

Appendix C – Bridge at Corridor D

APPENDIX C BRIDGE AT CORRIDOR D

C1 INTRODUCTION

The purpose of this appendix is to provide technical background and additional information to assist Transport Scotland in determining the most suitable solution from the different crossing options available at the present time.

This appendix looks at the two preferred bridge options currently being considered for this project in one crossing corridor.

A number of other reports have been prepared as part of this study, in particular Report 3: Option Generation and Sifting published in December 2006. This report looked at an additional four crossing corridors (a total of five), and considered both a tunnel and bridge option for each of the crossing locations. From the findings in this report three tunnel and two bridge options have emerged as favourites. Both of these bridge options has been earmarked for further development at this stage. The development of the three preferred tunnel options has been covered in a separate technical annex.

By the very nature of the Firth being wide and the need to span over the navigation channels, any bridge crossing the Forth would tend to have long spans. Any final bridge would, in all likelihood, combine a long span or spans over the navigation channel combined with approach viaducts of shorter length.

The suitability and form of any bridge will depend on several design issues relating to constraints which include:

- Navigation channel width and headroom clearance;
- Underlying geology and its suitability for foundations;
- Environmental constraints;
- Archaeological and historical sites;
- Pipelines and electricity transmission cables;
- Urban developments; and
- Proximity to existing bridges.

The above constraints were presented in outline in Report 3. Both suspension bridges and cable stayed bridges would be able to accommodate these constraints and each are now outlined below. This appendix discusses in more detail the design of the bridge options, construction, maintenance, and cost issues.

This appendix deals first with general design issues and the manner in which they affect the bridge cross section. For both suspension and cable stayed bridges, further sections then deal with the following issues:

- General description of bridge;
- Detailed description of design issues including best practice;
- Detailed description of construction issues including best practice; and
- Construction programme.

C2 ISSUES AFFECTING BRIDGE CROSS SECTION

The following deck cross sections have been reviewed in detail and their implications on the design of the bridge have been incorporated:

- Dual 2 Motorway Standard (D2M); and
- Dual 2 Motorway Standard plus Light Rapid Transit (LRT) (bus or rail based).

It should be noted that the references to light rapid transit throughout this report are to illustrate the possibility of introducing light rapid transit and does not imply any commitment by Transport Scotland to such a proposal.

C2.1 D2M (Refer to Figure C1)

As a starting point, it has been assumed that the crossing is designated as an urban motorway with a maximum speed of 50 miles per hour, as the existing bridge. The justification for this classification is that the bridge is on the fringes of urban areas and it would be adequately lit. Based on current Highway Standards (TD 27/05), as detailed in the Design Manual for Roads and Bridges (DMRB), section 1, part 2, this leads to the following critical criteria:

- Lane widths to be 3.65 metres and each carriageway to be 7.3 metres wide.
- Hard shoulders to be 2.75 metres wide. The equivalent width for a rural motorway is 3.3 metres, but the proposed justification for urban classification is provided above. It is particularly important for a long span bridge to keep the overall width of the deck to a practical minimum as the overall cost of the bridge is roughly proportional to the width of the deck. As the width of the deck increases the dead load increases, and this in turn leads to an increase in size and cost of all the major structural members. For comparison, the Second Severn Crossing has hard shoulders with a width of 2.6m.
- Central Reserve to be three metres total width.

- Safety barriers at the edge of the carriageway with a working width allowance of 1.3 metres clearance to the nearest face of the lighting columns.
- An access way with a minimum width of 2.6 metres to be provided between the lighting columns and the structural hangers. This access way would also act as a combined footway/cycleway. Guard rails would be provided each side of the access way. The rails around the hangers would be provided with anti-climb mesh to prevent vandalism to the hangers and would be boxed around the hangers. The access way would allow routine inspection and maintenance work to be carried out without the need for carriageway restrictions or hard shoulder closures. Wind-shielding two metres in height would be provided with parapets at the edge of the bridge.

Since the bridge is to act as a replacement crossing, provision in the design will be made for pedestrians and cyclists to cross the bridge. The resulting full width of bridge deck is approximately 40 metres.

For the case where the bridge was to be an additional crossing, it would be reasonable to consider that the existing bridge would continue to provide access for pedestrians and cyclists. The pedestrian guard rails each side of the access way could be eliminated, and the access way reduced in width. The clearance between the lighting columns and the structural hangers would be maintained as a minimum dimension of 2.6 metres at the narrowest cross section between the lighting columns and the hangers. The overall width of the bridge deck would be approximately 1.2 metres narrower than the deck incorporating provision for pedestrians and cyclists.

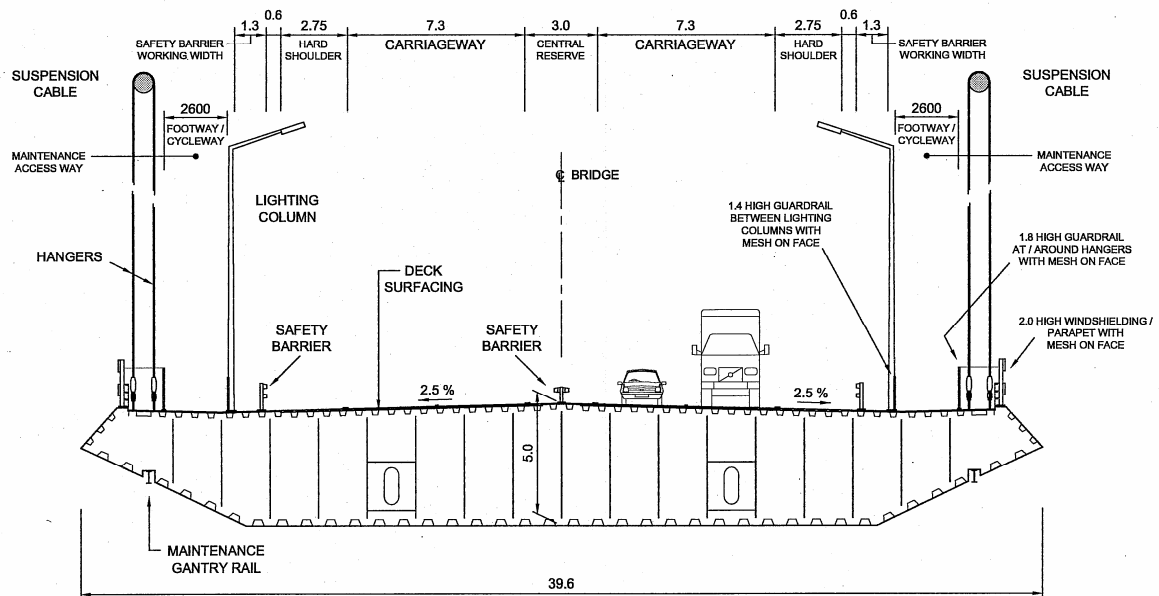


Figure C1 - Deck Cross Section for Dual 2M

It should be noted that whilst a 2.75 metre wide hardshoulder is appropriate for this design standard it could not support the use of hardshoulder running by public transport modes or HOVs. If this was necessary (and this is described in Chapters 8 and 9 of the main Report 4) then the hardshoulder would need to be 3.3 metres wide.

C2.2 D2M plus LRT (Bus or Rail) (Refer to Figure C2)

Light Rail

As part of this study, potential schemes for accommodating bus or light rail on the bridge options have also been examined. For the purposes of this report, the study has only looked at the changes to the bridge deck cross section.

In addition to the road considerations outlined above for the D2M scenario, the requirements for light rail have been reviewed separately against the principles outlined in the “Railway Safety Principles and Guidance” (RSPG) Part two.

Light rail could be accommodated on a new bridge structure in several ways. In the section below several options are briefly described, along with the associated effects on the bridge. These comments apply to both suspension and cable stayed bridges.

Single Deck

A light rail system could be accommodated either at the centre, or at the edge, or outside the road carriageway. For light rail located at the edge of the bridge this would occupy the zone immediately adjacent the structural hangers. This option would move the extremely valuable maintenance access way from the area where it would be most required for the purpose of maintaining the hangers and main cable for the suspension bridge, and the cable stays for the cable stayed bridge. For this reason this option has not been explored in any more detail.

A further option was considered in which the light rail could run outside the safety barriers at the edge of the carriageway. At the ends of the bridge one of the light rail tracks would need to cross under or over the two carriageways in order to tie in to the transport network. The equipment serving the two tracks would need to be duplicated in order to serve the two remote tracks.

A third option would be to locate light rail at the centre of the bridge. All the essential equipment serving the rail could be located in one zone, and the trains accommodated more compactly, thus keeping the overall deck width to a minimum. At the ends of the bridge the two tracks could be accommodated on a single structure, bridging over one carriageway or passing under one carriageway in order to tie into the transport network.

Preliminary inspection of the road network, particularly at the north landfall, indicates that the option of locating the trains at the centre of the bridge would be significantly less complex than locating trains outside the carriageways.

Double Deck

The introduction of light rail may lead to concerns regarding the visual interference of mixing road and rail traffic on the same bridge deck. A possible solution would be to provide a double deck design in which the light rail or tram could run underneath the road traffic. Another possible advantage of this type of deck is that it keeps the width of the deck as narrow as possible, reducing the land take at each end of the bridge. In turn this also leads to a reduction in the width of the main towers, piers and their foundations.

A box girder would no longer provide the best solution. It could be replaced by a truss system, both longitudinally and transversely. The sides of the deck could be enclosed by a structural cladding system in order to provide a streamlined shape for aerodynamic stability. Ventilation would need to be introduced, possibly along the centre-line of the bridge deck. Within the box, emergency access routes would be run either side of the trams.

As described above, the width of the bridge would be as narrow as is possible for a multi-modal solution, and the overall width would be similar to the D2M option. In order to accommodate the train loading and provide sufficient headroom, the depth of the deck section would need to be increased. For comparison, the Tsing Ma Bridge, with a similar main span of 1377 metres and deck width of 41 metres, has a deck depth of 7.6 metres. This main span is 2 metres greater than that which is envisaged for a suspension bridge at this location. This increase in depth would affect the vertical alignment of the approach roads, and lead to an increase in cost of the approach embankments.

The increase in depth of the deck would lead to an increase in weight, which would in turn affect the design of the towers, cables, hangers and foundations.

As a result of these factors, this option was rejected and cross sections were developed for the rail located at the centre of the bridge running on top of the deck.

The following assumptions were made in order to derive the appropriate deck cross section width:

- The swept envelope width of 2.81 metres based on a train width of 2.65 metres. The swept envelope is the allowance made for several effects, including the displacement of the train due to its suspension, tilting, tolerances in track due to wear, and for the effects of horizontal curves in the bridge and track wherever applicable.
- Clearance of 0.35 metres between the train kinematic envelopes, plus an allowance of 0.25 metres for the width of trail mirrors was assumed to derive the separation of the trains.
- Based on the RSPG an allowance of 0.6 metres was made for the clearance between the train mirrors and the traction poles.

