e-mosty

ISSUE 01 / MARCH 2017

Queensferry Crossing. Forth Road and Railway Bridges.

***Front Cover:* Queensferry Crossing. North Tower.** Courtesy of Transport Scotland

***Back Cover:* Forth Road Bridge with Queensferry Crossing.** Courtesy of Transport Scotland

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Dear Readers

Three amazing bridges built in three centuries – 19th Century Cantilever Railway Bridge, 20th Century Suspension Road Bridge and 21st Century Cable-Stayed Road Bridge: three bridges across the Firth of Forth in Scotland.

They show the endless human effort to overcome obstacles, invent, develop and implement new ideas and solutions. They are proof of highly professional engineering skills, knowledge, expertise and experience of all the people and companies involved.

This issue brings information of the three bridges´ design, construction, maintenance and operation accompanied with drawings, photo and video galleries. Our effort to cover everything relevant and important ended up in this issue being exceptionally longer than our other e-magazines.

I thank you all for your contribution, cooperation and assistance.

Our magazine e-mosty aims to be technical, with focus on design, construction, and technical details in a descriptive way. At the same time we are happy to provide medial partnership to various international conferences and publications.

I do my best to keep it open access, with the content educational rather than commercial and avalanched by advertisements. I believe that financing of the magazine may be provided by partners / supporters.

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Thank you.

Magdaléna Sobotková

Chief Editor

e-mosty

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I would like to offer you partnership with our magazine „e-mosty“ (e-bridges).

Information about the magazine can be found at **www.e-mosty.cz** and previous issues can be viewed in the archive section of the website.

Since May 2015 when the magazine was established we have achieved especially the following:

- Established the Editorial Board
- Released 7 issues (typically 4 per year)
- Achieved an international profile with increasing numbers of readers (over 1000 already) with positive reviews and feedback.

„e-mosty“ magazine is unique and covers a big part of the construction market.

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We are looking forward to possible future cooperation with you and your company.

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FIRTH OF FORTH BRIDGES

THREE BRIDGES – THREE CENTURIES

Magdaléna Sobotková

Photo: Courtesy of Transport Scotland
Source: http://www.forth-bridges.co.uk

1. GENERAL INTRODUCTION

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

2. QUEENSFERRY CROSSING - PLANNING AND PROCUREMENT

The Forth Road Bridge has successfully carried road traffic across the Forth estuary since 1964. The deteriorating condition of the bridge, particularly in relation to the main suspension cables, and the great difficulty in rehabilitation without massive disruption to traffic, has resulted in the need for a replacement crossing to secure the future of cross-Forth travel.

In 2006 Transport Scotland commissioned the Forth Replacement Crossing Study to determine the optimum solution.

The sensitive location of the crossing necessitated environmental review at an early stage to consider a number of local, national and internationally protected sites. A total of 65 initial options were studied and appraised which led to a selection of five potential corridors, labelled A to E.

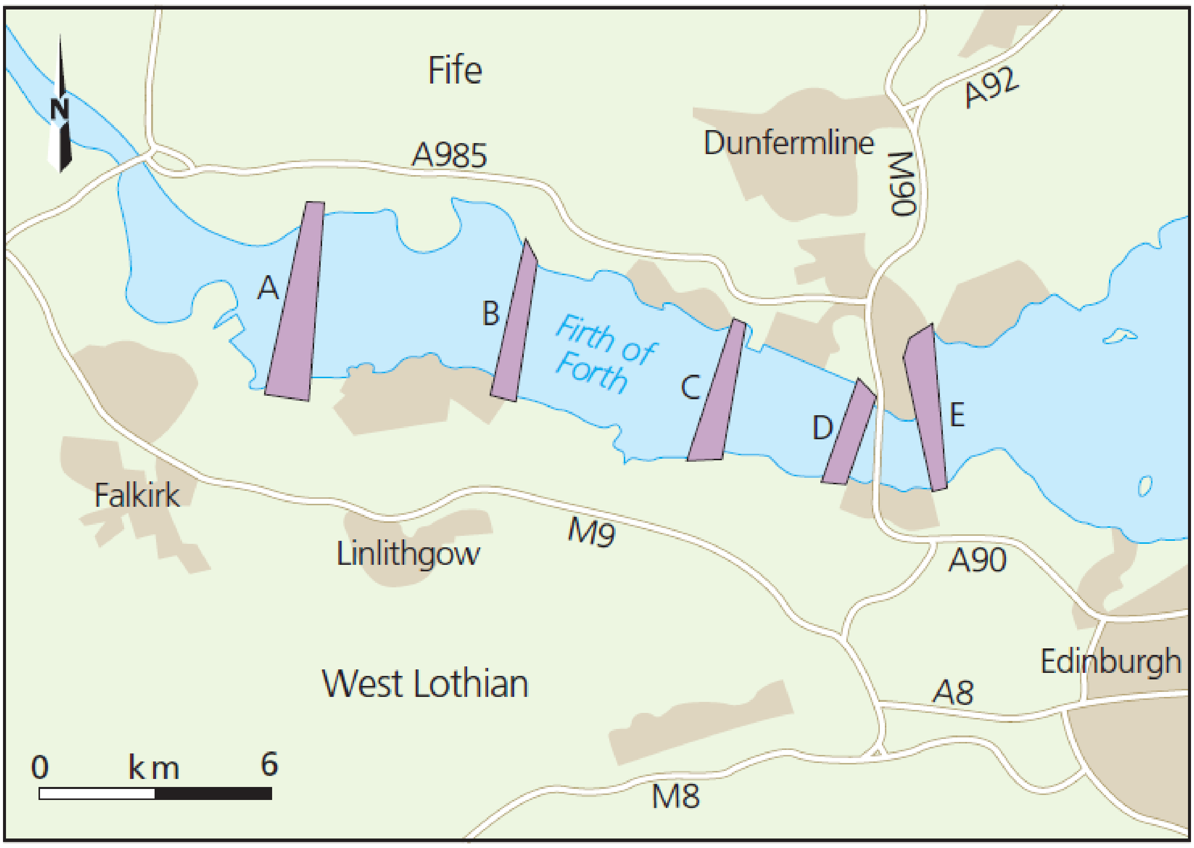


Figure 1: The five potential crossing corridors

Comparisons were made between options for tunnel and bridge. After careful and detailed analyses and assessment, a cable-stayed bridge in corridor D, slightly west of the Forth Road Bridge, was recommended as the preferred scheme as it was significantly cheaper than tunnel options. It could be delivered quicker, had fewer risks associated with construction and demonstrated the best value for money. Of the possible two types of bridges (cable-stayed and suspension), the suspension bridge would require complex foundations on the landfalls – the cable-stayed did not.

In January 2008, Transport Scotland, by way of a competitive tendering exercise, procured the services of the Jacobs Arup JV to assist with the management and delivery of the Forth Replacement Crossing project.

The scope of the commission included the development and assessment of the project proposals, concept and specimen design of the bridge, preparation of contract documents, assistance in the procurement and authorisation of the project and subsequent monitoring of construction.

The project is being managed by an integrated team of Transport Scotland and Jacobs Arup staff.

At this stage, the following were considered:

- Future use of the Forth Road Bridge: Following a second cable investigation, an improved prognosis for the rate of cable deterioration was given. Together with removal of general traffic from the bridge it provided some hope that a functional use for the bridge might be possible.
- The functional requirements of the Forth replacement crossing could therefore be significantly reduced.
- Making the best use of existing infrastructure where possible.
- Incorporation of an intelligent transport system: it is to be the first such application in Scotland.

These considerations led to a significant reduction in the extent of the road network connections. All of the previously described measures form the managed crossing strategy, with reuse of the existing bridge and a reduced extent of new road construction being key elements in revising the scope of the project.

These changes reduced the cost estimate from circa £4 billion to around £2 billion. The strategy was announced by the Scottish government in December 2009 and formed the basis for progressing the project.

The project was divided into three contracts. In view of the large size of the project and the likelihood of attracting international consortia to bid, a contract form based on the FIDIC standard conditions for turnkey project (FIDIC Silver Book, 1999) was adopted.

The competition for the principal contract was undertaken in parallel with the progression of a parliamentary bill. In early 2011 the preferred bidder was announced to be Forth Crossing Bridge Constructors.

In total, the overall Forth Replacement Crossing scheme is 13.7 miles (22km) long, including major motorway upgrades to the north and south of the bridge and also the first ever use in Scotland of variable mandatory speed limits to smooth traffic congestion via an Intelligent Transport System. This also controls dedicated bus lanes within the motorway hard shoulders – another first in Scotland.

