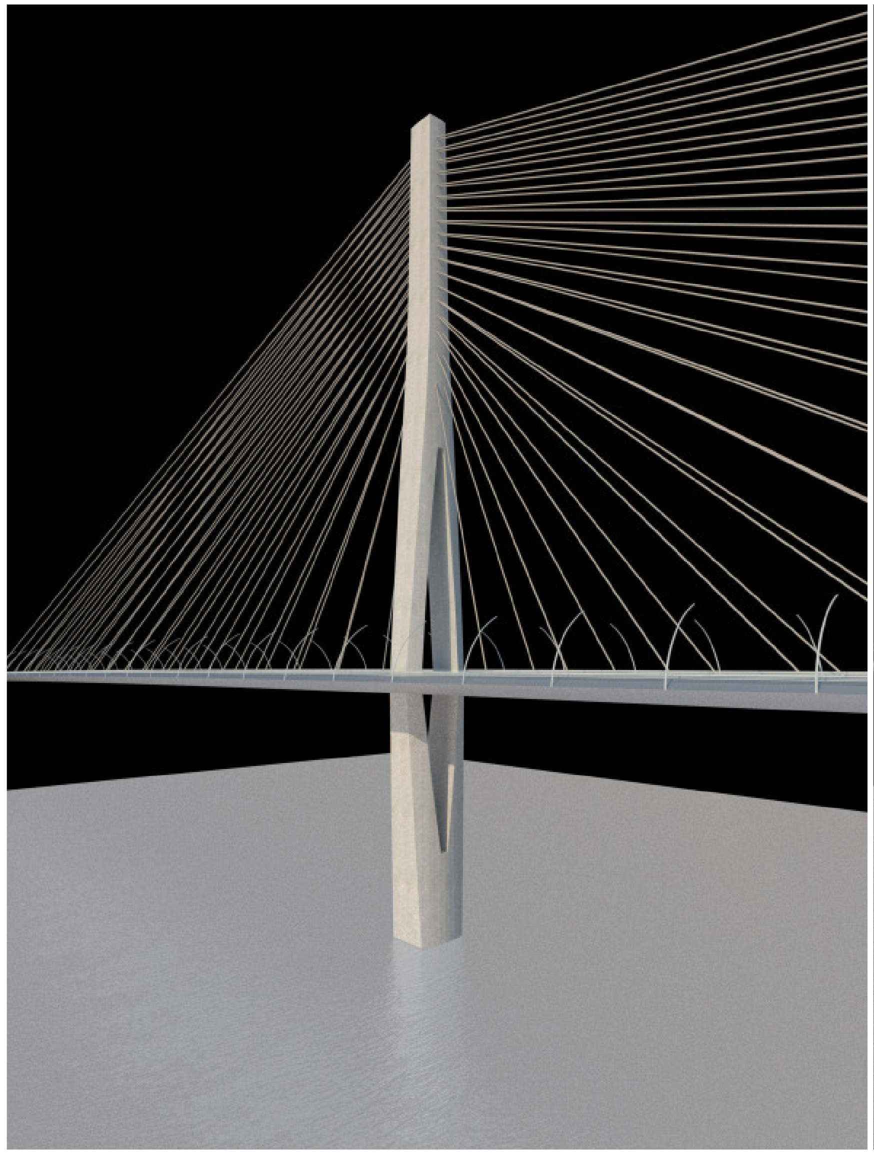
For the Three Corridor Option, the penetration of the tower through the deck is the key to achieving this. The alternative options where the legs straddle the deck would look squat and ungainly with the wide deck required for the Forth Replacement Crossing. Two options have been developed to avoid this; the Needle and the Inverted Y. The slim towers which can be achieved would be in scale with the towers of the existing road bridge. Moreover the shallow depth of the deck will be like a blade across the water. Comparing between these two options, whilst the Inverted Y could be developed into a good aesthetic solution, there is no doubt that the Needle emphasises the aesthetic ideal of a single element piercing the blade-like deck.

For the Double Level Option, the development of the H-Shape tower is an exercise in self restraint. Simple slender elements are arranged so as to complement the more complex truss form of the deck. With the tower having two vertical elements, it is even more critical that the tower should be simple in form to avoid dominating the towers of the existing road bridge. This is achieved with a simple conical form, sliced through with a plane on the inner face to create a shadow line and encase the deck. The crossbeams which are required structurally have been developed as slim minimalistic tubes.

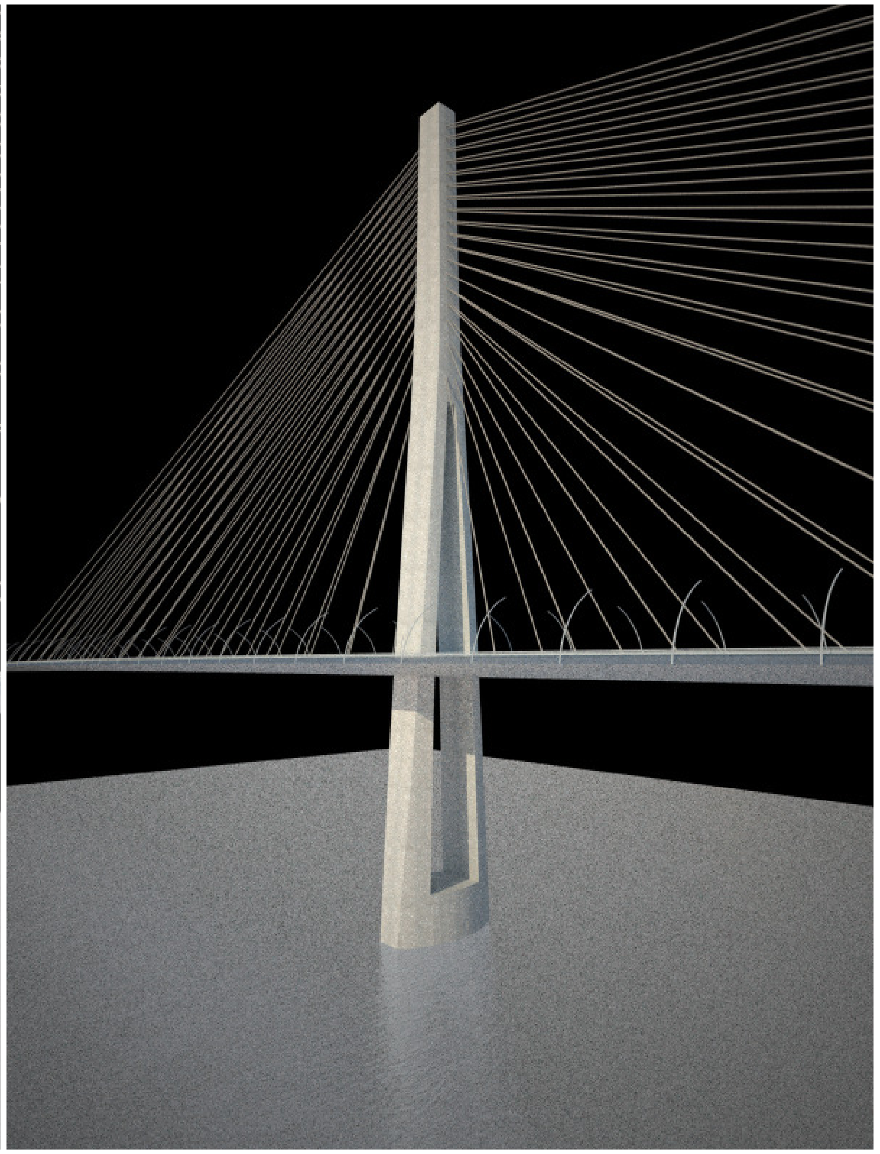
6.4.3 Summary

All three tower options are technically feasible with little to differentiate them apart from the footprint which is expected to lead to higher foundation costs for the H-Shape compared to the other towers. Similarly, all three options are believed to be good aesthetic solutions which can be developed into a final tower form worthy of the prominent site and in sympathy with the existing bridges. However, for the Three Corridor Option, the Needle Tower is believed to be aesthetically superior to the Inverted Y.

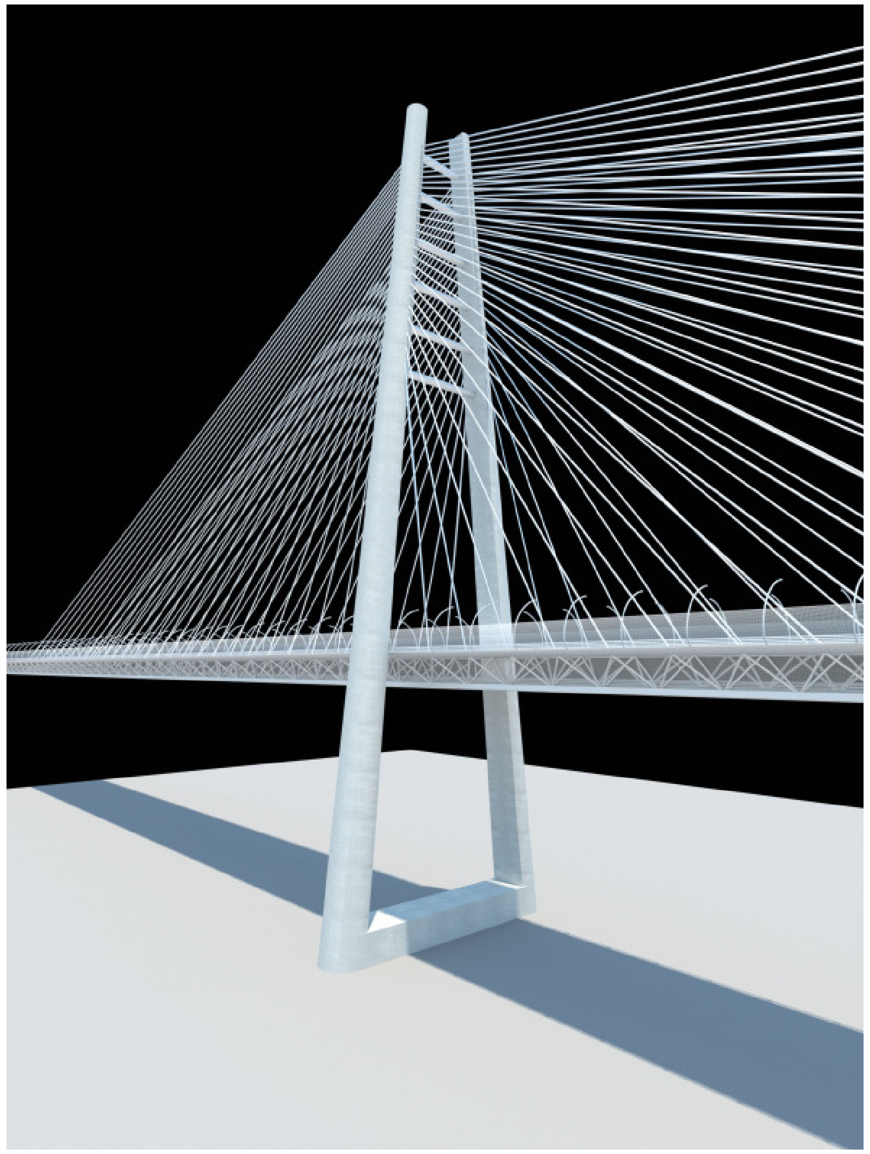
It is therefore recommended to develop the N2 and H1 options for the Stage 3 Assessment.