Based on these assumptions the resulting overall deck width is approximately 50 metres.

Buses (including Guided Buses)

This study has also looked at the possibility of providing other forms of light rapid vehicles, such as buses and guided buses.

Guided buses are fitted with small guide wheels close to their main road tyres that engage with raised kerbs laid along a smooth road surface - the guideway. The raised kerbs are usually placed on either side of the track and enable buses to travel where width is limited, as there is no need to allow for any natural sideways wander. In typical applications at grade these tracks would be constructed from concrete, formed into “L” shapes. Away from the track the bus uses the normal road, just like any other rubber tyred road vehicle.

If guided buses were to be introduced on the new bridge, the lane would need to be dedicated to their use. For the same reasons noted above for light rail it is proposed that guided buses use the centre lanes of the bridge.

If concrete tracks were to be introduced, this would impose significant increase in weight on the bridge. Alternatively, the tracks could be constructed from steel. The running surface would need to be provided with a suitable surfacing material. The steel track would also impose additional weight on the bridge.

The possibility of removing the mastic asphalt surfacing over the width of the guided bus lane has been briefly considered. This would provide a saving in weight. However, the composite action of the surfacing with the steel deck plate for control of the fatigue life of the plate would be lost. In addition, the steel would be exposed to the environment and would be subject to the same painting regime as the rest of the bridge. It is concluded that the mastic should remain.

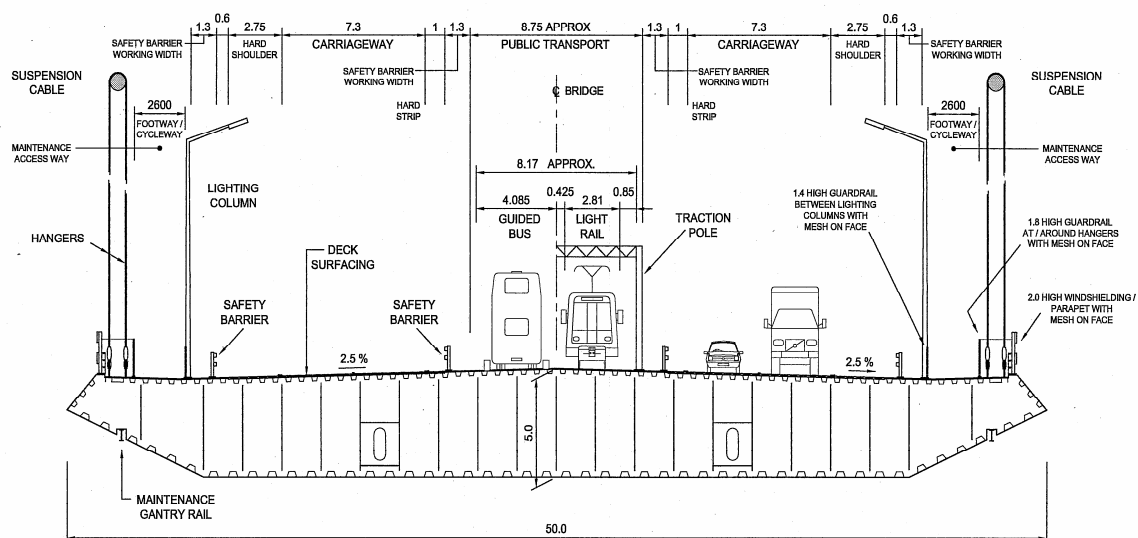


Figure C2- Deck Cross Section for Dual 2M Plus LRT (Bus or Rail)

C3 SUSPENSION BRIDGE OPTION

C3.1 General Description and Design Issues

Further development work has been carried out to review the suspension bridge option proposed for Corridor D in Report 3 and the STAG Part 1 Appraisal, in Chapter 3. This work has reviewed the optimum span for such a bridge, and reviewed the basic design assumptions. In order to verify the costs of the bridge, preliminary design has been carried out to determine preliminary quantities of steel, concrete and other civil engineering materials that would be required.

In addition, a detailed cost estimation exercise has been carried out for the bridge superstructure, including the main deck, cables, and hangers. The bridge superstructure represents approximately two-thirds of the cost of the bridge. It was considered that this was a reasonably significant proportion to study in more detail. The quantities for the substructure, which includes the main towers, piers and foundations, were recalculated and checked against estimates made in the 1990s. The rates for each structural item making up the substructure were inflated in accordance with the ROADCON Tender Price Indices, and checked against recently returned tenders. The overall cost of the bridge, plus its approach viaducts, was finally reviewed and compared to the final out turn costs of the world's major suspension bridges.

Two main bridge span arrangements were developed to illustrate potential costs. The most likely option to be taken forward would be 1375 metres main span option similar to that proposed in Report 3. As a comparison for costing and environmental issues a bridge with a reduced main span of 1200 metres has also been considered.

The 1375 metres main span bridge would run from the northern end of a toll plaza (if required) between Linn Mill and South Queensferry, over Beamer Rock to Cult Ness headland between St Margaret's Hope House and the Queensferry Lodge Hotel. The overall length of the bridge would be approximately 2.2 kilometres, with a main span of 1375 metres and two equal back spans of 416 metres. In addition, the southern approach viaducts would be approximately 400 metres long, consisting of six spans. The northern approach viaducts would be approximately 94 metres long, consisting of two spans. The cable dip between the tops of the towers and the mid-point of the deck would be 125 metres. The vertical alignment of the bridge would provide a minimum vertical clearance of 45 metres above mean high water spring tide level, which is the same as that provided by the existing Forth Road Bridge (FRB).

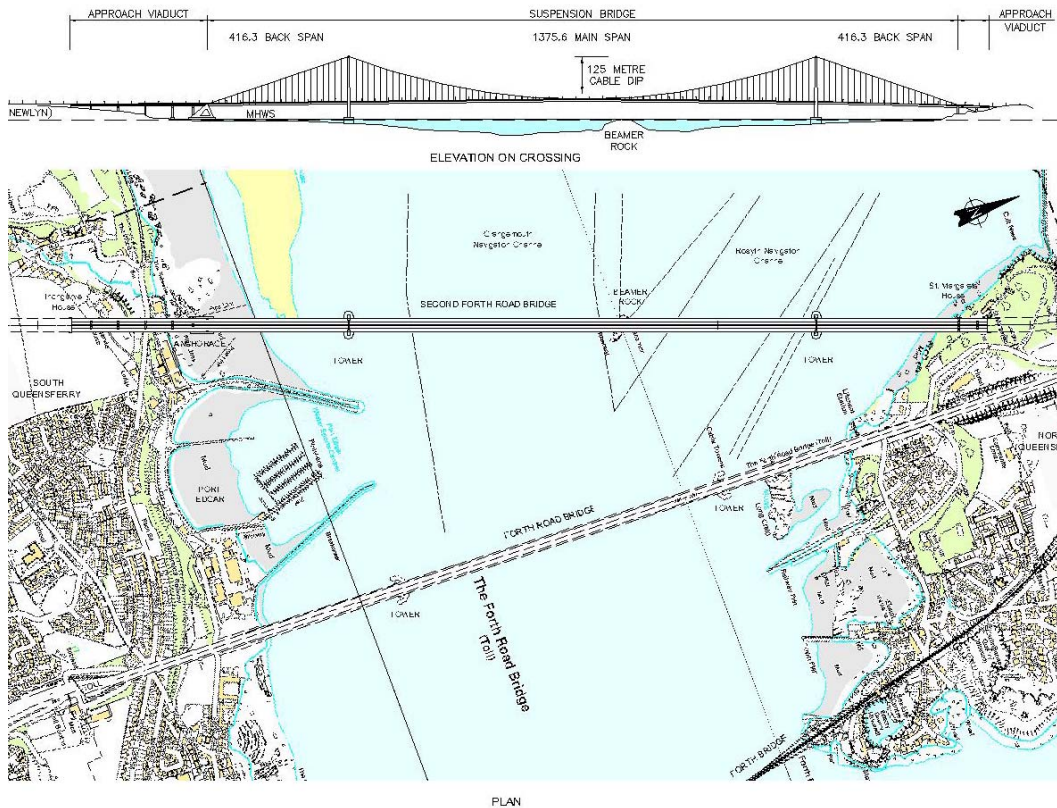


Figure C3 – 1375 metres Main Span Suspension Bridge

The proposed towers would be 184 metres high and constructed in reinforced concrete. The span arrangement allows the Grangemouth and Rosyth navigation channels in the Firth of Forth to be crossed by a single span, and avoids the requirement for an anchorage structure for the cables in the Firth of Forth. The towers would be approximately 30 metres higher than the FRB.

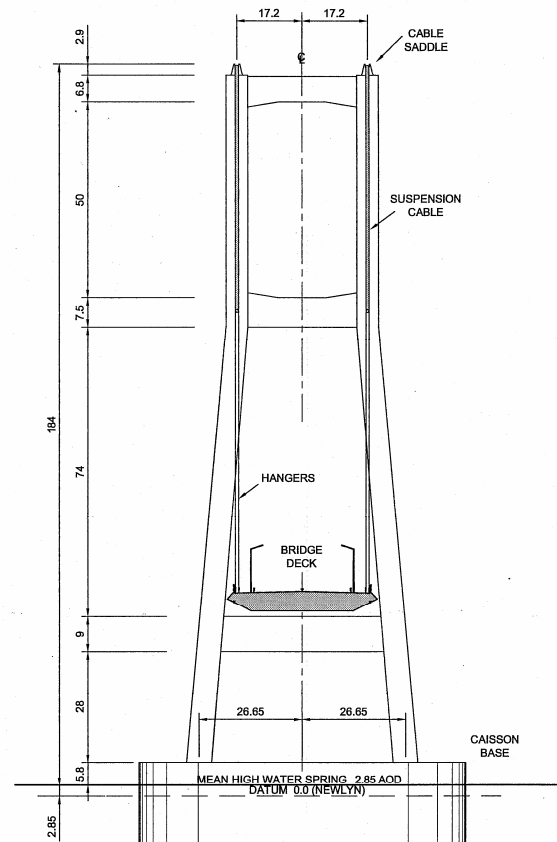


Figure C4 - Suspension Bridge Main Towers

The towers would be founded on reinforced concrete foundations taken down to the mudstones and sandstones below the soft alluvial and glacial deposits in the Firth of Forth. The southern tower foundations would be constructed to a depth approximately 43 metres below water level, which is approximately 34 metres into the soft deposits. The northern tower foundations would be of similar depth, although as the water is about four metres deeper at this location less excavation would be required. The plan size of the foundations should be designed to meet the requirements for shipping impact, both in terms of having adequate mass to resist the impact from a vessel, and to prevent the vessel overrunning the top of the foundation and impacting the tower leg.