The bridge is planned to open for traffic in 2017.

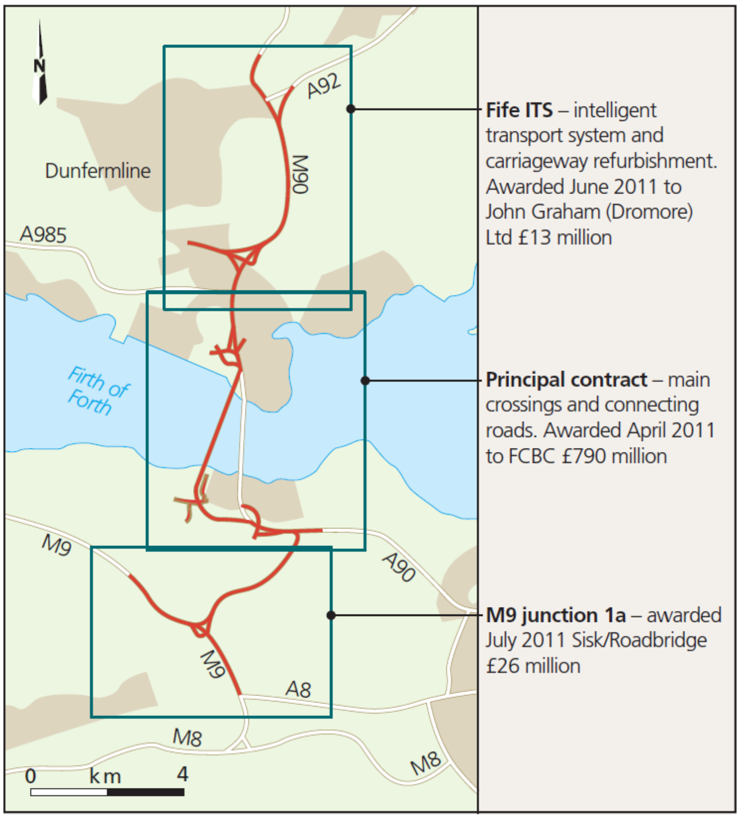


Figure 2: Forth Replacement Crossing Contracts

Stage	Date
Forth replacement crossing study	August 2006–June 2007
Public exhibitions	August 2007
Scottish government confirms bridge crossing	December 2007
Consultants Jacobs Arup appointed	January 2008
Environmental surveys begin	February 2008
Land searches	February 2008
Traffic surveys begin	March 2008
Marine ground investigations	May–August 2008
Topographical surveys	March–August 2008
Land-based ground investigations	March–August 2008
Scottish government confirms scheme details	December 2008
Public exhibitions	January 2009
Environmental impact assessment concluded	Summer 2009
Parliamentary bill introduced and statutory consultation	November 2009
Competitive dialogue process begins	December 2009
Royal assent – Forth Crossing Act 2011 (2011)	January 2011
Construction contract (principal contract) signed	April 2011
Construction begins (all contracts)	Summer 2011

Figure 3: Key project stages

QUEENSFERRY CROSSING

Magdaléna Sobotková

I. Key Facts

Rendering: DISSING+WEITLING architecture

Commencement of works: 2011

Opening of the bridge to traffic: 2017

Type of the bridge: Cable-stayed Road Bridge
(D2M with hard shoulder)

Main spans: 2 x 650m

Total length between abutments: 2 638m

Location: South Queensferry, Scotland, United Kingdom

Client: Transport Scotland

Client Adviser: Jacobs / Arup Joint Venture

Engineer: Jacobs/ Arup Joint Venture

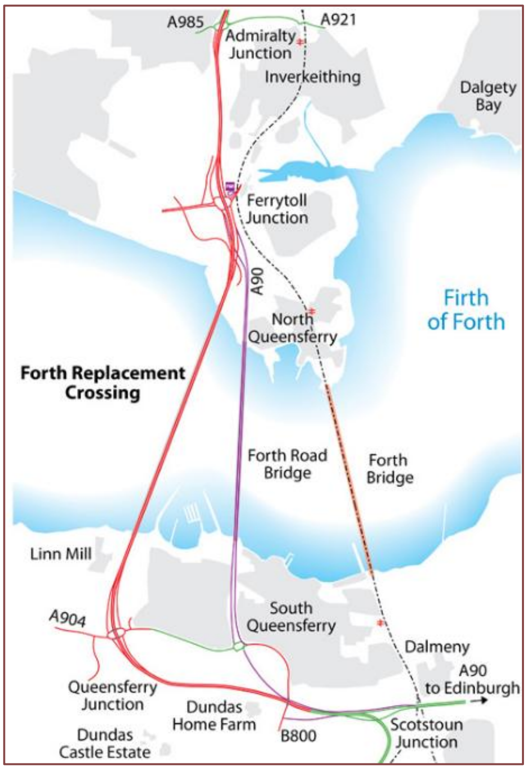
Architect: DISSING+WEITLING architecture

Design: The Specimen Design for the Queensferry Crossing was completed by the Jacobs Arup Joint Venture on behalf of Transport Scotland. The Final Design, on behalf of Forth Crossing Bridge Constructors (FCBC), was undertaken by the Design Joint Venture comprising Grontmij, Ramboll and Leonhardt Andra and Partners

Contractor: Forth Crossing Bridge Constructors (FCBC) – a consortium of Hochtief from Essen, Germany, Dragados from Cadiz, Spain, American Bridge from Pittsburgh USA and Morrison Construction, originally from Inverness, Scotland

Location of the bridge on the map:

Source: Google Maps



Principal Contract Map. Source: Transport Scotland

INTERESTING FACTS:

- The tower height is 210m above Ordnance Datum (683ft).
- Over 100,000 m^3 of concrete has been poured.
- 35,000 tonnes of steel has been used for the bridge deck (final design).
- The combined steel required for North and South viaducts weighs 7,000 tonnes.
- 23,000 miles of cabling has been used (this would very nearly stretch around the entire Earth).
- It is estimated the construction has involved approximately 10 million man hours.
- The centre tower deck has been recognised by Guinness World Records as the largest freestanding balanced cantilever in the world. The record-breaking cantilevered structure comprises 36 steel and concrete composite deck sections, which are some 40m wide, 16m long and 5m deep, and weigh an average of 750 tonnes.
- In 2013 the world's largest continuous underwater concrete pour was achieved as foundation work for the huge 210 metre high Queensferry Crossing towers progressed.

  The huge 15 day, 24 hour non-stop operation successfully poured 16,869 m^3 of concrete to the foundations of the south tower.

In September 2013, the world´s largest continuous underwater concrete pour was achieved as foundation work progressed.

Source: Transport Scotland

In October 2016 the central tower deck became the largest freestanding balanced cantilever in the world.

Source: www.forth-bridges.co.uk

QUEENSFERRY CROSSING
II. Specimen Design

Rendering: DISSING+WEITLING architecture

1. BACKGROUND

The need for the new crossing arose following a detailed inspection of the Forth Road Bridge in 2004. During an internal inspection of the main cables serious corrosion was discovered which, if left unchecked, could lead to the bridge being closed to heavy goods vehicles as early as 2014 and to all traffic by 2019.

The Forth Estuary Transport Authority (FETA) (*NB: the organisation that operated the road bridge then – since 2015 it has been operated by Amey*) initiated a programme to dehumidify the main cables in an attempt to arrest deterioration. The first of three dehumidification plants came into operation in February 2008 and cable inspections at that time showed an improved prognosis for the bridge.

The inspection indicated that with the assessed rate of deterioration restrictions to heavy goods vehicles were more likely to be considered at a later date, between 2017 and 2021. A refurbished Forth Road Bridge would be able to continue to carry pedestrian and cycle traffic as well as serve as a transport corridor for buses.

2. INTRODUCTION

The Queensferry Crossing is a landmark 21st century structure designed to complement the adjacent 20th century Forth Road Bridge and the historic 19th century Forth Railway Bridge. As with its illustrious predecessors the new crossing will span the Forth Estuary maintaining vital connections between Edinburgh and Fife. On completion, this project will create the longest three tower cable-stayed bridge in the world and also by far the largest to feature cables which cross at mid-span. This stiffening method provides extra strength and stiffness, stabilises the central tower and at the same time allows the towers and the deck to be more slender and elegant.

The Queensferry Crossing comprises three approximately 200m high main towers which support two 650m main spans with associated approach viaducts with a total crossing length of 2 638m.