N2 Tower



Y2 Tower



H1 Tower

7 Assessment of Options for Approach Spans

7.1 Three Corridor Deck Options

7.1.1 Orthotropic

An orthotropic deck has not been considered for the approaches due to the high cost of the steelwork which would be required. Whilst the additional expense is justified for the cable stayed bridge where significant savings can be made in the stay cables and tower due to the lighter deck weight, the same savings are not achievable in the approaches.

There is no argument related to visual continuity of the main spans that could suggest that an orthotropic deck be adopted in the approaches despite the higher cost since the composite deck option for the approaches is aesthetically the same as an orthotropic deck option.

7.1.2 Composite Box Girder

The natural choice for the approach spans is to continue with the composite deck section right through to the abutments. This has clear advantages for the visual continuity between main bridge and approach. It also offers the potential of structural continuity between the two elements. Composite construction is generally a competitive economic solution for the 80 - 90 m spans proposed here. However, due to the large width and use of stiffened steel plate soffit, it is not clear if it maintains this economy for this particular configuration. Composite construction offers the option of push launching from the abutments. The approach bridge for this option is illustrated in Drawing FRC/C/076/S/301

7.1.3 Concrete Box Girder

Although the composite deck approach spans is undoubtedly aesthetically favourable since it provides perfect visual continuity with the main span deck, this may result in increased cost compared to a concrete deck approach bridge. A concrete option has therefore been developed to allow comparative cost estimates to be carried out and allow the cost-benefit of the composite option to be assessed.

The prestressed concrete option has been designed to provide a reasonable balance of aesthetics and cost. The key driver for aesthetics is of course visual continuity with the main spans and whilst this cannot be achieved as perfectly as for the composite box girder option certain measures can be adopted in the design which provide for a reasonable synergy between the cable stayed bridge and the approaches:

- Continuity of visually striking edge detail
- Constant structural depth (and structural depth the same between approaches and main spans)

The approach bridge for this option is illustrated in Drawing FRC/C/076/S/321

7.2 Double Level Deck Options

7.2.1 Truss

For the double level options, the only aesthetically viable solution is for the approaches to be a truss of the same form as that adopted for the cable stayed bridge.

For the Warren Truss options, whether Two Plane or Four Plane, the superstructure is well suited to being continued into the approach spans with no discernible variation in structural form. Spans of 144 m are considered to be the most economic solution, albeit with shorter end spans.

The Vierendeel Truss is less well suited to the approach spans on account of its relatively poor capability to carry large global shears arising from self weight effects. As a result the end bays of the truss are provided with diagonal web members. These are orientated to act in tension and as such can be relatively slender. Nevertheless, the appearance of the structure is unavoidably modified.

The approach bridge for these option are illustrated in Drawings FRC/C/076/D/341 and FRC/C/076/D/342

7.3 Pier Forms

The pier is a very important feature in providing visual continuity between the cable stayed bridge and the approach spans. The factors which have been addressed are:

- Relationship between pier form and tower form
- Cable stayed bridge side span piers read as a continuation of the approach bridge piers
- Any pier widening necessary to accommodate movement joints to be carefully considered with a view to the movement joint pier being read as the same as the typical pier
- Rhythm of the pier spacing
- Accommodation of the large variation in pier height when the approach bridge is over land.

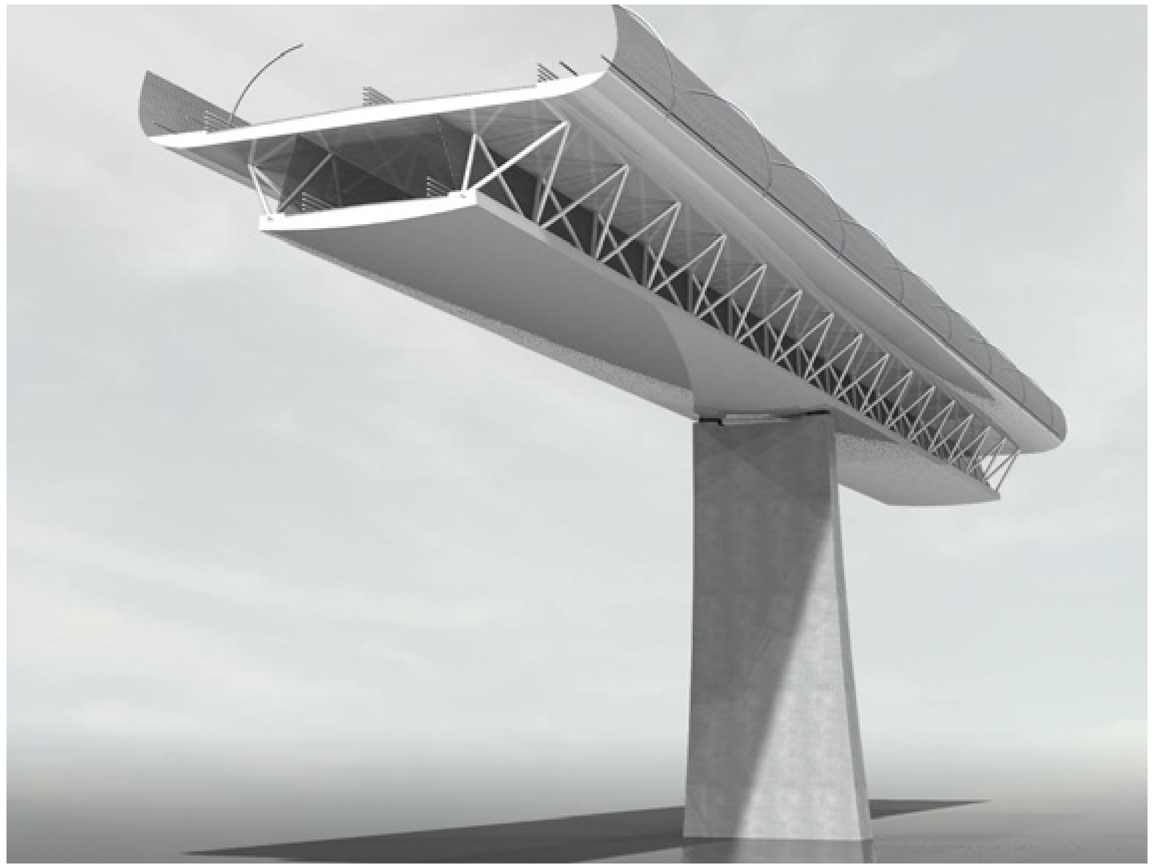
7.3.1 Three Corridor Option

Approach Pier Form

Both the Needle and the Inverted Y towers have common features of curved faces, inclined legs and a connection between the legs at water level. These features are adopted in the approach piers. The small variation in pier height over water due to the vertical profile of the bridge is planned to be accommodated by cropping the top of the pier so that the shape of the pier remains constant. The large variation in pier height over land is achieved by varying the inclination of the legs which results in a small number of unique pier shapes.

7.3.2 Double Level Option

The approach span piers for the Warren and Vierendeel Truss options consist of simple rectangular sections tapered in both elevations. In transverse elevation the inclination of the tapered edge matches the inclination of the main tower legs. The pier cross section will be of hollow reinforced concrete construction.



Approach Pier Form

8 Assessment of Foundation Options

8.1 Site Conditions

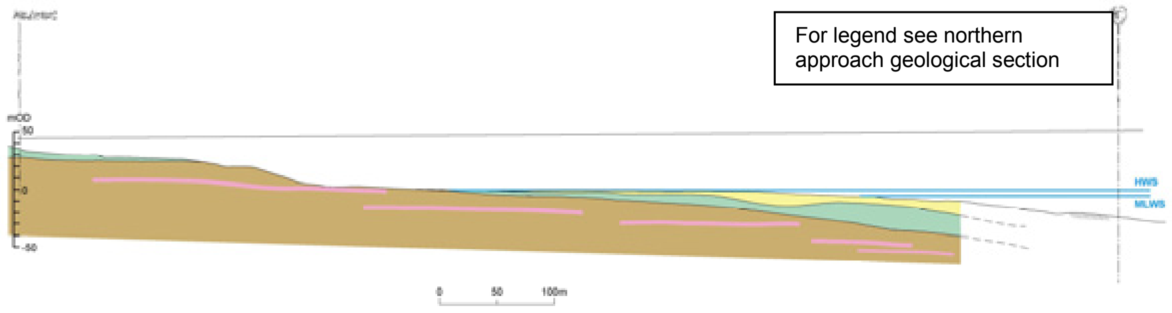
8.1.1 Ground Conditions

A preliminary assessment of the ground conditions along the line of the crossing has been made from published information, the investigations carried out as part of the Setting Forth Study in 1993 and preliminary investigations carried out in 2007 for the Forth Replacement Crossing Study. These have included bathymetric and marine geophysical surveys, and marine boreholes south of the Forth Deep Water Navigation Channel, around the Beamer Rock and north of the Rosyth Channel. However the existing marine boreholes on the south side of the Firth of Forth do not extend as far north as the proposed location of the south tower of the bridge and those on the north side lie to the west of the proposed crossing alignment.

At the crossing location the Firth of Forth has been cut into predominantly sedimentary rocks of Carboniferous age. However igneous rocks intruded into the sedimentary rocks now form the headland at North Queensferry and the Beamer Rock within the estuary. The 1:50,000 scale geological map shows some west east trending faults, and folding giving an anticlinal structure on the south side of the estuary at the crossing location. The dips shown on the map vary from 12 to 20°.