During the construction of the south anchorages for the FRB an explosion occurred at the bottom of the tunnels. This was believed to be due to a sudden and unexpected release of methane gas into the workings. This is an extremely important design consideration which must be incorporated into the design and construction of any new suspension bridge. Therefore, it is proposed to use gravity anchorages to reduce excavation and hence minimise construction risks. The gravity anchorages resist the tension force in the cable through its own weight and resistance to sliding. The gravity anchor would therefore be large and quite likely to be visually intrusive.

At the north landfall two possible methods have been considered, as there is also a potential risk of methane within the rock. In one method, the main steel suspension cables would be anchored into tunnels excavated into the rock. The tunnel would be approximately 80 metres long and at their deepest would be approximately 40 metres below sea level. At the northern landfall the proposed entrance to the anchorage tunnel would be in Cult Ness headland, and the tunnel would extend down into the outcropping rock. At the top end of the tunnels, anchorage chambers would be constructed from prestressed and reinforced concrete. This proposal has been assumed for this study. However, there is evidence, from recent construction work, that there is a possibility that methane gas is also present on the north side and hence a second option could include for gravity anchorages on the north side. The costs for the bridge allow for gravity anchorages at both the north and south ends.

Permanent access to the bottom of the anchorages would be required for inspection and maintenance.

C3.2 Global Structural Behaviour

Preliminary analysis of the 1375 metre suspension bridge has been carried out using two dimensional in-house computer program. The analysis and design of a suspension bridge is extremely complex at its detailed design stage. It depends, amongst other criteria, upon the interaction of the span, height of towers, and cable dip. As the cable dip for a given span decreases the following general trends result:

- The angle of the cable to the horizontal at the towers decreases and the force in the cable increases. This leads to increased size and cost of the main cable and potential increase in construction time;
- The increase in cable forces also leads to increase in anchorage size;
- Reduced height and cost of the main towers. This also leads to potential shortening of the construction time;
- Increase in stiffness leading to reduction in deflection and rotations of the main deck under live load. This is particularly important for options involving light rail, as rail traffic is more restricted by deflections and rotations of a bridge, particularly at expansion joints; and
- In addition, the longitudinal movements of the bridge due to live load are reduced. Again this is important for bridges supporting light rail.

As can be seen from the above, there are advantages and disadvantages for reducing the cable dip. However, another consideration is that of aesthetics and visual impact. For this stage it was concluded that a cable dip of 125 metres should be adopted giving a span to dip ratio of 1 in 11. The cable dip and the height of the towers should be reviewed and increased, if necessary, for the D2M plus light rail option as the cable area is large.



Figure C5 - Deflected Shape of Suspension Bridge Part Loaded in Main Span

C3.3 Deck Girder and Bridge Articulation

The deck girder, as well as providing the running surface for traffic, also participates in distributing live loads along the length of the bridge to several hangers. In earlier twentieth century suspension bridges the deck tended to be supported by stiffening trusses. These bridges include Golden Gate Bridge in San Francisco as well as the Tamar and the existing FRB. However, latter day suspension bridge decks, dating from the 1960s, tend to be fabricated as a steel box girder with suitable shaping at their leading edges to improve their aerodynamic stability. The Severn and Humber suspension bridges comprise steel box girder decks. The steel boxes are easier to paint than steel trusses and hence reduce long term maintenance costs during the life of the structure.

Traditionally, movement joints were provided in the deck girder at the main towers to permit temperature induced movements. The maintenance of bearings and movement joints is a disruptive and expensive activity and it is now common practice to reduce the number of joints. It is current best practice for suspension bridges to make the bridge deck continuous through the towers. This has a disadvantage in that it concentrates the deck movements at the movement joints at each end of the bridge. Another disadvantage of the continuous deck through the towers is that the forces in the deck become very large. One method of reducing these loads is to increase the hanger spacing at the main towers, thus making this section more flexible. In addition, the removal of vertical support at the tower locations also helps the deck girder.

The deck girder would be typically supported by hangers at approximately 18 metres centres measured along the length of the bridge. The final spacing of the hangers would be decided at the detailed design stage. For comparison, the existing FRB has hangers at approximately 18 metre centres. Storebaelt East Bridge in Denmark has hangers at 24 metre centres. The deck box girder would be constructed using steel plates strengthened by shaped troughs welded to the plate. Using the preliminary analysis, it has been established that the box depth would be five metres deep. The top deck plate would typically consist of 12 to 13 millimetre thick plates with six millimetre thick troughs. These are typical steel plate thicknesses used in suspension bridge construction, and were used for the construction of the Storebaelt suspension bridge. These plates would be thickened locally at the main towers. The bottom deck plate would be typically constructed using 10 millimetre thick plate strengthened with six millimetre troughs. The sloping web plates would be constructed in a similar manner, but the upper sloping plates would be thicker in order to transfer the high shear forces and loads at the hanger connections. The approximate average tonnage for the deck is 16.4 tonnes per metre along the length of the bridge.

Buffers would be provided at each end of the bridge deck in order to limit longitudinal movements arising from wind and traffic loads. These would be located between the anchor blocks and the deck girder on the south end of the bridge. On the north end of the bridge, if a tunnelled anchorage is used, additional strength would be required in the side tower to resist the buffer loads, or the load could be transferred through the deck of the approach span. They allow longitudinal movements due to temperature but provide a dampening effect on movements from traffic and wind. This would reduce wear and tear on the bearings and joints and reduce maintenance costs.

At the centre of the main span, the deck and main cables would be linked to reduce differential movement and bending of the short hangers at the centre of the main span. They also work together with the buffers referred to above by transferring the load from the deck into the main cables.

C3.4 Main Cables

For the D2M option, the main cables would be approximately 34.5 metres apart and would be supported on steel saddles on top of the main towers. The maximum cable force is typically at the end panel of the side span where the cable is at its steepest. In the detailed design, if it is found to be economical, it could be possible to anchor some of the side span wires at the top of the main towers to reduce the cables across the main span.

There are two main methods of erecting the main cables – Aerial Spinning (AS) and Preformed Parallel Wire Strands (PPWS).

In AS cables, the cables are erected by running individual wires between the anchorages and over the main and backspan tower saddles using spinning wheels. The individual wires are subsequently compacted together and wrapped to prevent corrosion. In this operation an aerial ropeway and catwalk working platform would be provided crossing from one shore to the other and passing over the saddles at the top of the towers. This is a high risk activity and can typically take three to four months to complete. It is on the critical path and therefore any delays would directly affect the overall programme.

High strength wire would be delivered to an unreeling shop at one end of the aerial ropeway where it would be prepared for installation. A number of wires (typically four) can be installed at the same time, and these are carried along the aerial ropeway on 'spinning wheels' to the far anchorage where they can be secured.

Cable spinning operations can be sensitive to adverse weather conditions, particularly high winds, and would be likely to occupy some 16 months of the construction programme.

Once the main cable was in place, the individual wires would be compacted together to form a tight group and be secured together by cable bands. A further outer layer, comprising a surface coating and a wrapping of galvanised wire, is then applied around the cable to act as a protective covering. This covering is painted to offer further protection.

Employing lessons learnt from the FRB, and examples of more recent suspension bridges, dehumidification systems would be installed within the cables during construction and used throughout the lifetime of the bridge to minimise corrosion to the cables.

In PPWS, the anchorages are likely to be larger and hence more costly than for AS cables. However, this method is slightly less prone to weather and poor visibility risks than aurally spun cables. The expected quality of a PPWS cable is generally higher due to reduced wire and galvanising damage during erection.

PPWS typically uses strands with 127 wires grouped together in a hexagonal shape approximately 60 by 70 millimetres. The appropriate number of strands would be lifted into place on the temporary footbridge and then into position supported on the main tower saddles.

The advantages and disadvantages of each system are set out below:

- A significant number of site operatives would be required for AS. Approximately 100 site operatives would be required for each of the two daily shifts. In addition, supervisory staff would also be required. This large number of site operatives would lead to a significant labour cost.
- PPWS would require approximately 40 operatives for each of the two daily shifts, and hence this leads to a saving in the labour requirements.

- PPWS would be more expensive in fabrication, but quality tends to be improved as the strands are fabricated under factory conditions.
- PPWS would require heavier handling equipment.
- Anchorages for PPWS would be larger due to the larger number of strands.
- Saddles are more complex for PPWS as they require more machining to accommodate the strands.
- In AS main cables, numerous splices are required to connect the individual wires. However, the presence of splices makes compaction of the cable more difficult. Splices at saddles and cable bands must be avoided on aerial spun cables. In PPWS cables, the number of splices is significantly less, and hence the quality of the compaction would be improved.
- The maximum force in the cable is located typically in the first side span panel closest to the main tower. If it proves economically feasible, it is possible with PPWS cables to terminate some of the individual strands at the main tower. This leads to a reduction in the number of wires in the cable supporting the main span and could lead to a cost saving. This potential saving has not been included in the costing included in this report.

It can be seen from the above list that there are advantages and disadvantages for the use of either system. Therefore, at this stage, both systems have been considered and comparative costs developed. From the work carried out, as the number of wires increases the AS system becomes the cheaper system. However, the final decision needs to take into consideration the likelihood that the better quality cables would be achieved through the use of PPWS.

For both systems, the cable would be compacted and wrapped in galvanised wire. It is widely considered that the most effective system for providing resistance to corrosion currently available is 'S-shaped' wire available in Japan. For a bridge of this size and quantity of wire, there is a reasonable likelihood that the wire could be manufactured economically in the UK provided that potential patent issues could be resolved.

C3.5 Hangers and Cable Bands

Hangers are typically fabricated from locked coil wire rope (individual concentric wires twisted about core wires) or preformed parallel wire strands enclosed within a high density polyethylene (HDPE) sheathing. The sheathing provides improved corrosion protection, and a reduction of the drag coefficient, which reduces the wind load picked up by the hangers.

The hangers would be connected by cable bands to the main cables. There are several methods of forming the cable bands, and these typically consist of clamps formed in two halves. The clamps can be connected to the main cables either by vertical or horizontal bolts. Different design approaches have been considered on the world's major bridges, and there are advantages and disadvantages for each method. However, there is evidence that vertical bolts have possible corrosion problems. At this stage, it has been assumed that the likely solution would incorporate horizontal bolts. It has also been assumed that the hangers would effectively be formed from a pair of hangers connected to the clamp passing over the main cables. Although this is a common method, it does have the disadvantage of taking up a wider space at its lower connection with the deck so increasing the deck width by approximately one metre. This could be reduced by careful detailing at the connection to the cable band.

The hangers could be connected to the deck through anchor sockets fixed to bulkheads inside the deck. Due to the limited space inside the box, replacement of the hanger ropes would need to be carried out from the outside using the maintenance access way.