It is located slightly to the west of the existing bridges, making use of Beamer Rock, a natural dolerite outcrop in the middle of the Forth. It allows the wide estuary with two navigation channels to be crossed by a pair of 650m cable-stayed spans, with an approach viaduct to the south.

The client, Transport Scotland, appointed the Jacobs Arup Joint-Venture (JAJV) in 2008 to assist with procurement and scheme preparation of a Design and Build contract. Although the final design would be undertaken by the successful tenderer, Forth Crossing Bridge Constructors (FCBC), a highly developed specimen design was undertaken pre-tender by the JAJV. This design was more detailed than is usual for design and built contracts and was commissioned by Transport Scotland because of the importance of aesthetics in the context of the two adjacent iconic bridges, and also to give tenderers confidence that what they were being asked to bid for could actually be built.

3. SPECIMEN DESIGN

3.1 Basis of design

The Queensferry Crossing is the first major bridge in Scotland to be designed to Eurocodes. It was necessary to provide rules and criteria appropriate to the bridge as well as clarify how some of the Eurocode rules should be interpreted. Aspects such as the site specific wind climate and the rules for ship impact criteria were defined as well.

3.2 Analyses

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

3.3 General Arrangement

The bridge is divided into a cable-stayed bridge and a southern approach viaduct; the structure is continuous from abutment to abutment with no intermediate movement joints. Longitudinal fixity is provided by a monolithic connection at the central tower located on Beamer Rock, with transverse support to the deck provided at all towers and piers.

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

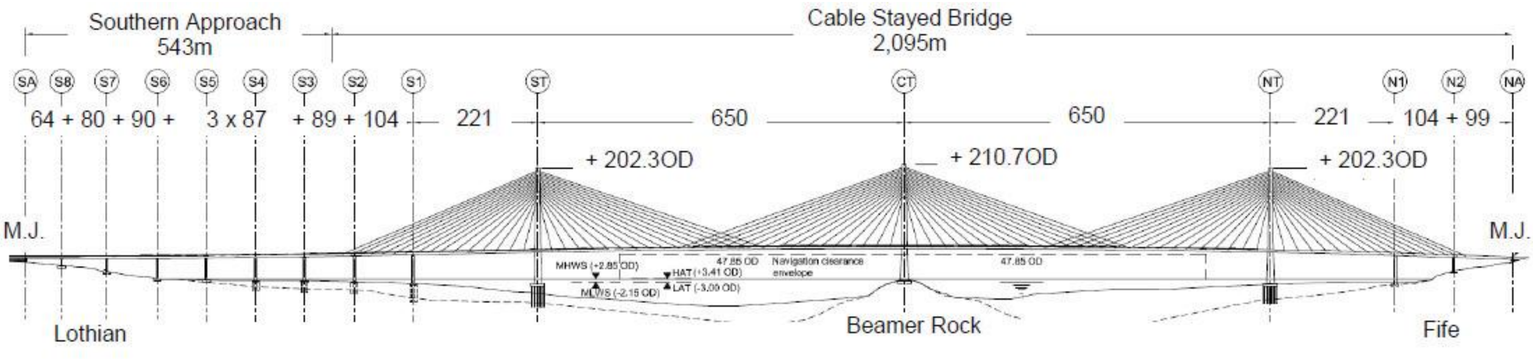


Figure 1: General Arrangement

3.4 Approach viaducts

For the approach viaduct, a key design requirement is visual continuity with the cable-stayed bridge and long spans to minimise the environmental impact of foundations. One span in particular must cross the Port Edgar barracks and adjacent road in a single span of 90m; the typical spans are dimensioned to achieve a visual rhythm with this key span. The aesthetic requirements are achieved by a pair of constant-depth box girders, of steel concrete composite construction, which merge into the cable-stayed bridge. Support is provided by V-shaped piers to minimise the size of foundations.

3.5 Superstructure

During preparation of the Specimen Design there was no clear advantage to distinguish between all-steel orthotropic and steel-concrete composite construction for the cable stayed bridge deck box. Therefore the contract permitted either to be adopted. The heavier composite deck variant had stay cables spaced at typically 16m. Incremental launching was adopted for the composite option.

A double-cell box girder was considered but eventually eliminated from the options especially due the following reasons:

- It is slightly more expensive to build and harder to maintain than a single-cell box.
- Because of the reduced torsional stiffness the twin-cell box required that the stay cables be anchored along the edges.

In the end, a single-cell box girder was adopted, with a double planar stay-cable fan. It was meant to give the best emphasis to the crossing stay cables, which are a unique feature of this bridge.

Considering the centrally anchored stay cables, studies were carried out to investigate the torsional behaviour of the deck under a number of different traffic scenarios to establish appropriate design criteria for the twist of the deck and to confirm that the design meets requirements. Changes in the cross-fall were considered with respect to performance of the drainage system, safety of the road and user perception.

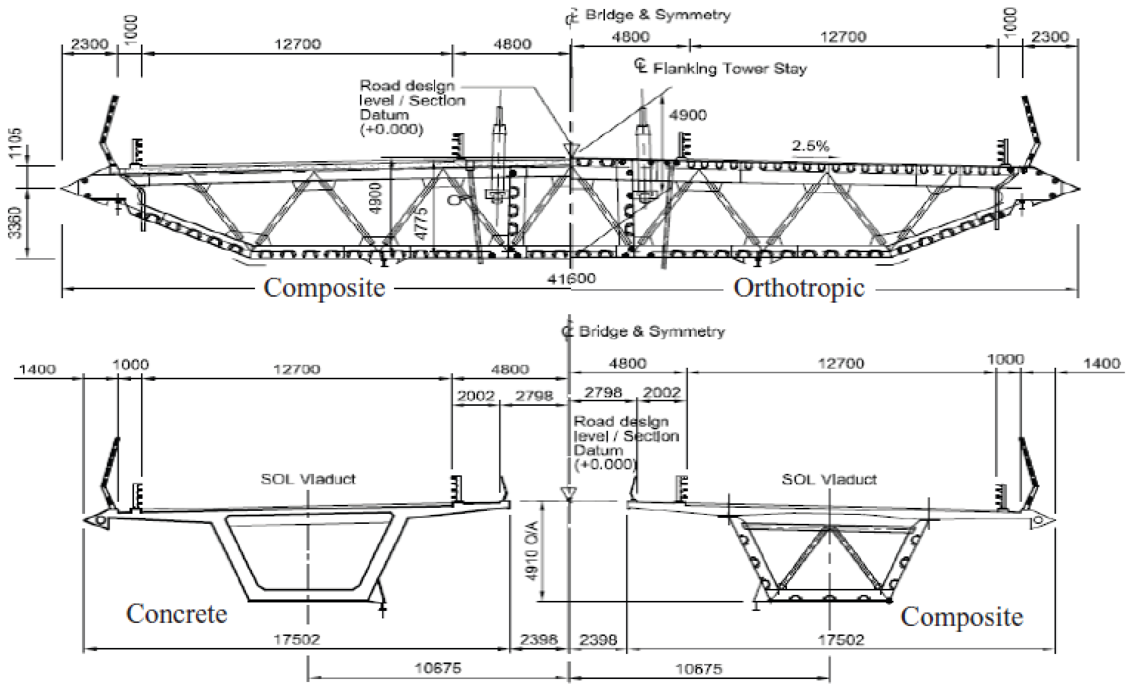


Figure 2: Deck sections, showing all variants

3.6 Foundations

The Firth of Forth is a fjord, formed by the Forth glacier in the last glacial period. The maximum water depth on the line of the bridge is 45m with rock being as low as about -85m OD (Ordnance Datum). The alignment of the bridge is dictated by the desire to position a tower on Beamer Rock. It is formed by a steep-sided dolerite outcrop that reaches an elevation of about +3m OD and forms a ridge trending north-west to south-east, which is almost perpendicular to the bridge alignment.

The area of rock exposed varies with the tide, reaching about 45m x 95m at low water springs. The rock is strong and, except within 2–3m of the surface, joints are usually tight, sometimes with calcite infill.

The central tower foundation is a 25m x 35m gravity footing founded at -5 m OD.

The envisaged construction methodology* was that a platform would be formed using marine plant working under water. A precast cellular foundation would then be floated and ballasted into position on pre-installed landing pads and the gap beneath the unit infilled by underwater grouting to form the contact with the excavated rock surface. The cells would then be filled with in situ concrete.