The south abutment of the bridge will be located approximately 265 m south of the shore of the Firth of Forth. The ground level falls from the abutment location towards the shore beyond which the alignment crosses gently sloping tidal flats to the west of Port Edgar marina. These extend for about 500 m from the shoreline before the river bed falls more steeply at a gradient of about 7° at the southern margin of the Forth Deep Water Navigation Channel reaching a lowest level of about -45 mOD in the channel.

The bedrock underlying this section of the crossing is the West Lothian Oil Shale Formation which typically comprises sandstone, siltstone, mudstone and oil shale with thin coal seams and limestone beds. Dolerite sills, up to about 5m thick but occasionally thicker, have been intruded into the West Lothian Oil Shale Formation Near to the shore rockhead lies at or close to the river bed level but falls towards the north and is overlain by variable glacial deposits which are themselves overlain by more recent beach deposits or alluvium. The glacial deposits range from till with a high fines content to coarser grained materials all of which may contain cobbles or boulders. The recent deposits are weaker and vary from sands to soft clay.

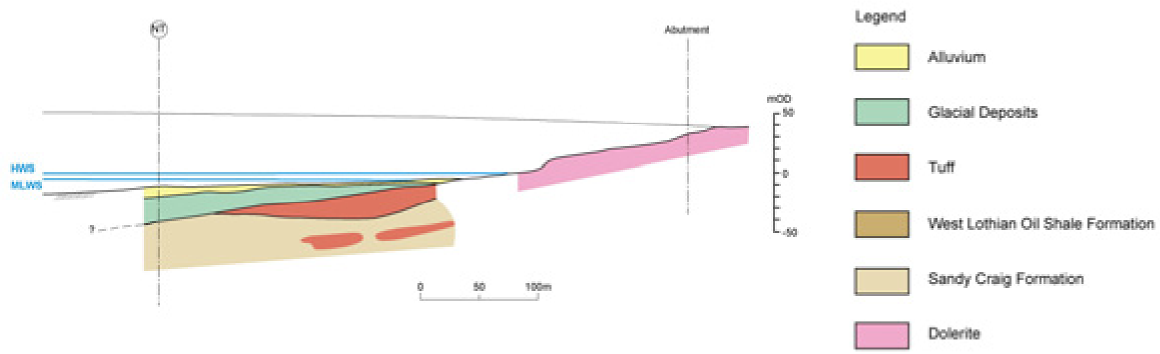


Schematic geological section - southern approach
(Dolerite sills are indicative only)

The Beamer Rock is formed by a dolerite outcrop which reaches an elevation of about +3 mOD close to the existing lighthouse. It forms a ridge trending north west - south east and the area of rock exposed varies with the tide reaching about 45 m by 95 m at low water springs. The bathymetric surveys have shown that the south and east sides of Beamer Rock are very steep with slopes of 60 to 65°. The northeast and southwest edges are less steep with slopes of around 25 to 30°.

To the north of the Beamer Rock the river bed falls to about -33 mOD in the North Channel. The bed level then rises northwards to the Rosyth Channel which is dredged to a level of -12 to -16 mOD. The north margin of this channel rises at a gradient of about 6° towards the more gently sloping north foreshore which extends for about 300 m towards the northern landfall near Cult Ness.

On the north side of the Rosyth channel bedrock consists of the Sandy Craig Formation. This typically comprises sandstone siltstone and mudstone but the previous investigations have revealed layers of volcanic tuff within the Sandy Craig Formation in this area. The bedrock is overlain by glacial and recent deposits which decrease in thickness towards the north shore. These are generally similar to the deposits previously described on the south side of the estuary.



Schematic geological section - northern approach
(extent of Tuff indicative only)

Ground level rises steeply at the north shore of the Firth and is underlain by dolerite. The bridge alignment is partly sidelong to the topography in this area reaching the northern abutment some 140 m from the shoreline.

Further marine and land ground investigations are being undertaken to extend the coverage of the previous investigations and further consideration of the foundation options described below will depend on the findings of these investigations.

8.1.2 Water conditions

The tidal range at Rosyth from MHWS to MLWS is 5 m and the Admiralty Chart indicates that peak tidal currents vary up to about 2.3 kts.

8.1.3 Constraints to foundation arrangements

Within the Firth it is proposed that the top of the foundations should be below the river bed level or -5 mOD, whichever is the higher. This is to ensure that the foundations will not be visible at any state of the tide whilst at the same time limiting the hazard that submerged foundations could pose to small craft. Suitable markers and warning lights will be provided to indicate the extent of the foundations.

8.2 Foundation Options

8.2.1 Key Considerations

The selection of foundation scheme will depend on

- Ground conditions, and in particular the depth to a competent bearing stratum or bedrock
- Water depth
- Constructability

These factors, which vary along the crossing alignment, will determine whether in situ construction within temporary cofferdams or precast caissons or pile caps installed under water are preferred.

A further key consideration in the design of the foundations will be the capacity to resist ship impact loads. These will be most onerous at the south tower location.

8.2.2 Caissons

In the FRCS Reference Design caisson foundations taken down to rockhead are shown for the main tower foundations. Precast caisson foundations have been used for recent major bridges including Oresund Bridge and the Second Severn Crossing where units have been positioned on a prepared rock formation. In both cases rock was at or close to the sea bed. Where a suitable bearing stratum is only encountered at significant depth below bed level caissons constructed and sunk in situ have historically been adopted for several major bridges, including the south tower of the existing Forth road bridge.

At the south tower of the existing Forth road bridge a pair of caissons were sunk from within a temporary cofferdam to bedrock at -29 mOD. Provision was made for excavation under compressed air, but the glacial deposits were found to consist of boulder clay of low permeability and the use of compressed air was not required. The water depth and anticipated thickness of soft alluvial deposits at the new crossing north and south tower locations are expected to be greater than at the existing bridge south tower, and at the new crossing south tower substantially greater. Furthermore, the glacial deposits appear likely to be more variable comprising interbedded fine grained and coarse grained materials. These conditions are much more onerous and sinking a caisson to bedrock would be challenging. The use of compressed air may not be practical at the depths required for a caisson at the south tower and alternative means of excluding water and maintaining a stable base to the excavation such as dewatering, grouting or ground freezing, would be difficult to implement and involve significant risk.

Precast caissons could be an attractive solution for some of the approach span piers where rockhead is at shallow depth, and could also be considered for the central tower.

8.2.3 Piles

An alternative foundation solution is large diameter piles. Current piling technology enables piles up to 4 m diameter or more to be drilled into rock using reverse circulation drills (RCD), and piles of 3.85 m diameter have recently been constructed for the new Kincardine bridge. For initial studies of foundation schemes 3 m diameter piles have been considered.

Large diameter piles with relatively high axial and bending resistance could be constructed by initially driving a steel tube to bedrock (or as deep as practical if boulder obstructions are encountered). An RCD would then be mounted on the steel tube and the plug of soil inside the tube drilled out. If the tube has stopped short of bedrock it would have to be taken down to rockhead with the drilling operation. A socket into the rock could then be drilled below the base of the tube and the socket and steel tube filled with concrete reinforced as necessary.

Layouts with both vertical and raking piles have been investigated but the preliminary studies have indicated that arrangements with vertical piles only are most practical and likely to be most efficient.

A key consideration with piled foundations is the method of pile cap construction as the pile caps will be fully submerged. Although one option would be in situ construction within temporary cofferdams, a cofferdam would be very difficult to construct in the deep water and ground conditions anticipated at the south tower location. An alternative would be to use precast pile caps and preliminary studies have indicated that this should be feasible using technology developed in the offshore industry.

These studies have considered a cellular reinforced concrete structure incorporating sleeves to take the piles. The size of the cap required will depend on the form of the tower and design ship impact force. Preliminary sizing for the Needle or Inverted Y towers gives overall plan dimensions about 45 m by 30 m and a depth of 6 to 8 m resulting in a total structure weight of 7,000 to 9,000 t. This could be constructed in a suitable dry dock or on a submersible barge and floated into position using temporary buoyancy towers to ensure that the unit floats in a stable position. Alternatively construction in sections that are then stressed together afloat could be investigated.

Further important considerations with piled foundation schemes will be the method of forming the connection between the piles and cap and construction tolerances. For the precast pile cap solution a possible construction method could be to initially install some locating piles using a sea bed template to accurately position the piles. The pile cap unit would then be floated over the piles during slack water, ballasted onto the piles and the sleeves grouted. The remaining piles could then be installed using the pile cap as a guide with connections also formed by grouting the sleeves.

8.2.4 Pad foundations

In areas of shallow water or on land where rock or competent strata are encountered at shallow depth pad foundations constructed in situ are a possible solution.

8.3 Central Tower Foundations

Foundations for the central tower will bear directly on Beamer Rock with the Needle, Inverted Y and H Shape tower forms imposing different constraints on foundation geometry and founding level. Some rock excavation will be required to reduce the existing surface profile to the required founding level and construction in water depths of up to about 10 m may be necessary depending on the tower scheme.

One solution would be to reduce the level of the rock by underwater excavation and float a precast caisson into place. Once ballasted on to the prepared platform a grouted contact with the rock would be formed at the base of the caisson.

A temporary cofferdam would be required to permit the alternative of in situ construction of a pad foundation in dry conditions. The temporary works required for this will potentially be difficult as toe penetration of driven piles into the rock will not be practical. A scheme using concrete walls with vertical anchors into the rock was however successfully used at the Mackintosh Rock for the north tower cofferdam for the existing Forth Road Bridge where conditions were similar.

8.4 Flanking Tower Foundations

8.4.1 South Tower

The south tower will be located in deep water - with 650 m main spans it will be located where the river bed level is about -22 mOD (shown as ST on the geological section). The tower location lies to the north of the corridor previously investigated during the Setting Forth study in an area where the geophysical survey results from the Setting Forth and Forth Replacement Crossing Study investigations were inconclusive. Whilst there is at this stage considerable uncertainty as to the ground conditions it is likely that there will be a substantial thickness of soft alluvium below the river bed and extrapolating from existing information the top of the glacial deposits has tentatively been assumed at - 35 mOD and it has been assumed that rockhead level could be as low as -55 mOD. Additional investigations are being undertaken to establish the ground conditions at this location..

It is proposed that a piled foundation scheme with precast cap should be developed further in the next stage for this location.

8.4.2 North Tower

The north tower location lies to the east of the corridor previously investigated during the Setting Forth study. With 650 m main spans it will be located where the river bed level is about -9 mOD (shown as NT on the geological section). Based on the existing information it has been assumed that the river bed is underlain by soft alluvium with the top of the glacial deposits tentatively assumed to be at -15 mOD and rockhead at -35 mOD. Further investigations are being undertaken to confirm the ground conditions at this location.