C3.6 D2M Plus LRT and Influence of LRT on Design of Suspension Bridges

The deck cross section for the D2M plus LRT is described in section C2.2 above. In addition to the additional train loadings that may occur if the LRT was rail-based, the bridge would be subjected to increased loading associated with the trackform, signalling and overhead power equipment.

C3.7 Deck Girder and Bridge Articulation

For the design of a long span bridge, the addition of any rail loading would have serious implications. In this section, a brief review of worldwide experience is carried out, and the structural issues discussed.

The consequences of placing rail loading on a suspension bridge are very significant. Suspension bridges are very well suited to carrying loads placed uniformly along whole spans, but are not so good at carrying short intense loads such as the weight of a train. The cables carry nearly all of the imposed weight, and tend to deflect sharply where such a load is applied. The stiffening girder counters this effect by smoothing out the load transfer into the cable. Increasing the stiffness of the stiffening girder helps to achieve this. The deformation of a railway bridge under live load is a critical issue and hence the bridge design may not be governed by overall strength alone. As a train crosses a suspension bridge, it finds itself in a dip, having to climb out of that dip all of the time.

Some components of the bridge, such as the stringers beneath the rails, would be designed by fatigue considerations rather than strength.

The stiffening girders in the main and side spans of a traditional suspension bridge are often three separate elements supported on rocker bearings at the main and side towers. In this case, when both a side span and adjacent main span are loaded, both spans would deflect downwards leading to a cusp at the tower (Refer to Figure C6 below). This does not usually cause a problem for road traffic, but it can significantly affect the ride of a train due to the fact that movement joints would be required in the rail at the main tower locations. The solution is to make the stiffening girder continuous, but this has the disadvantage of concentrating all movement at one or both ends of the bridge and complex expansion joints are required for the railway. This problem also exists at the ends of the side spans, where it is also likely that movement joints would be provided in both the bridge deck and the rail.

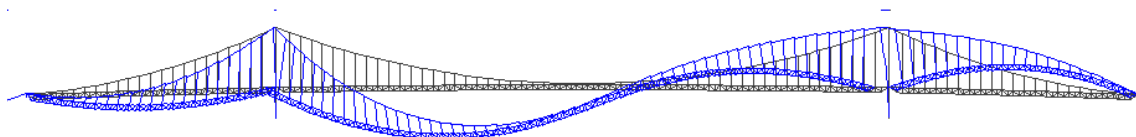


Figure C6: Deflected Shape of traditional Suspension Bridge (showing “cusp” at main tower)

For a suspension bridge carrying light rail, the effect of the train or tram is likely to be less than that for a bridge carrying heavy rail. However, the bridge would be subject to strict deformation criteria in order to limit the deformations at rail joints. Therefore, for the suspension bridge at Corridor D a continuous deck would be provided through the main towers, and movement joints would be provided at each end of the suspended structure. This would avoid the need for expansion joints at the towers.

As a result of the continuous deck, temperature movements in the deck would be concentrated at the expansion joints at the ends of the bridge. It is estimated that the movement would be approximately plus or minus 500 millimetres. The rail joints at these locations would need to be designed for these large movements as well as the angular rotations experienced in the deck as a result of bending in the deck. The shorter spans of the adjacent approach spans would help to keep the overall angular rotation in the deck and the rail to a practical level. However, the rail joint and the underlying structure would need to be specifically designed to cater for these movements. Possible methods of achieving this joint are as follows:

- The support structure underneath the rail could be designed to be sufficiently flexible such that the end rotation is smoothed out; and
- The movement and rotation parts of the joint should be separated such that the design of each part is simplified and the every day wear and tear on the joint is reduced.

As described above, bridges carrying rail are subject to several deformation criteria which are more onerous than those applied to a road bridge. In addition to the angular rotations described above, the bridge would need to be designed for the following effects:

- Impact factors arising from the deflection of the deck;
- Fatigue in the deck due to the passage of trains over the lifetime of the bridge; and
- Vibrations in the deck. This could be reduced by introducing resilient pads or similar between the track and the deck.

The safety of the rail system must be assured at all times. In order to achieve this, the vertical and horizontal accelerations at the level of the rail must be limited to ensure passenger comfort.

Passenger comfort must be achieved under everyday loading conditions. Preliminary analysis has shown that the deflections shown in Table C1 would be experienced.

Table C1 – Deflections and Rotations Predicted for Suspension Bridge

Type of Movement	Load Case	Magnitude of Movement
Mid-span deflection	One train at mid-span	0.4 metres vertical deflection
Change in alignment at movement joint	Maximum road and Rail Loading	0.026 radians rotation
Change in alignment at movement joint	One train at mid-span	0.0026 radians rotation

By comparison with other major suspension bridges across the world, these deflections and rotations are within the range of movements recorded at these bridges. For example, the mid span deflection for a train at the middle of the main span of Tsing Ma suspension bridge is of a similar magnitude.

Aerodynamic performance of the bridge.

Wind tunnel tests were carried out on the D2M bridge design in the 1990s but these have not yet been carried out for the D2M plus LRT option. Preliminary analysis indicated that the first natural frequency of the bridge in bending is 0.1 Hertz, which is comparable to Tsing Ma. Preliminary studies indicate that this would be acceptable. Similarly, the first torsional natural frequency has been estimated as 0.35Hz, and preliminary studies indicate that that this would be acceptable. For torsional behaviour of the bridge it is an advantage for the trains to be located at the centre of the bridge, as the movements experienced at the centre are much less than at the deck edge.

The deck girder would be of similar construction to the D2M option with similar plate thicknesses. The approximate average tonnage for the deck is 20 tonnes per metre along the length of the bridge.

C3.8 Main Cables

The main cables would be approximately 44.5 metres apart and would be supported on steel saddles on top of the main towers. They could be formed by either the AS method or the PPWS method as described for the D2M.

C3.9 1200m Span Bridge Design Issues

As part of the study, a suspension bridge with a main span of 1200 metres has been considered. This bridge would have two side spans of 350 metres and a cable dip of 110 metres. The main towers would rise approximately 170 metres above water level.

For this bridge, the southern anchorage would be a gravity anchor which would be located within the Firth. The southern approach viaducts would be 615 metres long and the north approach viaducts 190 metres long.

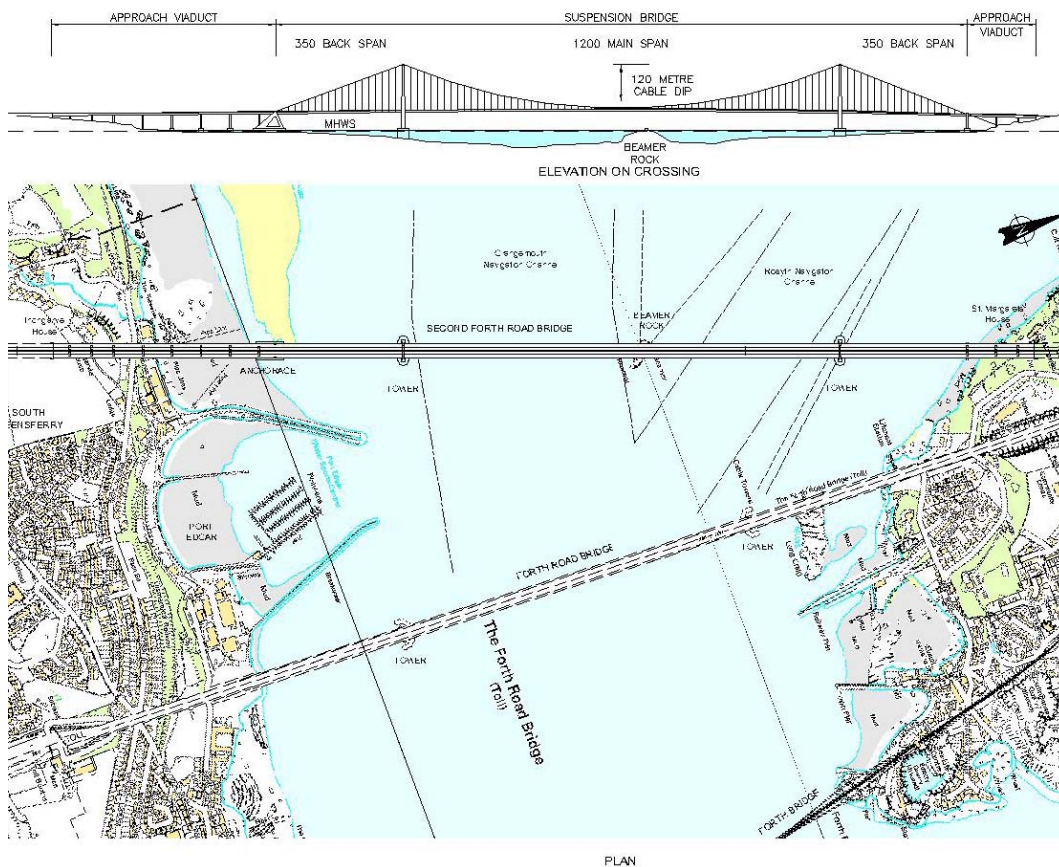


Figure C7 - 1200 Main Span Suspension Bridge

The advantages of this option are as follows:

- Reduced main span and side span lengths would lead to a reduced cost of suspension bridge;
- Reduced height of main towers would lead to reduced cost; and
- The anchorage would be less visible as part of it would be below the water line.

The disadvantages of this option are as follows:

- Increased length of approach viaducts;
- Increased blockage of the Firth due to the south anchorage;
- Main towers would be founded in deeper water and the depth to bedrock would be greater than for the 1375 metres main span bridge; and
- The main towers would be closer to the navigation channels, leading to increased risk of ship impact on the towers.

It was concluded at this stage that the net saving in costs compared to the 1375m main span bridge was minimal. Due to the increased risk of ship impact, combined with the increased environmental impact due to blockage of the Firth this option was not studied further.

C 3.10 Construction Issues Relating to Suspension Bridges

Construction Compounds and Assembly Yards

A number of construction sites would be required to service construction on both sides of the Firth of Forth, as well as the bridge itself. As the bridge would be the major construction activity, it is considered essential that at least one of the construction sites (of some 10 to 15 hectares) is located close to the Firth of Forth to enable the transportation of large precast or fabricated elements of the structure to their final locations. These elements may be precast concrete foundation caissons, or precast concrete and/or fabricated steel deck panels. In addition, marine access would be required for workers, plant and materials for a variety of operations within the Firth of Forth, including geotechnical investigations, foundation preparation or piling, placing and infilling of precast units and construction of bridge piers and towers. Accommodation would be required for at least part of an average workforce of some 800 (labour and staff) which may peak at some 1000 persons.