The flanking towers are supported on 29 m x 41 m pile caps with a group of 16 No. 3.4 m diameter cast-in-place piles. The top elevation of the pile caps is at 25 m OD so that the pile caps are not visible and are sufficiently deep at low tide so as not to pose a hazard to yachts or other leisure users of the Forth estuary. Foundation conditions vary, with the more challenging south tower being located in 22m of water with rockhead at -40m OD. The sedimentary rock is overlain by glacial deposits and alluvium. The envisaged construction methodology was again based on float-out of cellular precast caissons which make up the pile cap and tower base.

A similar sequence was considered for the north tower, although some dredging was required as the pile cap is partially below the existing seabed. Four piles were to be constructed using a template placed on the seabed. The caisson was floated in at high tide and sunk over the piles at slack water when the tidal current drops below 0.5 knots for at least two hours. The structure was ballasted on the falling tide so that it remains seated on the guides at subsequent tides.

The first four pile sleeves were grouted and temporary buoyancy elements removed. Further piling operations then took place, using the pile cap as the guide for driving the pile casing. Various dockyard sites close to the bridge site or further afield were considered for the precast units. Construction on a submersible barge moored alongside a quay or quayside construction followed by load-out onto a transport barge were also viable construction option.

** It should be noted that the construction methodology described here was that envisaged in the Specimen Design. In the final design, FCBC chose to adopt gravity foundations using caissons and cofferdams, rather than a piled solution. The final design solution is described in Part III..*

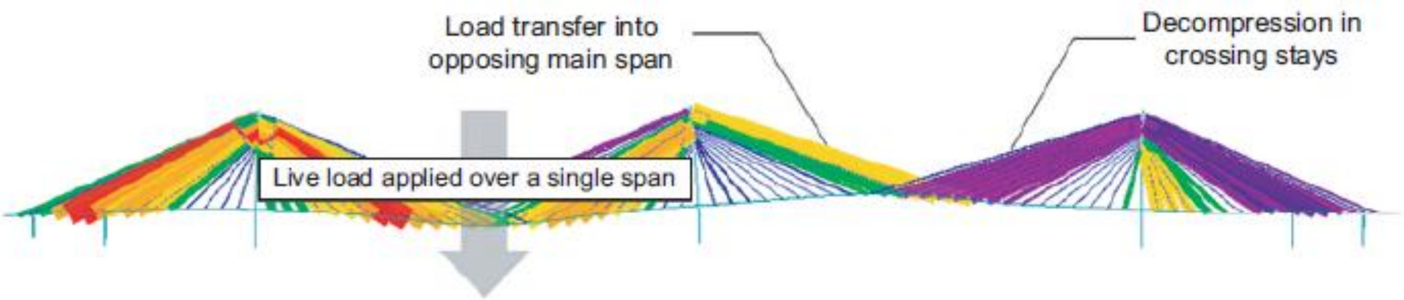


Figure 3: Crossing stay cable system: cable tensions due to out-of-balance loading

3.7 Crossing stay cables

3.7.1 Design solutions for a multiple main span cable stayed bridge

The problems of a cable-stayed bridge with three or more towers are well recognised. The central tower cannot obtain direct support by being tied back to anchor piers and out-of-balance live load on only one of the main spans would cause a significant sway of the central tower, resulting in large deflections and large bending moments in the tower and deck.

A number of parametric studies were carried out to investigate tower and deck stiffness. The method finally adopted in the specimen design was to overlap or cross the stay cables over approximately 25% of each main span. By overlapping the stay cables in this way, a virtual truss system is developed which provides overall global stiffness, improving both the static and dynamic performance.

When out-of-balance live load is applied to one main span, the tower movement causes the stays to lift the opposing main span. Over the region of the crossing stay cables a decompression is developed in the stay cables connected to the far flanking tower, which in turn is tied back to the far anchor piers.

3.7.2 Performance of the crossing stay cable system

The deflections of the bridge were considered when only one of the main spans is loaded. The specimen design and an identical bridge without crossing stay cables were compared (load model 1 was applied in accordance with the UK national annex to Eurocode 1 (BSI, 2008)). The orthotropic deck variant is generally more flexible than the composite deck because the lighter construction means reduced stay cable quantities with a consequent reduction in global stiffness. For this variant, the deflections increased if crossing stay cables are not provided, this would cause concerns over serviceability performance.

Deflections alone do not demonstrate the need for crossing stay cables for the composite deck variant. However, their benefit is more clearly revealed when the bending moment in the deck is considered. Figure 4 shows the live load bending moment envelope for the case with and without crossing stay cables. Both main spans are shown with the central tower located at x 5 0 m. Significant moments are developed in the deck when crossing stay cables are not provided. The virtual truss system is effective at reducing these moments, allowing savings in structural steel and preventing crack control difficulties in the concrete slab of the composite variant.

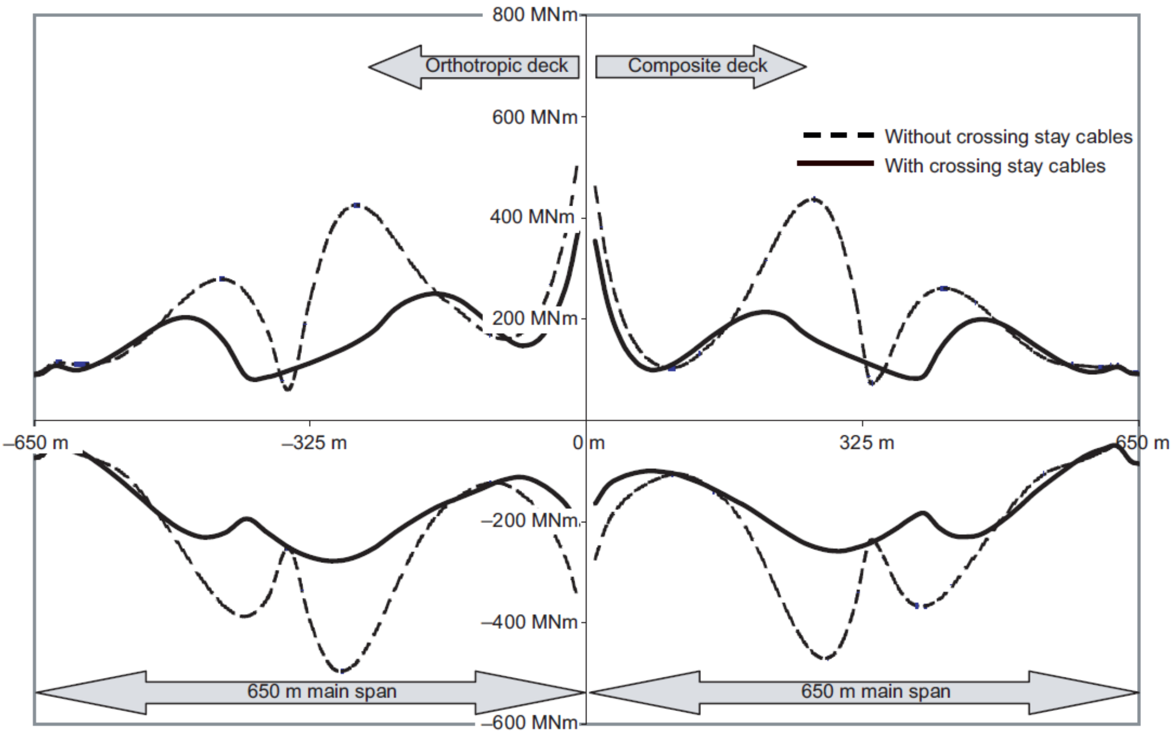


Figure 4: Crossing stay cable system: cable tensions due to out-of-balance loading

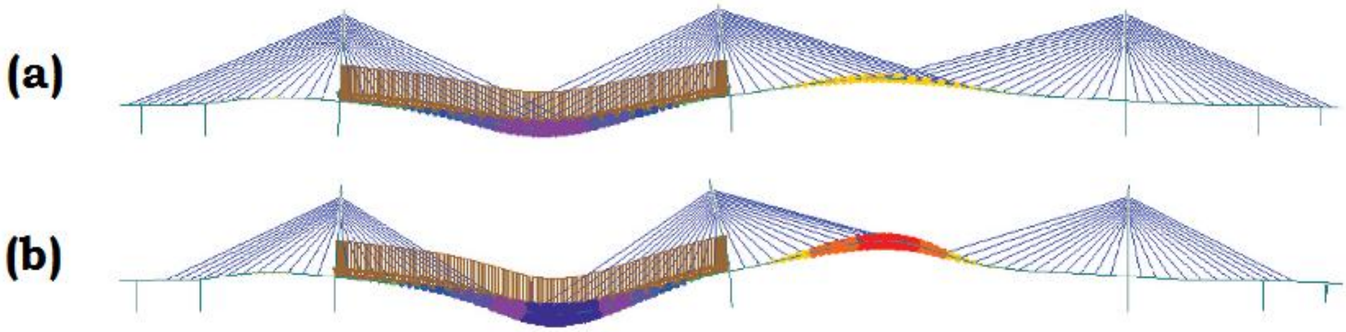


Figure 5: Scaled bridge deformations for live load on one span, (a) with (b) without crossing stay cables

3.7.3 Construction of the bridge with crossing stay cables

The typical construction method for a cable-stayed bridge is to cantilever to the middle of the main span and then install a closing key segment. The same option is also available for the case with crossing stay cables, but particular investigations were made of the stability of the central tower at Beamer Rock for the case immediately prior to closure of the main spans. Wind buffeting analyses were carried out and these indicated that the gravity footing will be stable and that although the torsional loads in the tower below deck are governing they are acceptable.