A piled foundation scheme with a precast cap could also be constructed at this location following some dredging to provide sufficient draft to float the cap into position. Alternatively an in situ pile cap constructed within a temporary cofferdam may also be feasible. It is proposed that both of these options should be investigated further in the next stage.

8.5 Side Span Foundations

The locations of the side span piers (S2, S1, N2 and N1) will depend on the structural form adopted. Some of the side span foundations are required to carry both compression and tension loads in which case rock socket piles would provide a suitable foundation solution. Where the foundations are required to carry compression loads only and rock is at shallow depth a pad foundation constructed insitu or precast caisson foundations may be an alternative. Pile caps are likely to be permanently submerged on the south side of the bridge but may be within the intertidal zone or on land on the north side of the bridge. It is proposed that solutions for the marine piers in which the pile caps are either precast and lifted into place or constructed insitu within a temporary cofferdam are investigated further in the next stage.

8.6 Approach Span Foundations

The spans of the approach viaducts and locations of the piers will depend on the structural form adopted. Several alternative solutions may be considered for the approach span foundations depending on the water depth at the marine piers and depth to a competent bearing layer. On land and near to the south shore where rockhead is shallow, pad foundations are likely to be feasible whilst with increasing distance from the shore and increasing thickness of alluvium and/or variable glacial deposits piled foundations will be required. Close to the shore pad foundations would probably be constructed in situ within temporary cofferdams but the alternative of precast caissons lifted onto a prepared base within a dredged pocket could be considered. Similarly where piled foundations are adopted either pile caps constructed in situ within temporary cofferdams or precast units lifted into place could be considered. These options should be further investigated in the next stage.

9 Options for Construction Methods

The design needs to take into account constructability issues and likely methods of construction. For each component of the bridge, several methods of construction are feasible.

9.1 Cable Stayed Spans

9.1.1 Towers

The tower construction options are similar for all forms of tower and deck. The difference between the options will be in the detail. Most recent cable stayed bridge towers have been cast with jump forms and this form of construction will be assumed for all the tower options considered. The detail of the required formed shapes and how these vary up the tower will be discussed with formwork suppliers to ensure the viability of the formed surfaces.

The raking legs of the towers will demand temporary strutting until connection is made between the two legs at the tower head. It is anticipated that the needle tower will require the minimum strutting. The required strutting points are to be determined in the next phase to allow the derivation of temporary work quantities and costs.

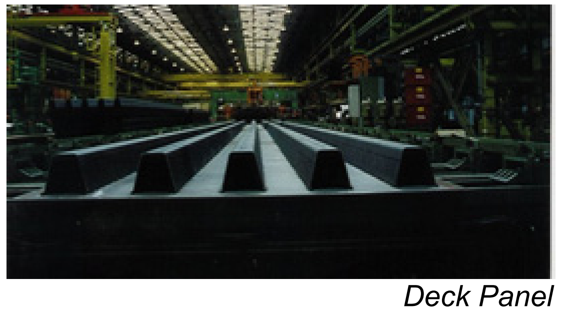
Common to the majority of recent cable stayed bridges the stay anchorage zone at the tower head is a composite steel and concrete element. The steelwork has usually been erected by one of three methods.

- Tower crane – Small piece erection with the size of piece dictated by the capacity of the tower crane (45t for the Pont de Normandie).
- Floating crane – Piece limited by the reach of the crane. In this instance, the 200m height of the towers will put it beyond the reach of floating cranes.
- Strand jack – The towers of Ting Kau Bridge were detailed to allow the erection of whole anchor boxes using strand jacks.

The tower crane and strand jack options will be assessed to determine programme and cost implications.

9.1.2 Steelwork Fabrication

There are many options available for fabrication of the steelwork in terms of source of supply, ranging from UK, Europe and the Far East. The limiting widths and lengths of plate available vary with source of supply. The size of order for this project is such that special lengths and widths will be possible to procure without premium. The contractor should be given latitude to optimise the panel width to suit the source of supply. This latitude should be limited for the deck plate of the orthotropic deck to prevent longitudinal welds on the wheel paths of the marked traffic lanes.



- Typically 3 to 4m wide
- Maximum 5m wide
- Length – Maximum 32m

Deck Panel Fabrication

The fabrication unit size will tend to be in erection unit lengths. This is feasible for the 12, 14 and 22m lengths of the truss, single level composite and orthotropic deck respectively. The truss chords could be manufactured in 24m lengths if a double unit were to be erected. Some fabricators would opt to splice the 22m plates in the works. As a result, transverse butt welds in controlled positions should be allowed.

9.1.3 Steelwork Assembly

Steelwork assembly for large bridges has traditionally taken place close to the bridge site, with the fabricated units being delivered flat pack either by road or by sea. The assembly site would not necessarily need to be an existing facility but a large area of storage plus a quay wall frontage adequate for load-out of complete units is required. There are suitable areas beside the Forth, including Rosyth, Burntisland and Methil. The area required for storage is dictated by the relative speeds of deck assembly and deck erection. Unit assembly takes longer than erection and hence either a buffer of completed deck units has to be stored prior to the start of erection or multiple assembly lines have to be used.

Typical assembly yard adjacent to site.

An assembly yard close to the site has the advantage of minimum transportation cost. If delivered by sea, the deck panels or truss members will be stacked to occupy the optimum volume of the vessel. Road delivery is also feasible and would be economic for UK sourced fabrication.

In recent years there has been a tendency to assemble complete units in shipyards with low cost labour or where efficiencies arise from the plant available at the shipyard. Øresund assembled in Spain and Carquinez assembled in Japan are examples of this. The advantage of lower cost of assembly has to be countered by increased transportation costs. When shipping assembled units, the volume of the unit dictates the size of the vessel rather than the weight and hence very large vessels are required to carry small loads. Shipping costs have been escalating very rapidly in recent years for two reasons; the commodity demand from the Far East and India and the rising cost of oil. This may act to reverse the trend of assembling away in distant yards.

Assembly Facility in China

In the next phase of design, the space required for either transhipment or assembly close to the site will have to be determined and possible worksite areas for both options identified.

9.1.4 Deck Construction Unit Size

(a) Considerations leading to large unit sizes

Generally there is benefit to be obtained in maximising the size of pieces pre-assembled and erected as single construction units. The larger the unit, the greater the amount of prefabrication and factory controlled work that is possible. In addition the work in assembly may be carried out off critical path for overall project completion. Work carried out on the erection front will almost invariably be sequential and on critical path.

For the Forth Replacement Crossing site, minimising work at the erection front will be particularly important as the wind climate will probably dictate that there is a significant proportion of time when work on erection is not possible. Hence, there is a further incentive to maximise the size of the piece and minimise the number of wind sensitive operations.

(b) Deck Erection Methods

It is likely that erection will be carried out either using winches or strand jacks mounted on a gantry at the erection front or by floating crane. As far as lifting capacity goes the strand jacks can operate together as multiple units and hence there is not a limiting capacity. However, a 500t strand jack is the maximum size that operators prefer as control of the strands and repair or replacement of locked grips is difficult when the strand bundle is larger than this. With a twin headed gantry and a pair of strand jacks mounted on each head an upper bound lift capacity would be 2,000t.

Very large floating cranes are available, for example Svanen which was used to erect the Oresund Bridge has a capacity of 8,000 t. However, a 3,000 t limit on lift size for floating cranes would be reasonable to maintain competition between marine subcontractors. Winches could be reaved to provide similar lifting capacities to the strand jacks although lower lifting capacities would be more practical. The maximum lifting weight being considered is 1,100 t for which winches would be sufficient.

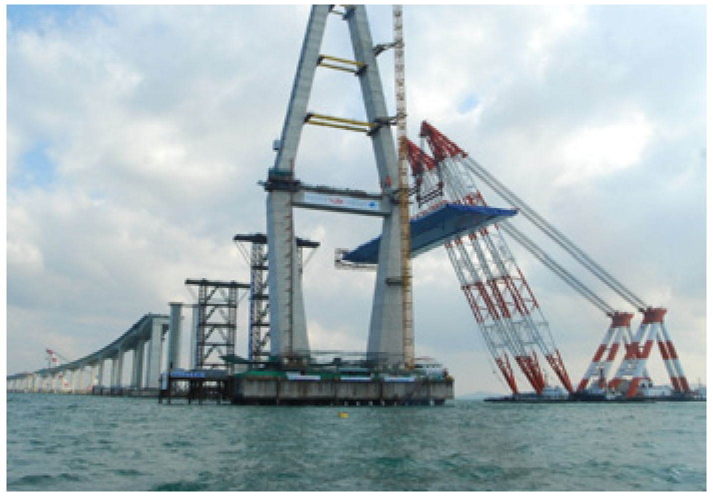
Floating Crane Deck Erection

Gantry Deck Erection

(c) Orthotropic Steel Box Girder

The design deck steel deck weight is approximately 22 t/m and generates a limiting length that can be lifted of 90 m and 135 m for the strand jack and floating cranes respectively. However, the structure would not tolerate erection of units of this length for cantilever construction.

However, the side spans could be erected in long units vertically supported on permanent and temporary piers. This could allow rapid construction of the side spans to reduce the number of cantilever construction erection fronts.



Large unit erection of cable stayed bridge side spans

(d) Steel-Concrete Composite Box Girder

The weight of the steel alone for a 14 m typical unit will be approximately 200 t. The intention would be to lift the deck section including the concrete slab, with the exception of the stitch concrete forming the joint between sections. This increases the weight to approximately 600 t. A double length unit of 28 m would be possible to lift but is likely to present too big a demand on the structure. The handling of the 1200 t unit in the assembly area would also have a cost premium. It is unlikely that a double length erection will be feasible. This assumption will be checked in the next stage of design.

(e) Double Level Truss Girder

For both the Warren truss options, the weight of the steel alone for a 12 m typical unit will be approximately 130 t. The intention would be to lift the deck section including the concrete slabs, with the exception of the stitch concrete forming the joint between sections. This increases the weight to approximately 450 t.

This is easily manageable as an erection unit but will involve relatively frequent erection splices. The additional section depth will permit greater lengths of cantilever and hence a 24 m double length unit will be considered for erection. It is unlikely that the deck will need to be strengthened for this double unit lift. However, it may require a staged stressing of the stays to prevent the front stay being overstressed.

For the Vierendeel truss option the weights are slightly higher at 260 t for steel alone and 580 t including the concrete for each 12 m unit. The same principles apply as for the Warren truss options with double unit lifting being a possibility.