Initial estimates are that at least 30 hectares would be required to accommodate the following activities:

- Concrete batching;
- Steelwork assembly and painting;
- Materials storage;
- Plant storage and maintenance including fitters' shops;
- General storage, including fuel storage;
- Access to construction fronts, including jetties;

- Site offices and materials laboratory;
- Site welfare facilities, including changing rooms, canteen and first aid; and
- Car Parking and visitor centre.

The choice of construction sites would be the responsibility of the eventual contractor and would be significantly influenced by availability and access. In the latter case, access for the delivery of materials and components and access to the construction front for plant, labour, materials, and components are all important factors. Much of the construction activity would take place within the Firth of Forth, and it would be important that at least one of the construction sites has adequate berthing facilities.

Three possible methods of fabricating and assembling the deck panels have been considered in the development of the feasibility of the project. The final method chosen would depend on the location and the capacity of the contractor's own fabrication facilities, availability of construction sites and specialist plant, most economic fabrication and assembly process, including consideration of fabricating overseas.

The first method assumes that fabrication of the deck panels would be carried out in the UK. If the fabricator were located near a docks area it may be possible to ship the full width panels approximately 36 metres long to the Firth. It may be possible to install the surfacing and safety barriers before shipping. To facilitate this method it would be necessary to use an assembly yard closer to the bridge, and a possible site could include the Kvaerner Oil and Gas dockyard at Methil in Fife. This area has been used for fabricating and assembling offshore structures and would be well-suited for the storage and assembly of large deck panels. If transported by road the panels would need to be limited in size to satisfy width and weight restrictions. This would lead to intensive welding assembly operations and painting operations at Methil before the completed panels could be transported by barge to the bridge site. This is discussed in more detail below.

A second possibility could involve fabrication and assembly of panels within Europe before delivery to the construction site similar to the process adopted for Storebaelt suspension bridge.

In a third scenario, the successful contractor may elect to construct the panels abroad, possibly China. The complete deck panels, which could possibly include parapets and surfacing, could be shipped and delivered to an assembly yard at Methil, as above. The yard would be used for washing down the deck panels to get rid of salt accumulated during shipping, and a final top coat of paint applied. The completed deck panels could then be delivered to the bridge site by a smaller barge and lifted directly up to its final position.

In addition to the site at Methil the following sites have been considered. Some or all of these sites would be required to support the assembly yard at Methil.

South Bridgehead

A site could be located at the most likely site for the toll plaza (if required) at the south bridgehead. An area of approximately 24 hectares (which was purchase in the mid 1990s by the Scottish office) has been identified and it might also serve as the main construction site for the approach roads to the south bridgehead. This area would lie adjacent to the residential area of South Queensferry, and, therefore, any proposals to use this area would be subject to strict environmental control in order to minimise its impact on the adjoining residential properties. In addition, in view of the difficulty of access for major plant to and from the foreshore it is likely that this area would not be used for heavy construction operations; its main use could therefore be for accommodation and storage of materials.

Rosyth

An area of approximately 27 hectares to the west of the naval base has potential for a construction site. As part of the advance works by the Naval base for another project a construction site was prepared. The site could be used for location of a batching plant, precast concrete yard, steel fabrication yard as well as a storage area for the main bridge materials. For ship access to the site, it may be necessary to carry out dredging work.

The construction of a suspension bridge follows a generally linear programme, with little opportunity for concurrent working. The exception is that more than one tower or foundation can be constructed at the same time if the resources, particularly specialist plant, are available. The broad sequence of activities is as follows:

- Construct foundations and anchorages;
- Construct towers and backspan piers;
- Install main suspension cable;
- Erect cable hangers and deck units; and
- Install finishes (road surfacing, bridge deck furniture, motorway communications).

Foundations and Anchorages

It is anticipated that most foundations sited within the Firth would be required to bear directly on bedrock to satisfy design and performance criteria for the structure. The main bridge piers and foundation would be designed to resist ship impact. For lesser loaded structures, such as the approach viaduct piers and possibly also the cable anchor blocks, foundations may be founded on the stiff cohesive glacial deposits or the glacial sands and gravels. However, it is more likely that these would be piled.

In principle, the foundations would comprise the construction of a sheet pile cofferdam within which the reinforced concrete caisson could be constructed. The cofferdam may be sandfilled if it is considered that it is not possible to achieve an effective seal to permit dewatering of the cofferdam.

The construction of the reinforced concrete caisson would be carried out by sinking a precast concrete shell inside the cofferdam. The shell would be precast at the construction site, either at Methil or at Rosyth, and transported by barge to the bridge site. It would then be lifted off the barge by a jack up barge and manoeuvred into position. The material inside the shell would then be excavated, and, as the material is removed, the caisson would sink under its own weight. Construction of the shell would continue in situ in vertical lifts as the caisson sinks through the soft material. Once the caisson shell has reached its founding level, a concrete seal or plug would be cast to prevent the ingress of further water and the caisson then dewatered.

The main volume of foundation concrete could then be placed in stages, with the caisson shell providing a dry working area. Some of the lower concrete pours would only need to be mass concrete, while for upper parts reinforced concrete would be used to construct the foundation cap in order to provide adequate connection to the tower legs, and resistance to loads coming from them.

Towers and Backspan Piers

In Report 3, it was proposed to construct the towers using reinforced concrete. This decision was made primarily following earlier analysis of the towers when subjected to fire following a ship impact. The results of the analysis indicated that the fire could be of sufficient magnitude to cause significant damage to steel towers. It was therefore decided that concrete towers should be used for increased fire resistance.

In order to reduce the work in the Firth of Forth, prefabrication of the tower reinforcement cages and precasting of the tower cross beams would be undertaken at the construction site in Methil or Rosyth. These would then be transported by barge and lifted into place by the jack up barges.

The main tower reinforced concrete legs would be constructed using slip forming or jump forming techniques, which uses moveable formwork to form the concrete. As the towers rise in height temporary steel bracing would be required to stabilise the legs before the cross beams are stressed to the legs. The upper cross beams would be lifted into place using hoists supported on top of the tower.

Main Suspension Cables

The installation of the main suspension cables could commence when the anchorage, towers and back span piers were in place. As described in the section above covering the design, the two possible methods by which the cable can be erected are AS and PPWS.

For each method it would first be necessary to erect an aerial ropeway and catwalk working platform crossing from one shore to the other and passing over the saddles at the tops of the towers. The catwalk would be provided with a system of storm ropes to reduce sway, and to prevent damage during high winds. First, a pilot wire would be installed between the anchorages along the Firth of Forth bed. Barge mounted cranes would be required to lift the cable over Beamer Rock. The pilot rope would then be lifted to the tops of the towers by crane and hauled up by winch in order to provide clearance over the channel. It would be necessary to close the shipping channels for approximately six hours when the pilot rope was being lifted. The pilot rope would then be used to haul the strands supporting the catwalk. The catwalk would then be equipped with a deck surface, lighting, handrails and supports for the installation of the main cable.

For the AS method, high strength wire would be delivered to an unreeling shop at one end of the aerial walkway. It is most likely that this would be the southern end. The wire would be delivered in reels to the shop by road transport. It is likely that the equipment required to unreel the wires would straddle Society Road. In this event, the road would be adequately protected. A number of wires (typically four) could be installed at the same time. These would be carried along the catwalk on spinning wheels where they could be secured. This is a very labour intensive method and can employ approximately 100 site operatives per shift as described earlier. Typically, two shifts would be required to ensure minimisation of the spinning process. In addition, a large number of supervisory staff would also be required. An example of the aerial spun method is shown in Figure C8.



Figure C8 - Tsing Ma Suspension Bridge Main Cable (Aerial Spun Method)

In the alternative PPWS method, preformed parallel wire strands comprising 127 wires each five millimetres in diameter, in the form of hexagonal bundles, would be used. For the length of the bridge, each strand would be delivered in a reel weighing approximately 60 tonnes, and each reel would be seven to eight metres in diameter and 2.5 metres wide. The reels would be delivered to the southern end of the bridge by barge and hoisted up to position. The strands would then be hauled along the catwalk between the anchorages.

An example of PPWS method is shown in Figure C9.



Figure C9 - Jiangyin Suspension Bridge Main Cable (PPWS Method)

Fabrication and assembly of Deck Panels

As discussed above, fabrication and assembly of the deck girder panels could be achieved by several methods. The first method assumes that fabrication would take place in the UK. This could involve fabrication of the panels in sections, the dimensions being limited by transportation requirements. Typically, for transportation by either road or rail the panels would be limited in size to approximately four metres wide by 20 metres long. The individual panel sections would consist of a steel flat plate strengthened with profiled troughs which would be welded to the plate. Internal transverse diaphragms would be welded to the deck plates together with additional stiffener plates as required. The individual panels would be delivered to the assembly yard and then welded up into complete full-width deck sections. These sections could be up to 36m in length. The final design would take into account the availability of suitable lifting equipment for handling the completed sections. The fabrication process is specialised, but is well established in a certain number of suitably experienced fabricators throughout Europe and the rest of the world. Techniques would be put in place to compensate for distortions arising from the welding process. All processes should be carried out within procedures agreed with the client. The panels would also be inspected by a representative of the client at the fabricator's yard.

A second method could involve fabrication and assembly within Europe. In this method, full-width panels could be delivered to the site by barge. It may be possible to deliver these panels complete with parapets and surfacing.

To take account of fabrication facilities in China, the panels could be completely fabricated and assembled as described above and could be delivered to the UK by barge complete with parapets and surfacing.

During the detailed design process, it would be normal practice to design the deck girder panels such that the arrangement of longitudinal and transverse joints was designed to fit the available fabrication facilities and planned assembly methods.

A typical arrangement of the longitudinal joints for the D2M suspension bridge is shown in Figure C10.

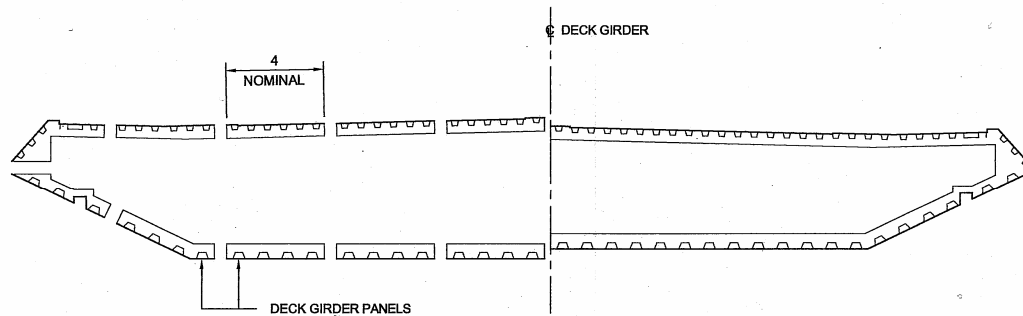


Figure C10 - Typical Longitudinal Joints for Assembly of D2M Deck Girder

After completion of fabrication and a final geometric check, the panels should be painted with a shop primer prior to shipping. During shipping the panels should be protected from salt contamination wherever possible.