A further complicating factor is that in the crossing region the stay cables are only sized to carry a reduced gravity load, which is shared with the stay cables from the opposite tower. As these cannot be installed until after the key mid-span segment, the stay cables are undersized for the cantilever construction case. If single-stage stressing is assumed for the stay cable installation, then the stay cables do not fully support the cantilever and hogging moments occur in the deck, which would then require significant strengthening. However, investigations were carried out to demonstrate that a two-stage stressing sequence could overcome this. The stressing sequence developed involved additional stressing and completely avoided de-tensioning of stay cables, making it practical for a multi-strand system.

An alternative method of constructing each cantilever to the beginning of the crossing stay cable region and then erecting a 136 m long central part of the deck in one piece using a heavy lift system was also considered.

4. DESIGN FOR EXTREME EVENTS

4.1 Wind

During development of the specimen design, a number of activities were carried out to establish wind climate at the site and investigate aerodynamic phenomena. A detailed wind climate analysis provided the design wind speeds for the bridge as well as defining the turbulence intensity and other parameters to allow a wind buffeting analysis of the dynamic response of the structure to gusting wind patterns. Data covering a 35-year period were available from an anemometer mounted on the existing Forth Road Bridge deck; this provided a valuable source of data for the site.

However, flow conditions around the bridge deck affect the anemometer and so data sourced from Edinburgh Airport and design standards were also used as references. For winds perpendicular to the bridge, the ultimate limit state 10 min mean wind speed at deck level is 42.3 m/s (95 mph). This corresponds to a return period of approximately 6000 years and is some 15% higher than the upper limit of gale force 12.

Two stages of deck sectional model wind tunnel tests were carried out. Preliminary tests at 1:50 scale were carried out on a number of different sections at the BMT fluid mechanics tunnel (Teddington, UK) to investigate aerodynamic stability and force coefficients of several different bridge configurations under consideration. After selection of the preferred scheme, additional tests were carried out at 1:40 and 1:30 scales at the Politecnico di Milano (Italy) to investigate a number of different options for wind shields on the bridge deck; these tests confirmed the aerodynamic stability and force coefficients as well as establishing the shielding effects of the wind shields.

Continuous 3.5m high wind shields with approximately 50% porosity are provided on the bridge to ensure a more reliable service than the existing Forth Road Bridge which suffers frequent traffic restrictions and occasional

Figure 6: Wind Tunnel Testing: Deck section and balanced cantilever from Central Tower

closures due to high winds. The purpose of the wind shields is to reduce wind speeds across the top of the deck to an acceptable level.

Since high-sided vehicles are more susceptible to problems, it was found to be more efficient to have greater shielding at a higher level. Making the wind shield more open at a lower level also reduces visual blockage. However, it was found that one rail is required at low level and this was carefully positioned to be behind the second parapet rail to minimise visual obstruction.

4.2 Earthquake

UK application of Eurocode 8 states that there is generally no need to consider seismic loading except for 'certain types of structure, [which] by reason of their function, location or form, may warrant an explicit consideration of seismic actions'. For the Queensferry Crossing, the scale of the structure and the potential consequences of failure warrant such consideration. An assessment of the site concluded that (a) there were no known active faults at or near the site (b) the levels of ground motion would be insufficient to cause liquefaction (c) the hazard from tsunami is sufficiently negligible to be excluded from consideration.

Based on this assessment, the specimen design was verified with a response spectrum analysis following the provisions of the Eurocode. Two performance levels were established, with the higher level corresponding to a 2475-year event under which the bridge may undergo limited ductile behaviour but should be open to emergency vehicles immediately after the event.

4.3 Ship impact

Each of the main spans crosses a navigable channel. The southern span crosses the Forth deepwater channel, which is the main shipping route in the Forth estuary, providing access to the ports of Grangemouth and Crombie. The northern span crosses the approach channel to the port of Rosyth immediately upstream of the bridge. Vessels of up to 40 000 DWT (deadweight) typically pass under the bridge and ship impact is an important design consideration that required thorough investigation.

The Forth Ports vessel traffic service (VTS) provided a rich data resource giving details of all vessels entering or leaving the upstream ports, recording the name of ship, time of movement, type and size of vessel, cargo and draft. A database of these records was established and analysed. The VTS system also incorporates radar tracking that records the timing, position, speed and heading of all vessels within the surveillance area. Routes were identified from the raw radar data using geographic information systems (GIS) software.

The VTS radar paths showed that, with 650m main spans, the proposed tower locations were well clear of the existing vessel transit paths and that ships would not have to modify their navigation routes once the bridge is built. This was confirmed by navigation simulations carried out at South Tyneside College where local pilots independently reported that 'the completed bridge will have little impact on the ability of ships to navigate in the River Forth' (Michel and Walker, 2009).

A quantitative marine collision risk assessment, based primarily on Eurocode 1 (BSI, 2006), was carried out to assess the design impact forces for each of the foundations. Risk acceptance criteria were established considering the as low as reasonably practical (ALARP) principle. Dynamic analysis was carried out to assess the foundation capacities using large-displacement finite element models and considering energy absorption in plastic hinges in the piles. This ductile design approach as expected showed significant additional capacity compared with an elastic design.

QUEENSFERRY CROSSING

III. Final Design & Construction Process

1. INTRODUCTION

The Queensferry Crossing is a cable stay bridge with three towers over 200m high and the cable fans arranged centrally between the two carriageways. The two main cable stayed spans are 650m, the two back spans are 223m and there are approach viaducts at each end of variable spans up to 104m.

Transport Scotland undertook a year-long tender dialogue process during 2010 for a design and build contract to construct the Queensferry Crossing and connecting roads. Forth Crossing Bridge Constructors was the successful tenderer. FCBC's contract commenced in April 2011 and will be completed in summer 2017.

2. FOUNDATIONS

The location of the crossing exploits a narrow crossing point within the Firth of Forth estuary, but the depth of water is comparatively deep at up to 45m. However, the estuary is punctuated by a small rock island called Beamer Rock which divides it into two channels. This shallow tidal rock island was defined as the central tower location. The remaining two towers sit towards the edge of the deep channels where the water depth reduces to circa 15m.

The foundation solution had to address a range of marine conditions, from shallow tidal waters to deep water conditions. The competent founding strata for the Flanking Towers was the sedimentary rock, below alluvial, fluvio-glacial and glacial soils, at a depth of up to 20m below bed level.

While the specimen design adopted piled foundations for the piers and flanking towers, FCBC chose to use gravity foundations throughout. All three towers have reinforced concrete foundations, however the Central Tower has differences in construction compared to the Flanking Towers (North Tower and South Tower) due to the differing sub-structure foundation and bridge articulation.

The South Tower structural foundation (30m(dia)*9.0m high) is also larger than the North Tower structural foundation (24m(dia)*9.0m high), primarily due to the design for potential ship impact forces. The Central Tower foundation (35.0m*25.0m*3.50-6.0m) sloping surface was formed using sacrificial stainless steel "Hy-rib" panels, however as with all other parts of the foundations this is not visible at low tides due to rock armour positioned on top of the foundation and blended into the surrounding "Beamer Rock".

2.1 North and South Towers

The North and South (flanking) towers were built within caisson foundations and were designed to be founded on the bedrock. The steel caissons are typically 30m in diameter up to 41m in height and weighing in excess of 1200 tonnes.

The caisson consists of two parts. The bottom part of the caisson (permanent caisson) is a double walled 30m diameter and 30m high steel ring. On top is a temporary caisson, a single skinned sheet pile wall that was removed at a later stage.

The temporary caisson is 11m high and creates a dry working environment for the foundation and early stages of the tower works. Both parts of the caisson were fabricated in the CRIST ship yard in Gdynia, Poland and trial-assembled to guarantee a tight sealing.