9.1.5 Deck Erection

The primary considerations relating to deck erection are:

- Size of piece
- Method of delivery
- Method of lifting

(a) Size of Piece

The size of piece to be lifted has been discussed above, with the piece length varying from 12m to 24m depending on the structural form. The weight of the piece varies between 450t for the 22 m long single orthotropic deck option to 1100 t for the double segment lift for the truss option. A staged construction analysis of the structure will be carried out for each of the options to prove the feasibility of these lifts in the next phase of design. The expectation is that only the 1100 t double segment lift will require temporary staying or staged stressing to prevent overstressing of the stays at the erection front.

(b) Method of Delivery

There are a number of permutations that will dictate the method of delivery of the deck segment to erection front. If the unit has been assembled close to the bridge individual units will be delivered to the erection front by small flat topped barges if the deck is erected by gantries or picked from the quay by floating crane and carried to the erection front directly if erection is by floating crane. There is adequate draft over nearly the whole length of the span to deliver the piece directly under its final position. On the south end of the cable stayed spans, tidal working will be required. On the north end, the construction method used for the approach spans can be employed for the first three segments.

The self propelled trailers that move the segments around the assembly yards would either deliver the unit under the hook of the floating crane or would drive onto the flat top barge with the unit. The choice of delivery method would dictate the draft and quay wall load-out capability required for the assembly area.

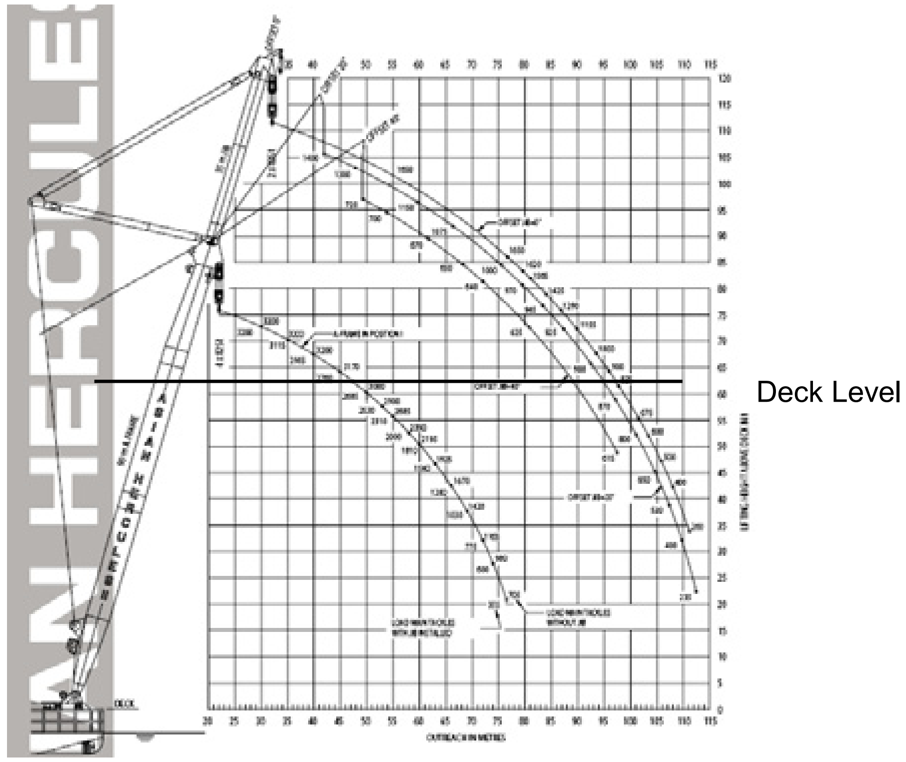
If the segments have been assembled distant to the site, they could be delivered by ocean going vessels directly under the erection front. This has the advantage of avoiding the cost of an assembly area and a storage area. This method may not be the most appropriate for a cable stayed bridge though where the segment erection cycle is longer than for a suspension bridge. There are significant demurrage costs for these vessels (the daily rate for the vessel). There will need to be pilotage and tug assistance whilst the vessel is under the bridge and it will present a significant obstacle to shipping whilst in position. The erection cycle time associated with each segment of the cable stayed bridge will therefore have to be minimised but is unlikely to be less than 7 days. Hence if assembly is distant to the site there is likely to be transhipment and delivery by flat top barge or floating crane as described above.

Delivery with Ocean Going Vessel (Suspension Bridge)

Hence with all methods of delivery, an area of land within coastal navigation range will be required for the either segment storage or combined segment assembly and storage. These areas exist close to the Forth but their availability will have to be assessed.

(c) Method of Lifting for Erection

The deck will erected either using floating cranes or by gantries mounted on the erection front. Both have advantages and the design should allow for both options to be possible. The advantage of the floating crane is that it performs both the delivery and the erection function. It would be able to operate in smaller weather windows and it would be able to service more than one erection front. There are six erection fronts possible on this three tower bridge.



Lifting Capacity Chart for Largest Floating Crane in Smit Tak Fleet

The disadvantage is that there are few cranes with the reach and capacity required for this bridge. This scarcity has a number of implications: the contractor has to ensure the availability of a particular vessel, advance booking of vessel will attract programme risk, there are cost implications associated with scarcity and these vessel tend to service the oil exploration and production business which can drive the price up in times of high demand.

It should be noted also that the temporary steelwork required to hold the unit temporarily between release from the crane and completion of the deck joint is of a similar order of complexity as the gantry that would support a strand jack or winch lift.

Temporary Segment Connection

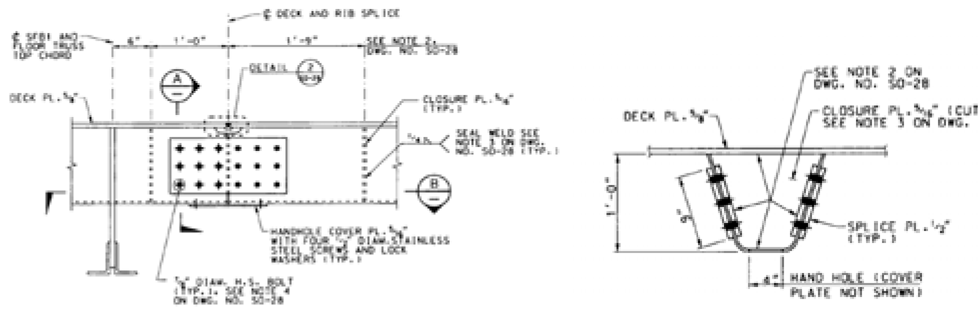
Gantry erection has the advantage of being relatively low cost, but the movement of the gantry forward adds to the erection time cycle. There is also the disadvantage that a special erection method is required for the pieces at the towers. Gantry erection would permit the storage/assembly area to be further away from the site. Up to a day sailing distance away would be practical. It is also possible to attach temporary stays to the gantry. These stays would be stressed by strand jacks and allow the lifting of the 24m long unit.

The design going forward is to allow for both floating crane and gantry erection.

(d) Orthotropic Deck – Erection Splice

Two forms of orthotropic deck erection joints are in common usage. The detail prevalent in Europe is an all welded connection, whilst in the US, Korea and Japan, a hybrid connection is used where the deck plate is welded and the trough bolted.

These hybrid connections and they were first used on Kessock Bridge and a bolted trough was also used on Dartford, the deck was also bolted and then covered with a thin slab. The detail has not been repeated since in Europe as far as we are aware. It has however been used in the US with a demand to ream out both ends of the splice from the splice plate, one end in the works and one end in situ. This we understand was to avoid having to predict the weld shrinkage in the deck plate. They also tended to have a closing plate welded inside the trough. All of the above resulted in greater time being expended on the detail than for the all welded version, however some of this time is not at the erection front.



Typical Hybrid Connection from US

The crucial aspect will be how many (if any) of the troughs need to be connected prior to stressing the stay cables and moving the gantry/erection aid forward. Then how many more need to be connected before the next piece can be erected.

On cable stayed bridges, joint welding can commence almost immediately after segment erection, but there are closely following processes dependent on the strength of the connection.

To conclude, the all welded solution is preferable if one can take the welding of the majority of the troughs off the critical path. If the welding of a significant proportion of the troughs is on the critical path, then the hybrid version would be the preferred option. Both details have proven fatigue performance and are considered to be equivalent in-service.

9.1.6 Deck Erection Phasing

The sequence of deck erection will be determined by the pace of construction of the towers and the stability of the free cantilever deck, prior to joining at midspan. In theory it would be possible to work on all six erection fronts at the same time. However, this would not allow one to smooth the labour requirements both in terms of numbers and mix of workers. It would also involve the maximum quantities of plant and temporary materials as minimum re-use would occur.

The spread footing foundation for the central tower on Beamer Rock is likely to be constructed more rapidly than the piled foundations for the flanking towers. The central tower could therefore be available for deck erection in advance of the flanking towers. The central tower does not benefit from the erection stability offered by the anchor piers S1, S2, N1 and N2 in the side spans adjacent to the flanking towers. The design will have to establish that the deck is aerodynamically stable when erected as far as the start of the overlapping stays for the single level options and as far as the end stay for the truss options.

The expected optimum sequence would be to start deck erection on the two fronts extending from the central tower. The erection from the first flanking tower (anticipated to be the north tower due to the lesser pile depth) would then commence at the point when half of the deck attached to the central tower is erected. Then once erection from the central tower had progressed as far as aerodynamic stability allows, the plant and labour would be moved to the remaining flanking tower. With this sequence four erection fronts are being used for half the erection duration and two erection fronts for the other half.

An alternative sequence to be considered would involve construction of temporary piers in the sides pans to allow erection of the side spans as large units independent of stay cable installation. This could potentially reduce the construction time or the number of erection fronts.

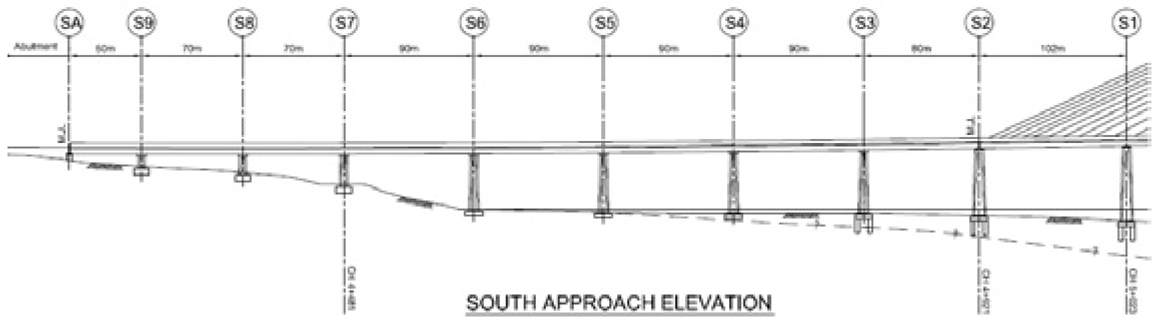
In the next phase of design, these sequences will be analysed to predict programme duration and derive labour histograms and plant requirements. The design will also prove the viability of the proposed sequences by analysing the structure at critical temporary stages.