Once the individual panels have been delivered to the assembly yard at Methil (or an alternative yard) the panels would be washed down and welded together to form the complete full width panel. Templates would be constructed on which the panels could be supported during welding to control the final geometry. Significant lifting equipment would be required in order to move individual panels including overturning panels as required.

Each completed deck panel would be matched with its adjacent completed section during pre-assembly in order to reduce assembly work at the bridge site, and to provide a check for the position of the hanger connections to the deck.

The final paint coats would then be applied prior to erection of the panels.

Erection of Deck Panels

The deck panels would be transported from the assembly yard to the bridge site by barge. First the girder sections would be loaded onto the barges using self-propelled multiple axle trailers. The barges would then be towed to the bridge site by tugs.

Prior to starting erection of the deck panels, the extent of the working areas and timing of the operations would need to be agreed with all interested stakeholders, including Forth Ports PLC, Rosyth, and Grangemouth dock yard.

In order to lift the deck girders into position, large floating cranes or cable mounted gantries would be required. Due to the high rates for renting floating cranes for the duration of this work, it is likely that lifting gantries would be specifically designed and fabricated for this work. The gantry itself would be lifted into place by a floating crane.

Once the gantry was in place the deck girders would be connected to spreader beams to prevent overstressing during lifting. These spreader beams would be attached to lifting strands for the gantry. The deck girders would be erected in accordance with a pre-determined construction sequence. This would normally entail starting from the centre of the main span and working towards the main towers symmetrically. The presence of Beamer Rock would introduce additional constraints to the erection sequence which may require the use of higher capacity lifting equipment, as the crane would not be able to work at an efficient distance from the bridge. With the availability of heavier lifting equipment it should be possible to erect 36 metre long panels. It is an advantage to make the panels longer as this reduces the amount of onsite welding. Each girder would be welded to the adjacent panels behind the erection front with a lag between the operations to ensure that the welded joints are not overstressed during the erection of further units. Once the deck girder panels were in position the girders would be attached to the cable hangers.

An example of how deck panels are constructed on a suspension bridge is shown in Figure C11.



Figure C11 – Lifting Deck Panels at Tsing Ma Suspension Bridge

Finishes including Surfacing

Once the bridge deck was substantially complete the deck surfacing could be laid and the deck furniture and communications equipment installed. In order to minimise the overall construction time, it should be possible to commence the installation of the deck finishes such as bridge deck furniture and some items of communication equipment before all the deck units have been erected.

It is likely that materials for the deck surfacing would be managed from the southern end of the bridge. The approach viaducts would need to be complete before the surfacing commences in order to provide access for the laying machines. Due to the experience gained from the existing FRB it is proposed to provide 70 millimetres thick surfacing. The existing bridge has a thickness of 38 millimetres. During resurfacing it has proved to be extremely difficult to remove the surfacing without damaging the waterproofing or the steel deck panels. With the increased surfacing thickness, it would be possible to remove the upper worn part of the surfacing but retaining the lower sound material. The new upper layer would then key into the lower existing layer, avoiding damage to the waterproofing and steel deck. The additional thickness has an additional benefit in its capacity to spread point loads over a larger area hence reducing the live load stresses in the steel deck plates.

C4 CABLE STAY BRIDGE OPTION

C4.1 General Description and Design Issues

Further work has also been carried out to review the cable stayed bridge option for Corridor D in Report 3. This work has reviewed the optimum span for the bridge and reviewed the basic design assumptions. Preliminary design has been carried out to determine preliminary quantities of steel, concrete and other civil engineering materials in order to verify the cost estimates of the bridge.

In addition, a detailed costing exercise has been carried out for the bridge superstructure, including the main deck, cables, and hangers. The bridge superstructure represents approximately two-thirds of the cost of the bridge. It was considered that this was a reasonably significant proportion to study in more detail. The cost estimate was reviewed in a similar manner to the suspension bridge.

The cable stay bridge option would consist of two main spans of 650 metres with equal back spans of 350 metres. The back spans would also be supported by additional piers introduced to provide stiffness at the ends of the cable supported spans. The central pylon would be founded on Beamer Rock. The alignment would be similar to the suspension bridge, running from the northern end of a toll plaza (if required) between Linn Mill and South Queensferry, over Beamer Rock to Cult Ness headland between St Margaret’s Hope House and the Queensferry Lodge Hotel. In addition, the southern approach viaducts would be approximately 635 metres with nine spans. The northern approach viaduct would be approximately 115 metres with two spans of 60 and 55 metres. The vertical alignment of the bridge provides a minimum vertical clearance of 45 metres above mean high water spring tide level with its crest over the pylon at Beamer Rock.

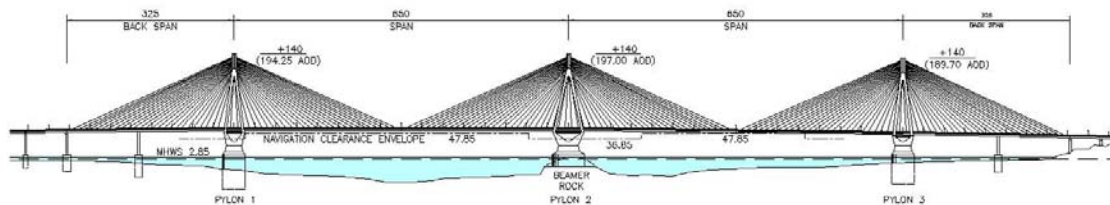


Figure C12 - Cable Stay Option

The pylons would be approximately 190 metres high above the water level and constructed in reinforced concrete. The pylons would support the cable stays via anchorages incorporated in the pylons. Internal access for personnel to carry out maintenance would be provided within the pylons. The position of the sockets would need to be carefully controlled to ensure geometric control of the bridge. A system employed on Rion-AntiRion bridge was the provision of prefabricated steel shutter with template openings for the anchorage sockets. A similar approach could be adopted for the bridge at corridor D.

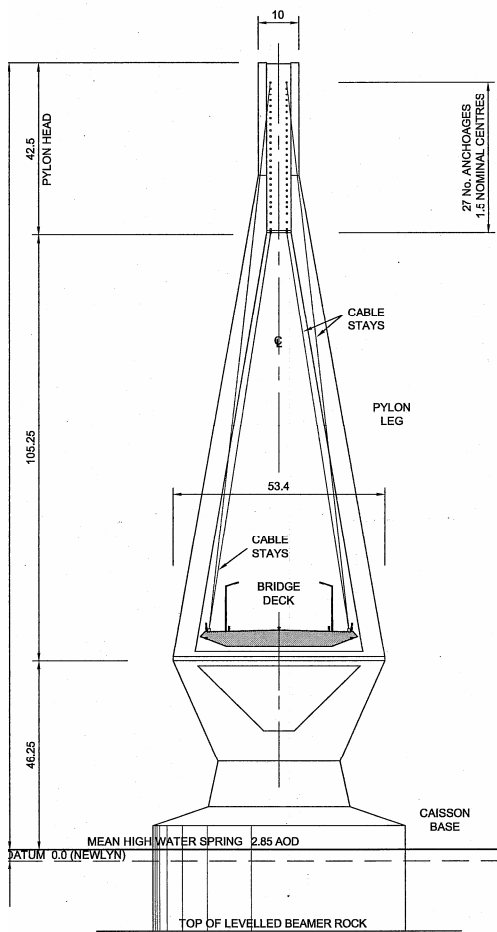


Figure C13- Cable Stay Tower

The central pylon founded on Beamer Rock would be an inverted “Y”-shaped above deck level with four legs to provide stability. Below deck level the pylon legs would slope in to reduce the footprint of the foundation on Beamer Rock. The section beneath the deck would be partially infilled with reinforced concrete, the exact profile of which would be determined during detailed design. The pylon could be supported on one large caisson or four caissons, each approximately 30 metres in diameter with the caissons connected by interconnecting reinforced concrete walls. The caissons would be benched into Beamer Rock.

Studies have been carried out to determine the most appropriate form of construction for the outer pylons. The Grangemouth navigation channel runs close to the southern pylon, and hence the longitudinal spread of the pylon could not be as great as the central pylon. It is possible to provide a narrower tower design with four legs, but the risk of ship impact potentially increases. As the spread of the legs increases the overall stiffness of the bridge reduces and also the natural frequency of the bridge reduces. In addition, due to temperature changes, the outer pylons would be subjected to bending. The wider the spread of the legs, the larger the moments attracted to the pylon legs. In further detailed studies of the bridge, the appropriate balance of the above issues should be considered to determine the optimum solution.

The outer pylons would be founded on reinforced concrete foundations taken down to the mudstones and sandstones below the soft alluvial and glacial deposits in the Firth of Forth. The southern tower foundation would be constructed to a depth approximately 40 metres below water level, which is approximately 30 metres into the soft material. The northern pylon foundation would be constructed to a depth approximately 40 metres below water level which is approximately 30 metres into the soft material. Similar to the suspension bridge, the foundations should be designed to resist ship impact and prevent the vessel overrunning the top of the foundation and impacting the tower leg.

C4.2 Global Structural Behaviour

Preliminary analysis of the double span cable stay bridge has been carried out using a 3D computer program. The analysis and design of a cable stayed bridge is extremely complex, and only a preliminary analysis has been carried out to assess principal member sizes of the deck girder and cable stays. In addition, natural frequencies, deflections and rotations have been assessed. Preliminary analysis has been carried out to determine approximate prestress forces in the cable stays such that under the most onerous traffic loading the cables do not go slack. In the detailed design this analysis would be extended to adjust the distribution of forces in the deck to optimise the design.

The pylons would be approximately 190 metres above water level. This height has been determined by considering the angle of the shallowest cable stay which supports the deck closest to the middle of the span. The cables would be arranged such that the span of the deck between the cable stays is limited to approximately 12 metres.

As per the suspension bridge option, movement joints and bearings have been kept to a minimum in order to reduce the maintenance liability, and to reduce disruption to the bridge users. The bridge would therefore be continuous through the pylons, with movement joints provided at the ends of the back spans. As discussed for suspension bridges, this would introduce large forces in the deck girder, which would need to be designed to cater for these large forces.

The central pylon would consist of four legs shaped as an inverted “Y”. The pylon would support inclined cables which in turn support the main deck at approximately 12 metres centres measured along the length of the bridge. It is likely following detailed design that the cable stay spacings along the length of the deck would vary, particularly near the pylons and at the centre of the spans. The shape of the pylons would provide more stiffness, particularly in torsion.

C4.3 Deck Girder and Bridge Articulation

Two forms of deck construction have been considered:

- Ladder deck with a composite concrete slab; and
- Box girder similar to the suspension bridge.