Throughout fabrication weld and pipework testing was undertaken, and after completion as-built dimensional surveys and trial builds with the permanent and temporary caissons were undertaken.

The caissons were transported by a semi-submersible barge from Poland to Rosyth. They were off-loaded in the middle of the Forth from the semi-submerged transportation barge by a shear-leg crane and moored. The barge had to be partly submerged to reduce the lifting weight of the caissons from up to 1200 ton to 650 ton by using the uplift effect of the hollow double skin wall.

The initial placing of the caisson on the seabed had to be within a design tolerance of +/- 250mm in 20m water depth. Placing was measured using GPS at the shear-leg crane mounted on the caisson which enabled the monitoring of the exact position of the caisson including its tilt during the whole initial placing process.

The caissons were lowered by removing material from the inside creating ground brakes underneath the caisson wall and therefore overcoming friction and sinking the caisson using gravity and its own weight. Two barge mounted clamshell excavators were used in parallel to control the tilt during sinking process.

A “sealing-ring” made of a double-interlocking row of jet grout columns at -40m between the caisson and the rockbed was required to allow final excavation and cleaning of the rock bed.

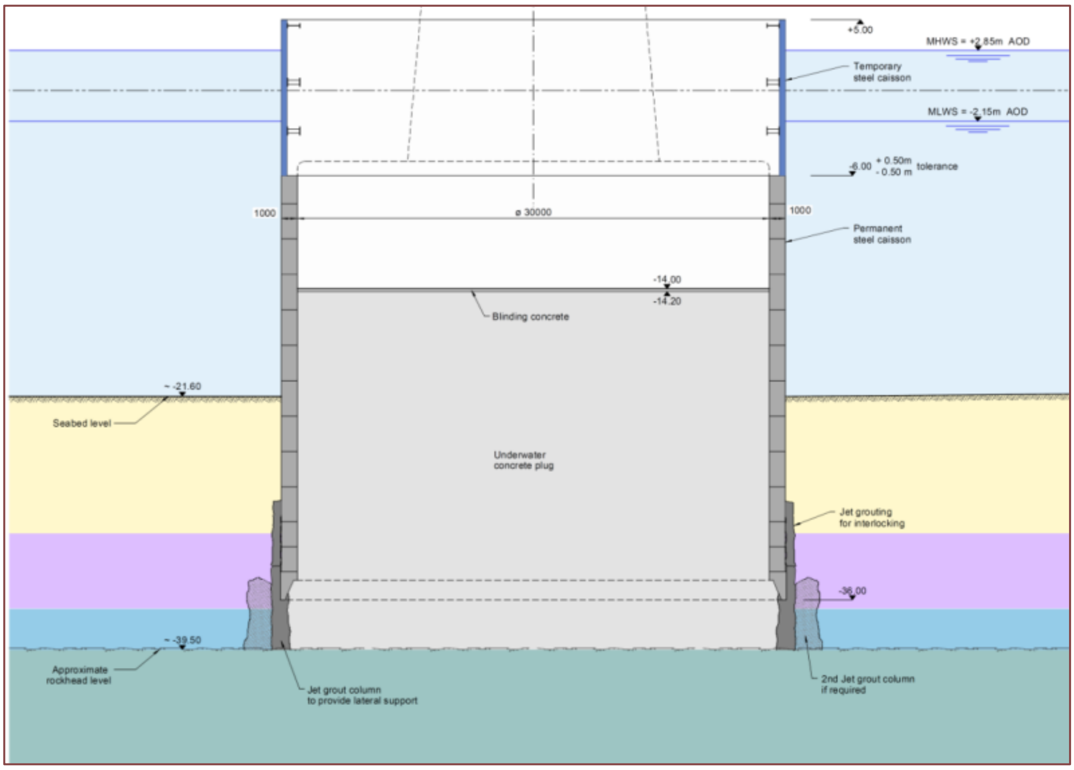


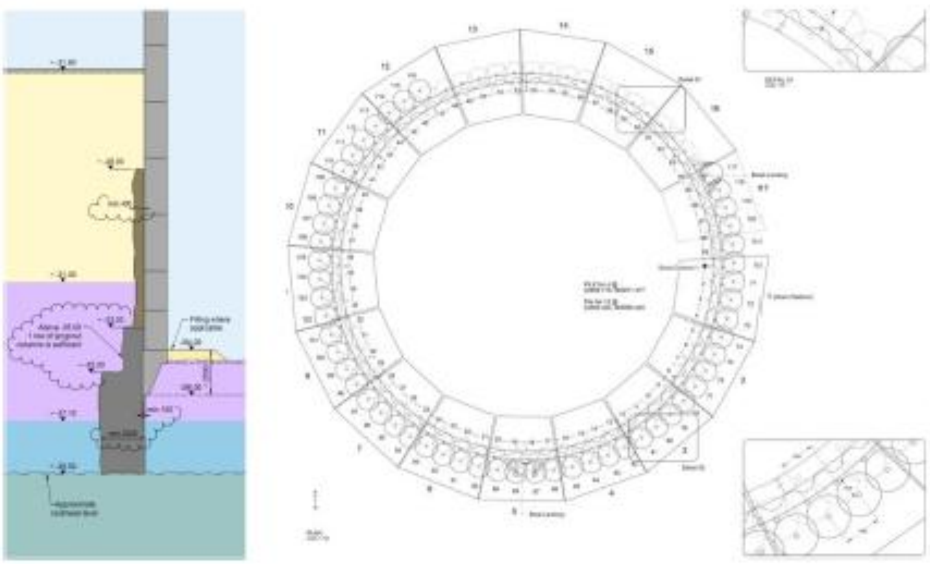
Figure 1: Typical Caisson cross-section

Figure 2: Permanent Caissons in a line during fabrication

Figure 3: Off-Loading with Shear-leg crane in the Forth

To achieve a plug seal, underwater concrete was required to be poured in a continuous process without interruption. 16,852 m^3 of concrete was supplied at the South Tower (the largest continuous offshore supplied underwater concrete pour in the world), continuously supplied with an average rate of 60 m^3/hr, to the deposition point over 2 miles away from the loading station at the quay wall.

After pouring the underwater concrete plug the caisson was dewatered and the top of plug cleaned and inspected. Once blinded, the reinforced concrete foundation could be constructed.



Design:

- 0.80 m to 2.00 m "Jet Grouting Wall" from interlocking columns
- Single or double row, 1.85 m diameter, rock at -40.5 m AOD
- 121 columns South Tower, 60 columns NT, 70 column S1
- 4000 kg / m cement for columns, 1800 kg / m friction grouting

Figure 4: Design of jet columns

Figure 5: Pour Set- Up at Caisson

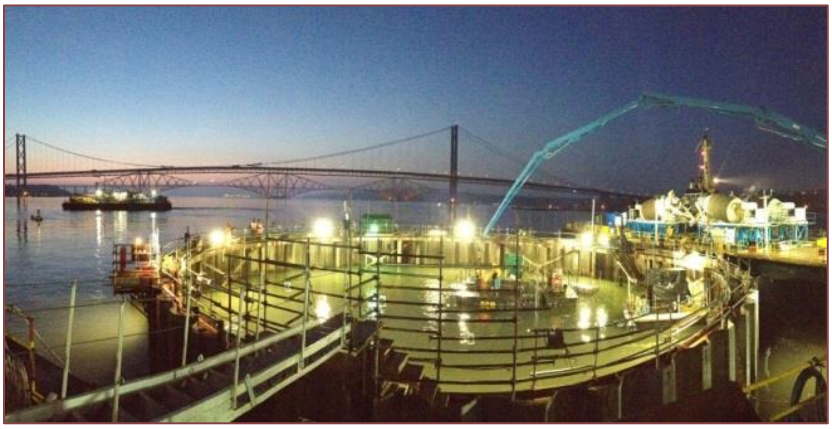


Figure 6: Continuous Pour 15 days

2.2 Central Tower

For construction of the reinforced concrete foundation in dry conditions for the Central Tower a modular cofferdam was selected.

The ten modular precast cofferdam units were constructed in the port of Rosyth. The sheet pile sections were slid together horizontally with the reinforced concrete base slabs being cast as a wall and then the unit rotated into an upright position.

To form the foundation pocket blasting was undertaken to first fracture the strong basalt rock. Bulk excavation followed four production blasts to form the general pocket. Following excavation and before the units were installed, 44 number 63.5mm diameter rock anchors were installed from a jack-up barge and a small modular sister platform. These were designed to hold the cofferdam units in place during the dewatered stage.