9.2 Approach Viaducts

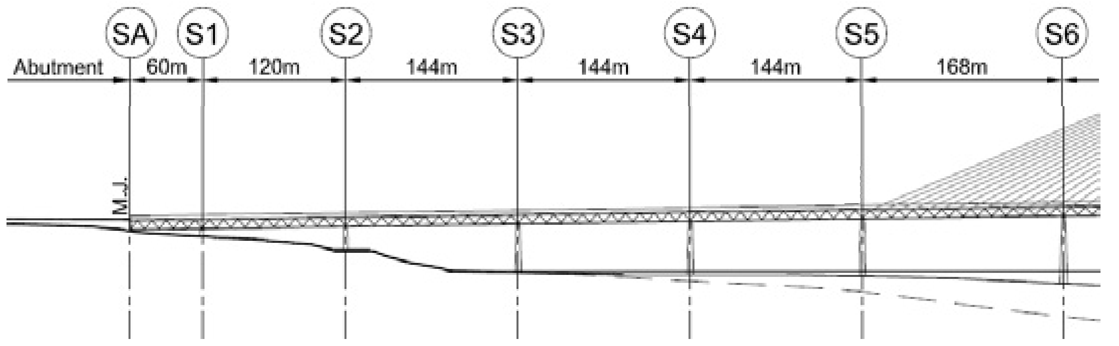
9.2.1 South Approach Viaduct

There are significant constraints to the construction of the South Approach Viaduct. There is the proximity of the listed buildings within Port Edgar and the 'Sites of Importance For Nature Conservation in Edinburgh' (SINC). The intertidal zone is difficult for construction as neither marine nor land based plant is suited for this area.

Three deck forms have been considered for the viaduct. The single level options would either extend the composite deck box to the abutment or transition to a segmental concrete multiple box option. The two level truss options will maintain the same section to the abutments. The typical span length for the single level options is around 90m and for the double deck option it is around 144 m .



Three Corridor Option - Span Arrangement



Double Level Option - Span Arrangement

(a) Deck Launch

All options can be launched with the exception of the concrete deck. The truss options would require temporary piers to allow the launch as the span is significantly larger and the roller/skid loads on the chord when the roller/skid shoe is half way between truss nodes is an onerous condition. The Vierendeel truss may still require temporary bracing members to be installed to accommodate the high shears that occur during launching.

The launch would most likely involve an assembly area behind the South Abutment. The steelwork would be assembled and launched forward. The cantilever section would require a launching nose or a temporary stay arrangement or possibly a combination of both. It is unlikely to be economic to have the concrete in place for the section of the deck that acts in cantilever during the launch but it may be possible for the concrete to be included on the trailing section. The alignment of the webs within the box sections has been kept at a constant offset even though the deck is tapering in plan to allow a launch without having to move the roller/skid point laterally.

An alternative to assembly behind the south abutment would be to create a trestle platform between Piers S1 and S2 and to erect segments in the same way as for the cable supported spans and then once joined, launch southwards to the abutment. This would have the advantage of not having to replicate the plant and equipment associated with assembly.

It should be noted that this method of construction could be used from the South Abutment to north of Pier S1. This will allow rapid completion of the cable stayed deck erection associated with the South Flanking Tower as an alternative to the large unit erection described in Section 9.1.4(c).

(b) Small Piece Construction

The truss deck in particular could be constructed by small pieces. This could either be done with a derrick mounted on the erection front or if a causeway had been created for foundation construction in the intertidal region, the deck could be erected from the ground. It is envisaged that construction would commence from the abutment and the truss would cantilever towards the next pier. As with the push-launched option temporary piers would be needed to reduce the length of the cantilever. Although this option is likely to require a longer programme than the launch, this element of construction does not need to be on critical path.

It is unlikely that this would be cost effective for the complex assembly associated with the single level box skin.

(c) Concrete Construction

The span length of the concrete option dictates that the construction method should be one of the following:

- In-situ balanced cantilever
- Precast segmental balanced cantilever

Although in-situ span by span construction is feasible for this span length it would require a very heavy erection gantry which makes it unlikely to be economical compared to balanced cantilever construction.

If in-situ balanced cantilever construction were adopted, the same access used for foundation construction would be used for material delivery. This is most likely to be either a piled jetty or a causeway for the intertidal region. Travelling formwork would be used to incrementally construct the cantilevers.

If the erection is by precast balanced cantilever then it is likely that an overhead gantry would be the most efficient form of construction with segments delivered along the completed deck until they could be lifted by the erection gantry. The gantry would be very much lighter than that required for in-situ span by span construction since it need only support the weight of a single segment at a time. Each of the three lines of deck segments could be erected in turn by the same gantry. Construction could commence from deepwater and work towards the abutments to allow segments to be delivered by barge. Alternatively segments could be sized for road transport with construction commencing from the abutments.

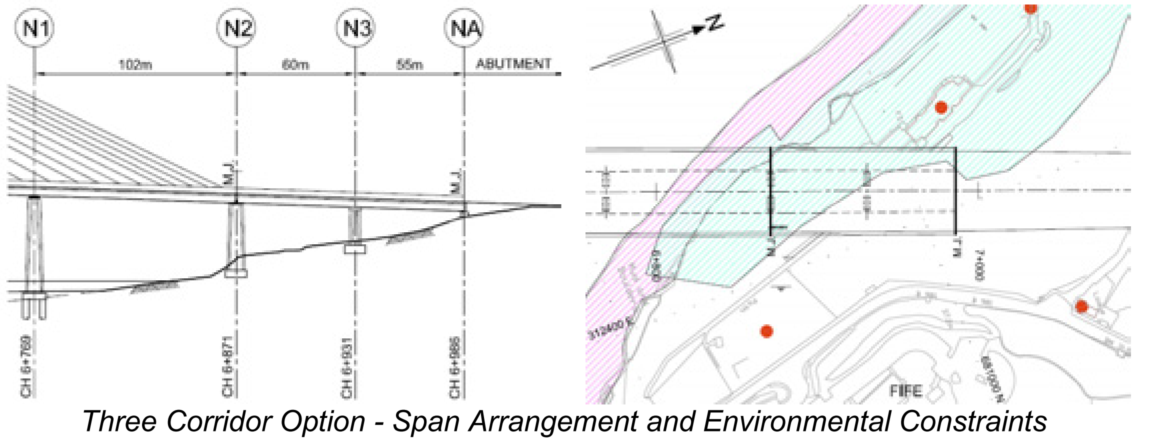
Balanced cantilever segment erection by overhead gantry

The first 85m of bridge which is south of the SINC could be constructed in-situ on falsework directly supported off the ground.

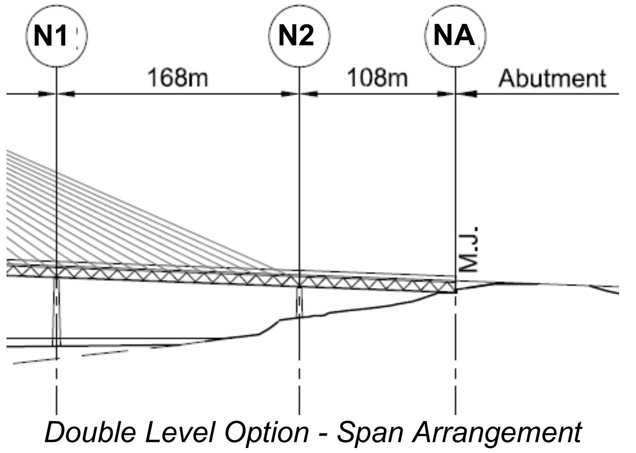
The duration of construction would be longer than the launch but as noted above this is not a critical driver as this element will not be on the programme critical path.

9.2.2 North Approach Viaduct

The shoreline between Piers N1 and N2 is an environmentally sensitive area designated as a Special Protection Areas (Scotland) and Site of Special Scientific Interest (Scotland). Above the shoreline the hillside is included in the semi-natural woodland inventory roughly as far as pier N3. Finally, St Margaret's House immediately west of the north abutment is a listed building. These environmental constraints will be important to the construction of the bridge in this region and the options described below will be analysed in the next phase to determine which is optimum in economic and environmental terms.



Three Corridor Option - Span Arrangement and Environmental Constraints



Double Level Option - Span Arrangement

(a) Trestle Supported Launch

The relatively short length of viaduct on the north side means that the creation of an assembly area is unlikely to economic. Hence delivery will be by marine plant unless small piece erection is used. A self-supported launch is not viable if one is launching away from the pier N1 as the first span is the largest. The preferred option is therefore to create a trestle supported track beam at the underside of deck level. Deck segments would be erected onto the track beam either with the use of a gantry or floating crane and the unit then skidded along the track beam into position.

The disadvantage of this method is that if gantry erection is being used for the cable supported spans, then this element of construction would be on critical path.

(b) Small Piece Erection

The comments for the South Approach Viaduct are equally applicable, but it would not be worth assembling a derrick which would be bespoke for this viaduct only.

(c) Track Skid Under Approach

As with the trestle supported launch, it is not worth setting up an assembly area for such a small portion of viaduct. This option would involve the creation of an inclined skid track at ground level beneath the deck and a load-out quay wall with adequate draft for the segment delivery barge. Segments would be delivered to the load out quay by flat top barge or floating crane and the units would then be skidded up the incline. Once joined the units would be lifted on trestle towers. Construction of the permanent piers would then occur. The infill section adjacent to Pier N1 would be strand jacked directly from a barge positioned beneath.

Considering the environmental constraints this is unlikely to be a viable option compared to the trestle supported launch.

(d) Concrete Construction

It would not be worth mobilising the overhead gantry for this small length of viaduct. The most economical form of construction would be to cast the decks in-situ on falsework directly supported off the ground. To reduce the environmental impact, spanning falsework could be used, possibly with intermediate supports which could reduce the span of the falsework to about 25m whilst still having a relatively small footprint on the ground.