One possible method, known as a 'ladder deck', would comprise welded longitudinal steel plate girders with welded steel transverse girders at approximately four metre centres. The girders would act compositely with a reinforced concrete slab. This method has typically been used for cable stay bridges with spans up to approximately 600 metres, and is relatively easy to fabricate and erect as it does not involve welding of trough stiffeners. This method has been used for the Second Severn Crossing and the Rion AntiRion Bridge in Greece. From inspection of the drawings of the Rion Bridge it can be seen that the steel plates are very thick, up to 90 millimetres, and are a higher grade of steel than typically used in bridge construction in the UK. It can be inferred, therefore, that, for the Rion bridge with spans of 560 metres, the design is beginning to push the limits of technology. In addition, the Rion Bridge is narrower than that proposed at the Forth, and this would increase the span of the transverse beams. Preliminary analysis of this construction type supports these deductions. The concrete slab would add significant weight to the overall deck construction. This weight would increase the cable stay sizes and pylon sizes required.

For the length of span considered, aerodynamics would become particularly important. The ladder deck is not as stable, aerodynamically, as the box girder. It can, however, be enclosed in a glass reinforced plastic (GRP) enclosure to create the same shape, and hence provide the same aerodynamic properties as the box girder. The ladder deck is more complex than a box girder to paint as it has several faces of steel plate. The introduction of the GRP enclosure would provide a safe internal access to carry out the painting and other maintenance work.

The box girder provides a relatively light, stiff, deck which would be relatively easy to paint. It would also be aerodynamically stable. It has been decided, therefore, at this stage, that a box girder would be the most practical solution for the cable stay bridge deck.

The construction of the deck would be similar to the suspension bridge. From the preliminary analysis, the D2M bridge deck would be 3.5 metres deep. The top deck plate would typically consist of 12 to 13 millimetre thick plate with 6 to 7 millimetre thick troughs. These plates would be thickened locally at the pylons. The bottom deck plate would be typically constructed using 10 millimetre thick plate strengthened with six millimetre thick troughs. The sloping web plates would be constructed in a similar manner, but the upper sloping plates would be thicker in order to transfer the high shear forces and loads at the cable stay connections.

Buffers would be provided at each end of the bridge deck, similar to the suspension bridge, in order to limit longitudinal movements arising from wind and traffic loads.

C4.4 Cable Stays

For the D2M option, the cable stays would be approximately 35.5 metres apart, and be inclined when viewed along the length of the bridge as they would be attached to the upper portion of the inverted “Y” pylon.

In the design of cable stays the most important considerations are as follows:

1. The cables must be easy to maintain such that all cables can be freely inspected and, if necessary, individually exchanged. In early cable stay bridges, the number of cables was much smaller, and hence the cables were of a relatively large diameter. The current trend is to provide a larger number of relatively smaller diameter cables. These are easier to erect and makes replacement of individual cables easier.
2. Access must be easy to allow inspection of the full length of the cable stays and the anchorage sockets at the main pylons and deck. The main pylons would be designed with provision for personnel access up the height of the pylon in order to gain access behind the anchorage sockets. At the deck level, the cable anchorages can be inspected using the maintenance access ways. Access inside the box would be provided to inspect the sockets and bulkheads below deck level.
3. The connection to the deck socket in the deck must be sealed to prevent corrosion.
4. Adequate dampers must be provided in order to eliminate or reduce potential wind-induced oscillations.

In addition to the above, the structural design of the cable must take into consideration the following:

1. The cable forces under permanent loads must be determined such that under the most onerous load case the cables do not go slack.

2. Cables are susceptible to fatigue loading and they need to be designed taking into consideration fatigue.
3. The bridge needs to be designed for the 'cable out' situation in which the cable is removed either by accident or during replacement.

Various types of cable stay have been used in the past. The following have been considered for use on the bridge option:

- Cables from Parallel Wires or Strands.

These cables consist of several parallel wires, ¼ inch (6.3 millimetres) in diameter or parallel strands, 0.6 inch (15.7 millimetres) in diameter. Corrosion protection of cables is particularly important; some of the earlier cable stay bridges have suffered from corrosion leading to early replacement of cable stays. This protection can be achieved by enclosing the cables with a polyethylene (PE) pipe and injecting cement grout to fill the gaps. The PE pipe would have sufficient resistance against the weather including resistance against ultra-violet light. Finally the pipes would be wrapped with tape for extra weather resistance.

- Locked Coil Ropes

Locked coil ropes were used for the earliest cable stay bridges. For corrosion protection, all wires in such cables would be hot dipped galvanised. The galvanising also improves the fatigue strength of the rope, as it acts as a lubricant and avoids friction between the individual wires. The inner voids would be infilled with polyurethane and zinc, or a resin known as Metalcoat. The outer surface would then receive further coatings and finish coatings of similar material type. The locked coil system could then be enclosed within a PE pipe.

Locked coil ropes perform less well under fatigue, shipping, and handling and durability, when compared to parallel wire strands.

The final choice of cable stay would depend upon the Contractor's choice. For the purposes of this study, it has been assumed that parallel wire strand cables would be used.

For the D2M bridge option, following a preliminary analysis, the cable stays are estimated to vary from 43 strands for the cables closer to the pylons up to 73 strands for the cables closer to midspan.

C4.5 D2M plus LRT (Bus or Rail)

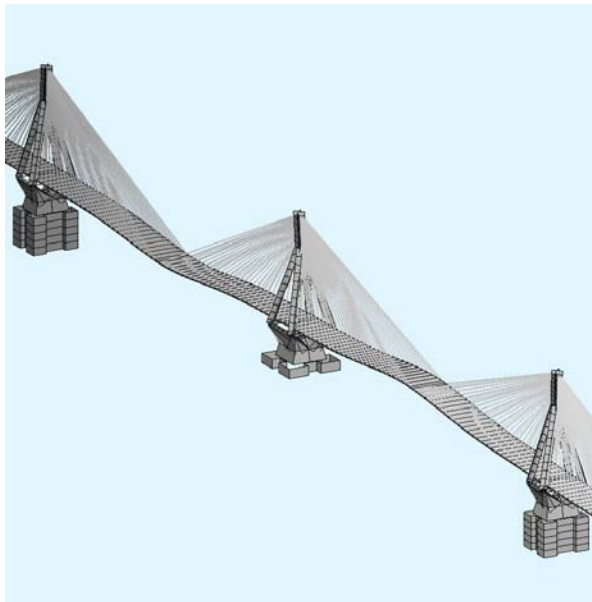
The deck cross section for the D2M plus LRT is described in section C2.2 above. In addition to the additional train loadings, the bridge would be subjected to increased loading associated with the trackform, signalling, and overhead power equipment.

C4.6 Deck Girder and Bridge Articulation

Similar to the suspension bridge, the consequences of placing rail loading on a cable stayed bridge are very significant. The deformation of a bridge incorporating railway loading is a critical issue, and hence the bridge design may not be governed by strength alone. Similar to the suspension bridge, there are strict restrictions on the deflections of the deck girder and the rotations especially at expansion joints. The deck would, therefore, be made continuous through the towers and movement joints provided at the ends of the back spans of the cable supported structure. The shorter spans of the adjacent approach spans would help to reduce the overall angular rotation of the deck and the rail at the movement joints. The rail joint and the underlying structure would need to be specially designed to cater for the large movements and rotations in a similar manner as the suspension bridge.

In order to analyse the effect of the bridge on the train, the deflections of the bridge and natural frequency of the bridge have been determined from preliminary analysis.

The preliminary analysis indicates that the first natural frequency of the bridge in bending is 0.3 Hertz. Preliminary studies indicate that this would be acceptable. Similarly, the first torsional natural frequency has been estimated as 1.3 Hertz and preliminary studies indicated that this would be acceptable.



First natural frequency in Bending



First natural frequency in Torsion

Figure C14 – Natural Frequencies of Cable Stay Option

The deflections obtained from the preliminary analysis are presented in Table C2.

Table C2 – Deflections and Rotations

Type of Movement	Load Case	Magnitude of Movement
Mid-span deflection	One train at mid-span	0.1 metres vertical deflection
Change in gradient at movement joint	Maximum road and Rail Loading	0.009 radians rotation
Change in gradient at movement joint	One train at mid-span	negligible

The box girder would be similar in construction to the D2M, and would be approximately 21 tonnes per metre.

C4.7 Construction Issues Relating to Cable Stay Bridges

Construction Compounds and Assembly Yards

Many of the requirements for construction yards and assembly yards are similar for suspension and cable stayed bridges. For the cable stayed bridge, it would also be intended to use a site such as Methil to act as the main storage and assembly yard. The deck fabrication and assembly issues would be similar, and therefore a similar area would be required.

The construction of a cable stayed bridge deck typically uses a cantilever approach. The broad sequence of activities is as follows:

- construct foundations;
- construct towers and abutments;
- erect cantilever deck sections progressively with cable stays; and
- install finishes (road surfacing, bridge deck furniture, communications etc).

Foundations

A cable stayed bridge requires competent rock in order to provide a sound foundation for the main towers and abutments. The foundations at the abutments are less complex than those of a suspension bridge as there is no requirement for tension anchorages.

The foundation construction methodology for the outer pylons would be similar to that for the suspension bridge. Typically, this would involve the construction of a sheet pile cofferdam within which the reinforced concrete caisson would be constructed.

For the central pylon founded on Beamer Rock the procedure would be modified as follows. First the surface of Beamer Rock would be prepared by excavation. Precast caisson shells would then be grouted into position and extended in in-situ reinforced concrete. Four caissons approximately 30 metres in diameter would be required. These could be linked together by reinforced concrete walls to ensure that ship impact forces would be spread between the caissons. This arrangement would need to have a suitably streamlined shape to minimise the possibility of scour. Alternatively, one large caisson could be used.

Towers

The towers would be reinforced concrete and the construction method would employ slip forming or jump forming for the sloping tower legs similar to the suspension bridge. Temporary bracing would be required as the tower height increases, and before the top section stabilises the individual legs.

The construction of the pylon would become particularly complex at the junction of the four legs and the section above, due to the increased density of reinforcement and the setting out of the cable stay anchorages. Because of the geometry of the deck and the pylon, the cable stays would not be in a vertical plane, and each anchorage would have a unique orientation. There would inevitably be difficulties in casting the anchorages in situ to satisfy the close tolerances necessary for the anchorages. This could be overcome by the use of prefabricated steel templates with sections of the anchorages incorporated. This unit could then be incorporated in the construction of the pylon.

An example of tower construction and pre-fabrication is shown in Figures C15 and C16 below.



Figure C15 – Rion Bridge Towers under Construction



Figure C16 – Rion Bridge Prefabricated Tower Section

C4.7.3 Deck and Cables

The individual panels for the deck units would be fabricated and assembled in a similar manner to that described above for the suspension bridge option.