The cofferdam units were installed into the perimeter trench from the GPS Atlas floating crane with diver support. Each unit was placed and then the annulus between the unit and the formation filled with underwater concrete. With all the cofferdam units fixed into position, the gaps between each unit were filled and sealed. Large section props were then installed to ready the cofferdam to resist hydro static loads. Finally, de-stressing wells were installed in the rock formation to reduce up-lift pressures which might heave the formation on de-watering.

Once the cofferdam was fully constructed several large 150mm pumps were engaged and the water removed to expose the formation within the centre of the cofferdam.

Figure 7: Cofferdam during construction

Figure 8: Jack-up rig within the foundation pocket

3. TOWERS

3.1 Construction

Tower and foundation structural heights are of 215.717m for Central Tower and 216.367m for both North Tower and South Tower.

Each tower is of hollow reinforced concrete construction and tapers both longitudinally and transversely. At the base, the tower has outer dimensions of 14m longitudinally and 16m transversely. The tower tapers linearly in both directions:

- longitudinally to 10m at deck level and 7.5m at the tower top
- transversely to 8m at deck level and 5.0m at the tower top

Central Tower required thicker tower walls compared to the Flanking Towers (North and South Towers).

Longitudinal fixity is provided by a monolithic connection at the Central Tower with transverse support provided at all towers and piers.

As a result of this longitudinal fixity, the tower reinforcement quantities at the Central Tower (CT) are greater than those at the flanking towers (ST and NT), with typical examples of quantities below:

		ST	CT	NT	Total
Structural Foundations	Rebar	553 [t]	680 [t]	360 [t]	1,593 [t]
	Stainless rebar	0	21 [t]	0	21 [t]
	Concrete	6350 [m^3]	4348 [m^3]	4068 [m^3]	14766 [m^3]
Towers	Rebar	2150 [t]	2800 [t]	2150 [t]	7100 [t]
	Stainless rebar	41 [t]	32[t]	41 [t]	114 [t]
	Steel anchor boxes	445 [t]	415 [t]	445 [t]	1305 [t]
	Structural concrete	8600 [m^3]	8580 [m^3]	8600 [m^3]	24780 [m^3]

Table 1: Typical tower material quantities

Due to the high reinforcement tonnages in the lower level of the tower walls (41t maximum per wall elevation), the reinforcement was primarily constructed in-situ (lifts 1 to 20). Prefabrication of entire wall elevation reinforcement cages was utilised from lift 21 upwards as the 18t maximum wall elevation reinforcement weight was within the operational crane lifting limits.

Segments 38 to 51 incorporate a steel anchorage assembly for the stay cables within the tower head. Stainless steel pipes were cast into the concrete wall and connected to the steel stay pipe. Within the stay pipe there are cable guide deviators, to assist with cable installation, and dampers that ensures the dehumidified zone within the upper zone of the tower remains effective.

Fast construction was achieved by careful planning of the construction process using fabricated self-climbing formwork. All construction materials and cast-in items were delivered to the work point by barge and lifted into position by the tower crane.

With favourable wind conditions, a repeating cycle of 7 working days for each 4m construction lift was achieved.

Self-climbing tower cranes were placed outside of the deck footprint to enable continuous climb, unrestricted by deck erection activities.

All concrete was transferred from the marine yard concrete batch plant (typically 90m^3/hr concrete production) to the towers via barge.

Once at the towers, the concrete barges (72m^3 capacity) were pumped into a further pump on the crane working platform prior to being pumped to the concrete distributor.

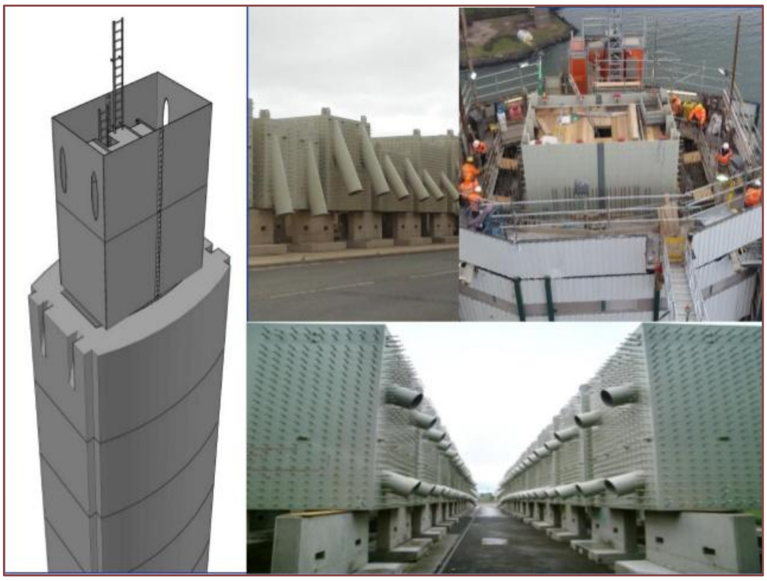


Figure 9: Tower steel anchor boxes

Figure 10: Barge to tower prefabricated wall

Figure 11: Concrete barge and distributor

3.2 Temperature Match Curing (TMC) and Heat Calculation Curing Plan

With tower wall concrete thickness exceeding 1m, each tower lift was monitored, via thermocouples cast-in to the north face, to ensure that the maximum peak (75°C) and differential (<28°C) internal hydration temperature requirements were not exceeded for individual concrete pours. 8 number 100mm cubes were taken from the last concrete barge delivery, placed in the water bath on the jumpform and the water bath connected to the cast-in thermocouples via wires projecting vertically through the construction joint. The TMC concrete cube test results provide evidence that in-situ strength of concrete pours reached the minimum requirements for both the removal of shutters (6.0N/mm^2) and/or the jumping of the formwork (10.0N/mm^2) in accordance with the designer's heat calculations, for summer and winter, that set requirements for early striking of the shutters to minimise risk of excessive cooling shock resulting in early thermal cracking.

The points of the segment pour centrelines, bridge and tower axes reference lines, curves and their centre points and local jumpform triangulation survey checks were marked using the 4 Global Navigation Satellite System (GNSS) on the internal and external jumpform working platforms and by Totalstations within the inner and outer jumpform platforms. Pairs of monitoring prisms were installed on the external tower face in every 5th segment and surveyed from the adjacent tower. As-built surveys from the previous segment pour were undertaken on the day after pouring concrete, processed and used for any required shutter adjustments for the proceeding lift. Allowable plan positional tolerances of +25mm (corners), +10mm (centrelines) and +10mm level (centrelines) were recorded for every segment.

3.3 Structural Health Monitoing System sensors

The surface area of the towers is approximately 26,000m^2 and whilst every part of the structure is accessible at touching distance, the towers had various sensors installed to monitor the structure. These sensors comprise of:

- Cast in: strain gauges (static), corrosion sensors, temperature (concrete),
- Post-fixed: strain gauges (dynamic), tiltmeters, accelerometers, temperature (air), GPS, anemometer.

Figure 12: North tower – temporary formwork is lifted off

4. DECK

4.1 Introduction

The structural steelwork for the deck was fabricated partly in China (main cable-stayed deck and components for the North Approach Viaduct) and partly in the UK (South Approach Viaduct, and assembly of the North Approach Viaduct). The main cable stayed deck is formed by a single box and constructed by the balanced cantilever technique, working on the two fronts away from each of the towers.

The South and North approaches are twin box constructions and they were assembled on site behind the abutments and launched across temporary bearings on the piers towards and over the water.

4.2 Approaches

The approach spans are twin open topped steel boxes, with composite decks and double composite at the supports. The South Approach Viaduct (SAV) totals 543m over eight spans, and the North Approach Viaduct (NAV) has just two spans. The maximum approach span is 104m in both the north and south approaches.

4.3 Launching of the South Approach Viaduct

The approach viaducts were both assembled and welded on site in twelves phases, alternating west and east. Each phase, equating to one span length of east or west box, was launched across the abutment towards the river, before the next phase could be delivered to the assembly area. The composite decks were cast after all launching was completed and the steelwork was transferred onto its permanent bearings.

The progressive launch was carried out using strand jacks fixed to the bearing pedestal of the abutment structure with the strands attached to welded “lugs” near the rear of the assembled box girder. Vertical and horizontal alignment was controlled on each of the six piers. The tip of the deck and the deflections at the cantilever were controlled by strands attached to the 35m high steel king post.

Launch phases ranged from 60 to 120m which with an average speed of 7m per hour gave an operation time of just two days.