9.3 Foundations

The construction options for foundations are discussed in Section 8 The access required for foundation construction may influence the choice of deck construction method, particularly for the approach viaducts. The inter-relation of these will be examined further in the next phase of design.

9.4 Construction Programme

Preliminary construction programmes have been developed based on the following assumptions:

- 5 ½ day working week
- Single shift working
- 10% loss of productivity due to weather (assumed)

The programmes are included in Appendix F. The total construction duration from site mobilisation to completion of finishes is:

- Orthotropic Deck – 66 months
- Composite Deck – 68 months
- Truss Decks – 71 months

For the truss decks the assumed lifting piece was 12 m. However, as has been noted in Section 9.14 above a 24 m lifting piece may be possible which would result in a comparable duration to the orthotropic and composite decks.

The approach viaduct construction is not on the critical path.

At the next stage of design more detailed programmes will be developed. The assumed 10% loss of productivity due to weather will require a quantitative assessment based on wind records for the site.

10 Durability, Inspection and Maintenance

10.1 Durability

The durability of the Main Crossing is of paramount importance considering the aggressive marine climate, large capital investment required for construction and the great cost and difficulty that can be associated with extending the life of, or replacing, such a major structure if it deteriorates to an unacceptable level once in use. The durability issues which have become apparent with the existing Forth bridges underline this point.

Requirements for durable structures make recognition of the fact that durability is not an absolute property of a material but can be affected by both design and construction factors. Definitions of design life require that the design criteria are achieved, not that materials or components remain in the same condition unchanged for the design period and imply maintenance and some repair for its achievement. Thus an assessment of durability would require that the processes of deterioration be examined on the one hand and the means of protection (by durability design) and mitigation (by maintenance) are assessed on the other hand in order to ensure that the design life can be achieved with a reasonable degree of confidence. The design life of the structure will be 120 years. For the major structural elements this is usually interpreted to mean the design life without replacement, for other secondary elements, systems and components where replacement is feasible a shorter service life is usually assumed.

Whilst a full durability assessment of the structure has not yet been carried out a number of principles have been established as well as a number of potential measures to ensure adequate durability:

- Specification of appropriate materials and finishes
- Provision of comprehensive facilities for the inspection and maintenance of the structure
- Design for ease of replacement of secondary elements and systems (e.g. stay cables, bearings, movement joints, deck furniture etc.)
- Use of dehumidification where appropriate to protect the interior spaces of fabricated steelwork (e.g. deck, tower anchor boxes)
- Special measures to protect the reinforcement in the outer layers of reinforcement in the intertidal and splash zones of the towers and piers to extend the life of these structures in the most aggressive of microclimates. At this stage, stainless steel reinforcement has been assumed in these areas but provision for cathodic protection is an alternative to be considered.
- Use of stainless steel guide pipes and/or facia plates in the upper tower to reduce the maintenance requirements for these high elevation and difficult to access locations

10.2 Inspection & Maintenance

A comprehensive set of facilities for inspection and maintenance of the structure will be included in the design. In addition to fixed access facilities throughout the bridge (walkways, stairs, ladders etc.), a suite of motorized access machines will be recommended which may include under-deck inspection gantries, lifts within the towers, an internal deck shuttle (for the Three Corridor Option), a stay cable inspection gantry and an access platform to be suspended from a Building Maintenance Unit at each tower top.

The overall approach is to ensure that, as far as possible, normal inspection and maintenance activities can be carried out with minimum disturbance to the traffic. At the same time easy and safe access for maintenance personnel must be provided.

10.2.1 Three Corridor Option

The access facilities for the Three Corridor Option cable stayed bridge are illustrated in Drawing FRC/C/076/S/501 in Appendix B.

The primary access for the bridge is for maintenance vehicles to drive along the walkways on either side of the bridge and park close to the work area in the same way that access is achieved to the existing road bridge. This completely avoids the need for parking on the hard shoulders of the main carriageways for routine inspection work.

To facilitate this, designated cross passages will be provided in the bridge at approximately 90m intervals. Each cross passage will consist of a secure and weatherproof hatch in the walkway to allow access to the interior of the bridge deck. Ladders and transverse walkways will be provided connecting the hatches on either side of the bridge to two additional hatches in the structural zone reserved for the stay cable anchorages. In that way personnel can access the structural zone between the motorway and the multi-modal corridor at any point on the bridge in complete safety with no disruption to traffic.

Inside the bridge deck a motorised shuttle and a pair of walkways allow for longitudinal movement of personnel and equipment.

The soffit of the bridge will be inspected and maintained by under-deck gantries. Three separate gantries would be provided thus allowing the gantries to pass the towers and piers and maintain the full length of the bridge. On the existing road bridge, in common with many other bridges, there is provision for personnel to access the gantry from the top surface of the bridge deck. This is incompatible with the anti-climb windshields and therefore access to the gantries will be only from the towers below deck level. However, in the event of mechanical failure of a gantry away from the towers access is still required in order to evacuate personnel and repair the gantry, This is solved by the operation of three independent gantries and provision will be made to move safely between the gantries allowing one gantry to “come to the rescue” of another.

Within the towers the main inspection and maintenance requirements are above deck level, to access the stay cable anchorages as well as lighting and instrumentation in or at the top of the tower. A rack and pinion lift will be provided from deck level to the tower top with a minimum capacity of 5 persons together with emergency ladders for escape in the event of mechanical failure of the lift.

10.2.2 Double Level Option

The access facilities for the Double Level Option cable stayed bridge are illustrated in Drawing FRC/C/076/D/511.

As for the Three Corridor Option the primary access for bridge maintenance is along the walkways on either side of the upper deck. Maintenance access for the lower deck will require traffic management measures to be in operation albeit that this will be relatively easy for the multi-modal corridor.

The truss structure will be dehumidified and therefore frequent internal inspection will not be required. However, some internal access may be required.

An pair of underbridge inspection gantries will be provided for the inspection and maintenance of the external parts of the truss. Upper levels to the gantry will also provide access to the stay cable anchorages and the underside of the upper deck cantilevers. Access platforms on hydraulic arms will provide access to the underside of the lower deck soffit whilst allowing the gantry to pass a pier and thus run the entire length of the bridge.

Inspection and maintenance of the central part of the underside of the upper deck will be via mobile hydraulic platforms running on the lower deck. Access within the towers will be similar to that proposed for the Three Corridor Option with lifts being provided in each tower leg.

10.3 WASHMS

Complementary to the physical inspection and maintenance facilities, a Wind and Structural Health Monitoring System (WASHMS) is proposed to provide real time data and also to allow investigation of the structure to be undertaken after an extreme event such as a major wind storm or an earthquake.

The real-time data can be used to assist with inspection and maintenance by immediately highlighting anomalies that could indicate a fault (for example oil-pressure out of range on hydraulic buffers) or else by tracking long term changes in bridge behavior (for example gradual increase of effective friction coefficient on bearings).

A four level system architecture is envisaged:

1. Data Collection Level

   System collects data from sensors and forwards to pre-processing.

2. Data Pre Processing and Transmission

   Data Acquisition Units (DAU's) distributed through the bridge pre-process the data prior to transmission to central processor (signal conditioning and conversion of analogue data to digital). As well as the fixed DAU's, there can also be a system of Portable Data Acquisition Systems which can be used in conjunction with removable accelerometers for specific field vibration measurements.

3. Data Processing and Analysis Level

   Collection, processing, analysis, display, archiving and storage of all data by a centralized Data Processing and Control System.

4. Structural Health Evaluation Level

   Analysis and interpretation of measured data, comparison with criteria for inspection and maintenance, display archive and storage of analyzed or interpreted results, production of structural health evaluation reports.

The range of sensors that can be included at the data collection level is very extensive and may include monitoring of climactic conditions external and internal to the bridge, structural displacements, accelerations and strains, direct measurement of chloride ingress into concrete structures, traffic measurements etc. as well as sensors specific to any bridge equipment that may be installed (e.g. to monitor stroke position, effective friction between sliding partners etc.). The detailed specifications of the WASHMS will be developed in consultation with Transport Scotland.

11 Preliminary Consideration of Assessment of Anticipated Departures From Standard

The UK is in the process of adopting Eurocode as the basis of structural design. The two year transition phase to the full use of Eurocode in the UK runs from April 2008 to April 2010 during which time Approval in Principle (AIP) documents for bridges may be submitted either in accordance with the old bridge code, BS 5400, or in accordance with Eurocode as modified by the UK National Annexes (NAs). Some National Annex documents remain unpublished. The design of the Forth Replacement Crossing will be in accordance with Eurocode, and a project specific Design Memorandum will document the design rules adopted and include supplementary rules which will complement Eurocode.

11.1 Functional Cross Section

Requirements for the main carriageways are based on a dual 2 lane urban motorway, but hard shoulders of 3.3m width, rather than the required 2.75m width will be incorporated.

The Multi-Modal Corridor when operated as HOV lanes does not correspond to a standard Road Type. It is proposed that the Multi-Modal corridor shall consist of 3.65m lanes separated by a central reserve incorporating a VRS. A total width between the faces of the central and nearside VRS of 6,000 mm minimum will be generally provided to allow additional width in the event of a vehicle breakdown.

A Departure From Standard is anticipated regarding the VRS set-back requirement at the central reserve from the value of 1200mm given in Table 4.11.13 of TD 27/05 for central reserves to a value of 500mm which is the setback required for the working width of the VRS. The justification for the departure is:

- Significant cost associated with provision of an additional 1.2m width of bridge deck for this long span cable supported structure
- The multi-modal corridor does not include an adjacent lane but does include an adjacent hardstrip. Furthermore the HOV usage will not include wide vehicles. Therefore vehicle positioning may be towards the nearside white line and the reduced offside VRS setback will not have a significant effect on driver behaviour and driver shyness.

A further Departure From Standard is anticipated regarding the VRS setback at the towers for the Three Corridor Option. The hardstrip is discontinuous at the towers and generally the minimum VRS setback would be 1200mm. A locally reduced value of 350mm is proposed. The justification for the departure is:

- Significant cost associated with provision of an additional 1.7m width of bridge deck for this long span cable supported structure
- HOV usage will not include wide vehicles
- The reduced value is localised and the general VRS setback will be compliant.

11.2 Post-tensioned grouted ducts

Internal post-tensioned grouted ducts are under consideration in two locations:

- transverse prestressing in the concrete slab of the Composite Deck
- longitudinal cantilever prestress in the deck of the Concrete Approach

A moratorium on the use of post-tensioned grouted ducts was lifted in 1996 but certain restrictions remain in place which are described in Interim Advice Note 47/02 which makes reference to the Concrete Society Technical Report TR 47 (2002). IAN 47/02 notes that the design of the post-tensioning system will be classed as an aspect not covered by standards, and subject to departure procedures.