The completed deck units would be transferred to a barge and taken out to the deck erection front. The deck would then be lifted into place supported by the inclined cable stays. The deck would be cantilevered out from both sides of each pylon in order to minimise out of balance loads on the towers and their foundations.

The deck girder at the pylon itself would need to be erected using an external crane, such as the jack up barge cranes used for the installation of the caisson shells. Once the deck girder unit was in place its supporting cable stays could be hauled up to their upper anchorage position on the pylon. Once connected the cable stays could then be stressed up. At this stage, cranes would then be lifted up to the deck leading edge. The next deck panels would then be transported to the erection front and hoisted up to position by the cranes mounted on the leading edge of the erected panel. The connection between the deck panels must then be made by onsite welding the panels together. It would be important to complete sufficient length of weld before stressing of the next cable stays could commence. This cycle would be repeated as a balanced cantilever until the two cantilevers met at midspan with a final drop in section.

The main risks associated with deck erection relate to the weather and the slenderness of the cantilever decks. As the cantilevers reached their maximum length just before the sections meet, the deck would be at its most vulnerable condition for resistance to wind loads.

An example of the deck construction of a cable-stayed bridge is shown in Figure C17 below.



Figure C17 – Rion Bridge – Deck and Cable Stays Under Construction

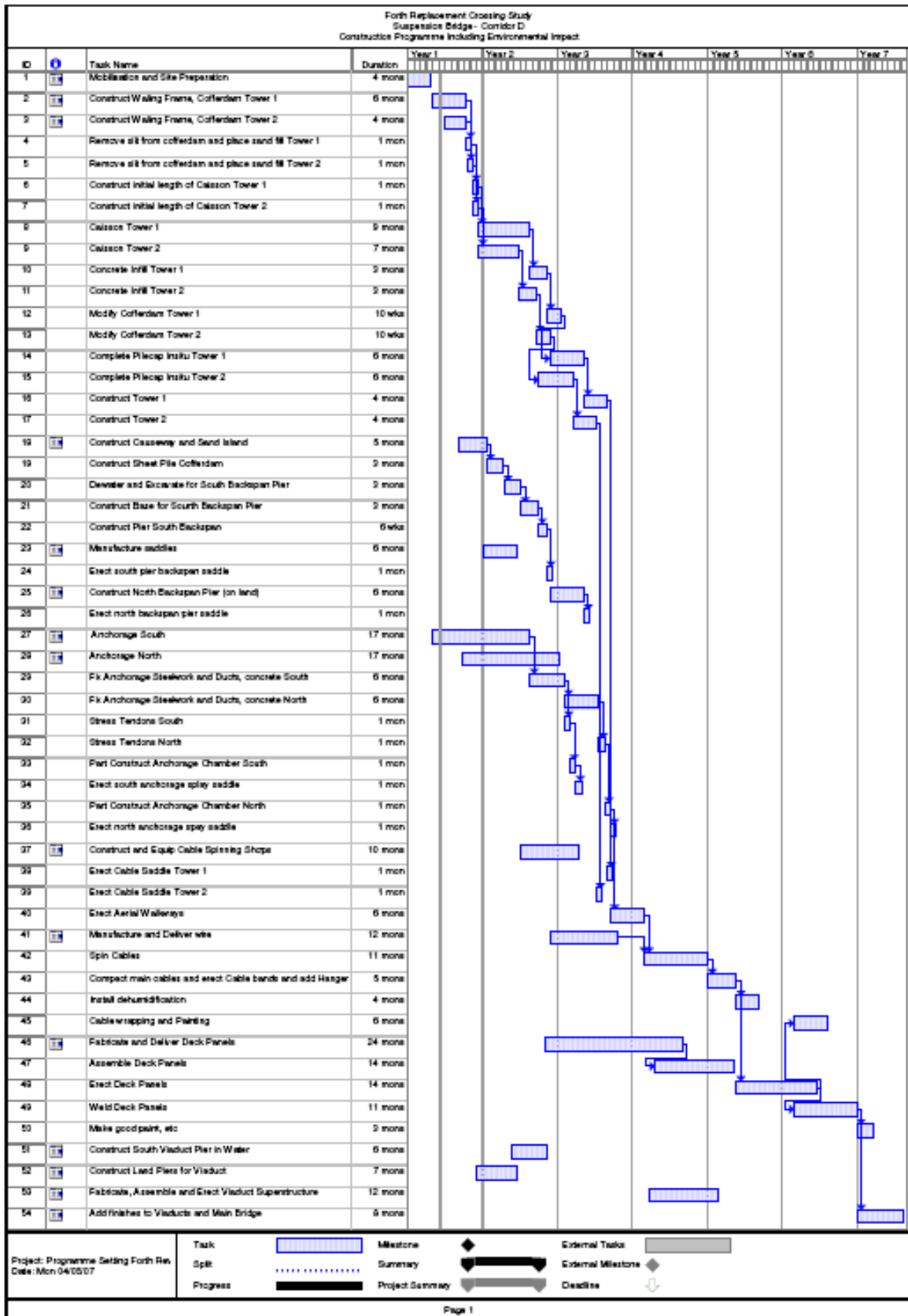
C5 CONSTRUCTION PROGRAMME

Outline construction programmes for the suspension and cable stayed bridges have been developed. Initially, it was assumed that environmental impacts would not lead to any changes to the programme. On this basis it was estimated that the suspension bridge programme would be approximately six years, and the cable stayed bridge programme approximately five and a half years.

The construction of a suspension bridge follows a generally linear programme, with little opportunity for concurrent working. The exception to this is that more than one tower or foundation can be constructed at the same time if the resources, particularly specialist plant, are available. For the construction of a cable stayed bridge, after the main pylons have been constructed there is scope to erect the cable stays and deck from the three main pylons concurrently. This offers a reduction in the programme as noted above.

These programmes were reviewed against the specific environmental constraints arising from summer breeding birds located on Long Craig Island and wintering birds located on the Special Protection Areas (SPAs) at Port Edgar and the north intertidal zone. It was assumed that work up to the bridge deck level in the vicinity of Long Craig Island would be interrupted for two summer months each applicable year. In addition, for the SPAs, two scenarios were investigated: one in which construction work up to the underside of the bridge deck in the vicinity of the SPA would be interrupted for two winter months every second year, and one in which the construction work would be interrupted for five months every third year. As a consequence the construction programme for the suspension bridge would be increased by a maximum of eight months and the cable stayed bridge a maximum of ten months.

The construction programme for the suspension bridge option incorporating allowance for environmental mitigation is included below:



C6 COST ESTIMATES

C6.1 Introduction and Background

Cost estimates were originally derived at the Setting Forth stage using, as a basis, rates obtained from the Second Severn Crossing.

At the Setting Forth Stage an outline design was developed to assess the material quantities for the principal components of the bridge. These quantities were then multiplied by rates derived from historical data and experience on similar projects. Additional costs for specialist plant, access, general overheads, design, profit etc were added to these cost estimates. In some cases these costs were considered to be a 'best estimate' of the lowest likely tender price if the contract for the bridge was to be let as a single design and construct bid (without the approach road element of the scheme).

Having established the 'best estimate' values the effect of risk on each of the components of the overall cost was critically considered. Risk was split into four elements for each component of the construction works:

- 'Design Risk' – principally due to the size and complexity of the structure, or members increasing as the design is developed from tender design to working drawings (expressed as a percentage of the quantity);
- 'Risk on rates' – to reflect uncertainty in the rates used to derive the 'best estimate' costs (expressed as a percentage of the rate);
- 'Construction risks' – to cover the general construction risks, including problems encountered during construction, increasing quantities in items such as foundations, but excluding the cost of consequential delays (expressed as a percentage of the quantity); and
- 'Delay risk' – identified as the effect on the overall programme or critical path arising from construction delays. The cost of the delay is derived by identifying the increased overheads over the duration of the delay (both operation specific overheads and general overheads).

The risk percentages identified were added and then applied to each 'best estimate' component to produce the 'high risk' cost estimates.

The high risk figure represents the cost at which a wide range of risks all achieve their maximum value at the same time. Statistically there is a very low probability of this occurring, and that probability was assessed at five per cent. At the other end of the cost estimate range, there is a low probability that the best estimate could be improved upon during competitive tender and that probability has been assessed at 10 per cent. Between these bounds the form of the probability distribution cannot easily be assessed.

Costs were also derived for several cable stay configurations.

C6.2 Further Cost Estimate April 2007

Detailed cost estimates of the bridge superstructure, for the suspension and cable stayed bridges for corridor D, was carried out between February and April 2007. The bridge superstructure includes the deck girder, suspension or cable stays, hangers, and saddles. The superstructure accounts for approximately two-thirds of the overall cost, and it was considered that this was a significant portion of the bridge on which to develop more detailed costing information. The costing exercise would also be useful to test the costs developed during the Setting Forth project.

The following schemes were costed:

- Suspension bridge D2M;
- Suspension bridge D2M including LRT;
- Cable stayed bridge D2M; and
- Cable stayed bridge D2M including LRT.

To establish the cost estimates, preliminary design was first carried out for the principal structural members. A detailed assessment of person-hours and material rates was then carried out, and an overall rate for the principal structural items making up the superstructure was then established.

Once the rates had been established, they were then inserted into a modified copy of the spreadsheet used in the Setting Forth project. This spreadsheet contains risk factors, applied to each principal structural item and are applied to the risk areas noted above, ie

- Design risk as a percentage;
- Risk on rates as a percentage;
- Construction risk as a percentage; and
- Delay risk in months delay on site.

The risks varied according to their severity. The risks were modified from the original analysis such that the increase was doubled but at the same time allowing the risk to decrease by 50% of the increase. A risk analysis was then carried out using the 'Monte Carlo' program, which analyses the probability of each risk reaching a maximum at the same time. Wherever it was considered that risks were linked this was incorporated in the model.

Referring to the Transport Analysis Guidance, TAG unit 3.5.9, states the following in relation to a Quantified Risk Assessment (QRA):

“3.3.17 A QRA allows a probability distribution around the costs of the scheme to be derived and enables the expected risk-adjusted cost estimate to be obtained. This expected outcome, also known as the 'mean' or 'unbiased' outcome is the weighted average of all potential outcomes and associated probabilities. This is the risk-adjusted cost of the scheme, and it is to this that the optimism bias will be applied. Operating costs and capital costs should all be based on expected values of the cost of the scheme.”

The expected risk adjusted (mean) value has been derived from the analysis described above and are presented in section 5.10 of the main report 4.

C6.3 Environmental Mitigation Cost Estimates

As outlined in section C5 above, an allowance was made for environmental mitigation. This resulted in a predicted eight month delay for the suspension bridge and a 10 month delay for the cable stayed bridge. The effects of these delays were built into the cost estimate.