A moratorium for precast concrete segmental construction using internal grouted tendon systems still remains in force. Although precast slabs are under consideration for the Composite Deck this moratorium is not relevant since the in-situ stitches allow effective splicing and continuity of the duct.

However, the moratorium is relevant to the longitudinal cantilever prestress in the Concrete Approach. Preliminary discussions with Transport Scotland indicate that a Departure From Standard to allow construction of the approach using internal tendons in precast segmental construction would not be approved. Therefore the design will be progressed assuming in-situ construction.

12 Conclusions and Recommendations

12.1 Cost Comparison

The conceptual designs proposals have been developed to scheme design stage to allow assessment of different scheme options. Structural quantities were derived in order to make a comparison of the relative cost of the different options. The results of the cost comparison are given below:

	Three Corridor Option				Double Level Option		
Deck	Orthotropic Deck		Composite Deck		2 Plane Warren Truss	4 Plane Warren Truss	2 Plane Vierendeel Truss
Approach	Concrete	Composite	Concrete	Composite			
Relative Cost	1.00	1.08	0.98	1.06	1.09	1.11	1.34

The relative cost is expressed as a proportion of the base option. The orthotropic deck with concrete approaches was selected as the base option (with a relative cost of 1.00) because it is the closest in configuration to the FRCS design.

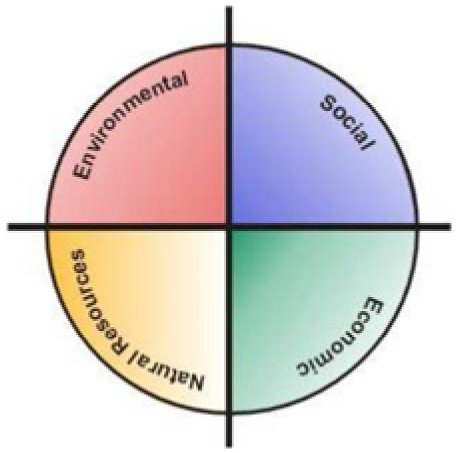
The above costs are for the Main Crossing from abutment to abutment exclusive of ITS systems. The relative costs do not include the transition structures required to stream the HOV lanes from the main carriageways to the multi-modal corridor.

The 2 Plane Vierendeel Truss option is shown to be significantly more expensive than all of the other options due to higher structural steel quantities.

12.2 Sustainability Comparison

Sustainability appraisal of the project has been included at the earliest planning stages. The FRCS studies identified 5 key indicators related to supporting sustainable development and economic growth. The main focus on meeting these key indicators was the promotion of sustainable transport modes which led to the inclusion of a multi-modal corridor on the bridge for future provision of trams or LRT together with designing the hard shoulders for peak hour bus operation.

Sustainability could also be considered as a differentiator in the later planning stages of the project to assist with appraising the relative merits of different options.



A sustainability appraisal should focus on the key elements of environmental protection, social equity, economic viability and efficient use of natural resources. However, many of these aspects have already been fixed as a result of the FRCS studies which have determined:

- The need for a bridge, the alignment and the basic form (double main span cable stay)
- The functional cross section in terms of number and usage of lanes

It is intended to carry out a Main Crossing Sustainability Appraisal at DMRB Stage 3 to confirm the suitability of the option carried forward to Specimen Design.

Considering the Main Crossing in isolation this assessment will be mainly focussed on the environmental impacts and depletion of natural resources associated with the construction methods and materials. One of the most challenging aspects will be to differentiate between steel and concrete as a more sustainable material in terms of future availability of resources and impact on climate change. Previous life cycle comparisons have not shown a clear and consistent preference. In some cases lower energy costs and emissions have been indicated for concrete bridges if new-won materials are considered. However, the relative ease of recycling steel compared to concrete changes the picture with lower energy costs and emissions associated with steel if recycled products are used.

At this stage several points are noted:

- The surface area of steel requiring painting is similar for all of the cable stayed bridge options
- The quantities of materials to be used in the cable stayed bridge are similar between the composite deck and the warren truss options. Therefore the sustainability of these solutions will be similar.
- The quantities of materials to be used in the orthotropic deck option are different with larger quantities of some items and reduced quantities of others. Therefore the sustainability of this solution may be better or worse than the alternatives.

12.3 Recommended Options for Further Development

A Main Crossing Options Selection Workshop was held on July 28th 2008 where the findings of the Scheme Assessment as documented in this report were presented and discussed. The issues discussed during the workshop and the conclusions are described below:

12.3.1 Comparison between Three Corridor and Double Level Options

The two different functional cross sections proposed at the conceptual design stage are both considered feasible options. However, a review of the relative merits of each option indicates the Three Corridor Option to be more favourable and it is recommended that this be carried forwards. The relevant criteria are:

(a) Multi Modal – Relative cost of transition structures in Phase 1 (HOV's)

Although a quantified cost comparison has not been made it is clear that the cost of transition structures to stream the HOV lanes from the main carriageways to the multi-modal corridor will be higher for the Double Level option. This is due to the grade separation between the main carriageways and the multi-modal corridor.

(b) Multi Modal – Driver Perception in Phase 1 (HOV's)

The grade separation is also likely to result in poor driver perception of the road layout for the Double Level Option since the lane transitions will involve curved ramp structures and minimum sight lines. In contrast the lane transitions for the Three Corridor Option will be on straight level sections of road with open sight lines.

(c) Multi Modal – Cost / Disruption to Modify to Phase 2 (Tram / LRT)

As noted in Section 4.5 the cost and disruption associated with modifying the approach transitions from the Phase 1 to the Phase 2 usage is anticipated to be significantly higher for the Double Level Option.

(d) Multi Modal – Phase 2 Operation (Tram / LRT)

Both configurations offer equally good operability of the trams or LRT in Phase 2 with the tracks located adjacent to each other.

(e) Relative Cost Excluding Transition Structures

The vierendeel truss option is excluded from the comparison since it can be rejected on cost grounds alone.

The work done to date indicates that if the composite approach were adopted for the Three Corridor Option then this would have a similar cost (1.06 to 1.08) to the warren truss options (1.09 to 1.11). Both the composite approach and the truss decks provide good visual continuity between the cable stayed bridges and the approaches.

However, the concrete deck offers the possibility of a cost saving for the Three Corridor Option, albeit at a slight aesthetic penalty.

(f) Tower Aesthetics

Any comparison on aesthetic grounds is somewhat subjective but a broad consensus has been built within the team that the towers of the Three Corridor Option are superior to the Double Level Option. Whilst the Double Level Option tower could be further developed into a reasonable solution it does not offer the instant appeal and excitement of the Needle Tower which is the preferred shape for the Three Corridor Option.

(g) Construction Programme

As described in Section 9.4 an initial assessment has been made of the construction programme which results in comparable total durations for both options assuming the truss units of the Double Level Option will be lifted in 24 m double length units.

(h) Environmental Impact

Environmental impact assessment of the two options has not been undertaken. However, given that the options comprise broadly similar structures, it is envisaged that their environmental impacts will be generally comparable.

(i) Sustainability

As noted above, a sustainability assessment has not yet been carried out. Although the material quantities are similar between the truss decks and the composite decks, the relative sustainability of the orthotropic and composite deck options has not yet been assessed.

(j) Maintenance

In broad terms the maintenance requirements of the different cable stayed bridge options will be similar. All of the options have the same basic elements of steel deck, proprietary stay cables and concrete towers. However, there are some minor differences.

Whilst the exposed surface area of steel is almost the same for the trusses and box girders, the large flat surfaces of the box girders will be easier to maintain than the multi-faceted small surfaces of the trusses. Furthermore, maintenance of some of the parts of the truss decks will require mobile elevated working platforms to operate on the lower deck which will be disruptive to the operation of the multi-modal corridor.

(k) Summary

A comparison table is given below. Where differentiation between the options is qualitative a three point scale is used: (Good – Fair – Poor). Reference should be made to the written explanations given above.

	Three Corridor	Double Level
MM - Phase 1 Relative Cost of Transition Structures	Lower Cost	Higher Cost
MM – Phase 1 Driver Perception	Good	Poor
MM – Cost / disruption to modify to Phase 2	Fair	Poor
MM – Phase 2 Operation	Good	Good
Relative Cost (excluding transition structures)	0.98 to 1.09	1.09 to 1.11 (excluding vierendeel)
Tower Aesthetics	Good	Fair
Construction Programme	Comparable	
Environmental Impact	Comparable	
Sustainability	Assessment to be made between Orthotropic and Composite Deck Options	Comparable to Composite Deck Option
Ability to maintain structure with minimal disruption to operation	Good	Fair

On the basis of this comparison it is recommended that the Double Level Option is not carried forward. The primary reason for this recommendation is the poor configuration of the multi-modal corridor for HOV usage in Phase 1. If the Double Level Option showed major advantages in other areas then further investigation could perhaps be justified but on all the other points of comparison the Three Corridor Option is shown to be comparable or superior.

12.3.2 Deck Type

As discussed in Section 5.1.4 it is recommended that both orthotropic and composite box girder decks be progressed with a view to offering both options to tendering Design & Build contractors. Further comparison between the deck types will include a sustainability assessment and more detailed proposals for the surfacing options on the orthotropic deck.

12.3.3 Tower Form

As described in Section 6.4.2 the Needle Tower better emphasises the aesthetic ideal of the single element piercing the blade like deck of the bridge. The recommendation of the Options Selection Workshop was that the Needle Tower should be developed in preference to the Inverted Y.

12.3.4 Approach Bridge Type

The cost comparison showed the Concrete Box Girder approach to be more economical than the wide Composite Box Girder approach. However, the Composite Box Girder approach gives a cleaner visual continuity of the cable stayed bridge into the approaches. It is recommended that both options are investigated further.

12.3.5 Foundation Type

Inclusion of the results of the marine ground investigation and the ship impact risk assessments will allow the proposed foundations to be further developed including more detailed assessment of the constructability of the precast pile caps and footings.

12.3.6 Conclusion

The following recommendations on which scheme options to develop further represent the consensus of the Options Selection Workshop attendees:

Functional Cross Section	Three Corridor Option	
Deck Type	Orthotropic and Composite Box Girder	
Tower Form	Needle Tower	
Approach Bridge Type	Composite Box Girder	Concrete Box Girder
Foundation Type (Towers)	Flanking Towers: Piled Central Tower: Pad Footing	

Appendix A - Reference Design Drawings