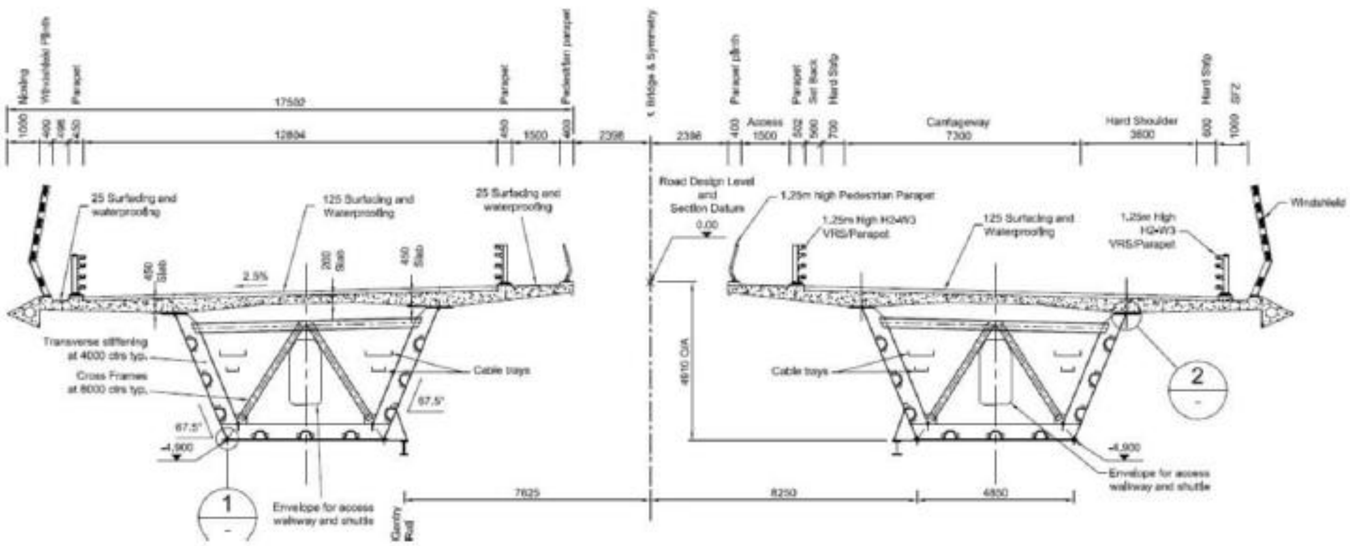


Figure 13: Cross section through approach viaduct decks with twin boxes

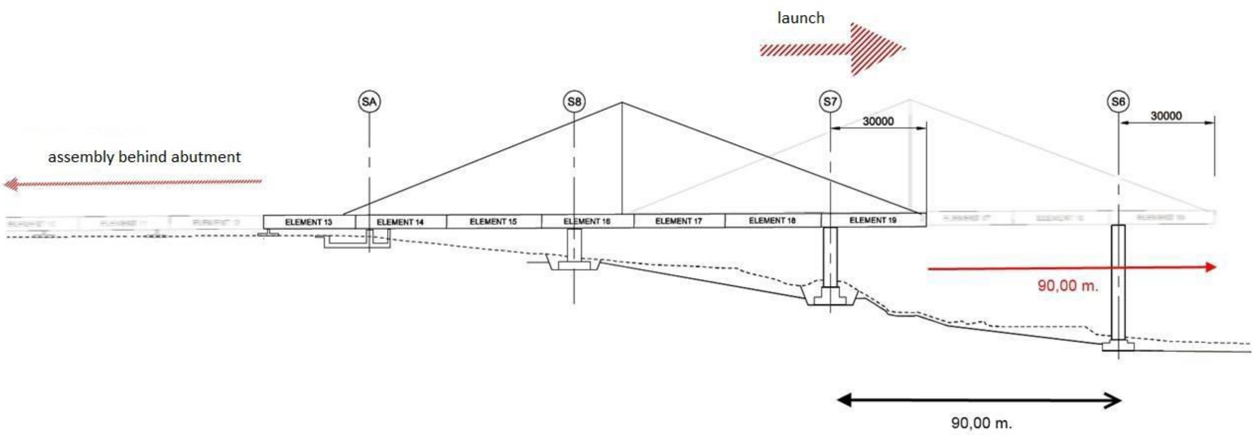


Figure 14: Typical south approach launch phase

Once the final launches were completed, the temporary king posts, bearings and guides all had to be removed.

Installation of the permanent bearings and final internal slabs followed before the construction of the in-situ reinforced deck could commence.

4.4 NAV assembly and launch

The North Approach Viaduct steelwork was similarly assembled and welded on stillages behind the abutment as the South Approach. However the North Approach is only 221m in total length, and was launched after full assembly as a single element in the steel only condition.

These spans comprise a 75m length of twin box in the first span, a transition segment, and 146m of single box complete with stay anchor webs, which connects to the cable stayed north fan.

Figure 15: SAV box girder and king post

Figure 16: SAV Complete

Figures 17 + 18: NAV during launch

5. CABLE STAYED DECK

5.1 Deck segments

The cable stayed deck comprises a single composite box girder approximately 40m wide and 4.9m deep comprising a steel open topped box „tub“ with a reinforced and post-tensioned concrete slab with extending side cantilevers. Cables descend in two planes approximately 5m apart from each tower to stay anchor webs either side of the bridge´s centre line, with crossing stays employed mid span to stabilise the central tower.

The segments are 29.8m wide and 16.2m long, weighing up to 320t. The cable stayed deck totals almost 31,000 tonnes. The segments were fabricated in China and delivered directly to the port at Rosyth. Each segment was trial assembled with its adjacent segment in correct alignment to achieve correct fit-up at the welded joint and match drill the bolted stiffener connections.

Each of the steel sections is made up of five key components; bottom plate, inclined web assemblies, stay anchorage webs, side cross frames and centre cross frames.

A 250mm thick reinforced and prestressed composite slab is connected by conventional shear studs to the steel superstructure. The slab thickens to up to 600mm at the towers. The slab is transversely post-tensioned to limit cracking in service. The fixed point of the bridge is at the Central Tower where the composite deck is effectively clamped to the tower by means of longitudinal and transverse post-tensioning through monolithic construction at the so-called Central Tower ‘power joint’. At the flanking towers, lateral support only is provided by means of a series of free sliding (non-guided) spherical lateral bearings, along the horizontal principal axis.

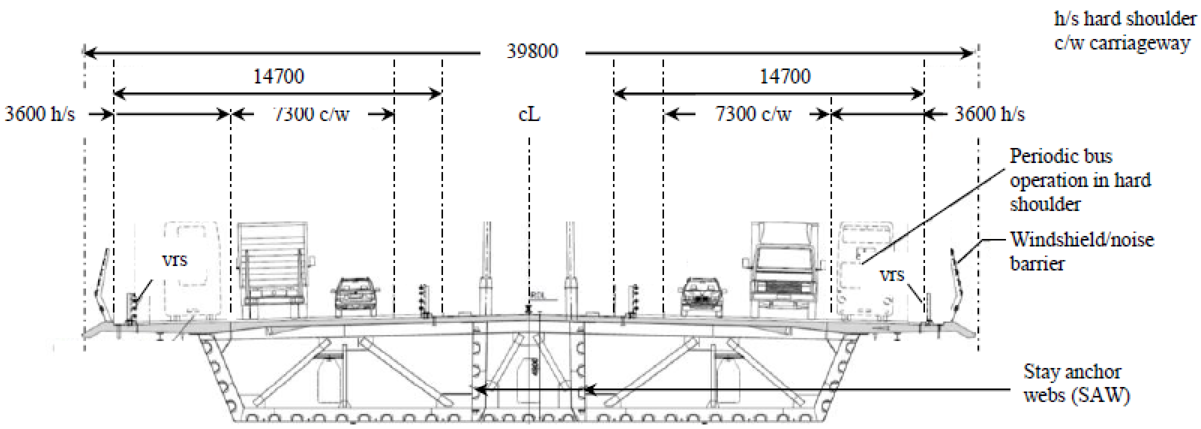


Figure 19: Typical cable stayed bridge deck cross section

5.2 Cable stayed deck erection

The main cable stayed bridge was constructed by the balanced cantilever technique, working on the two fronts away from each of the three towers.

Installation started with temporary falsework legs and platforms that were installed at each tower and four permanent segments were installed. The deck starter segments were erected at each of the towers once the towers had reached the 18th of the 54 lifts in total (210m). After their installation they received temporary attachments to allow for the Erection Traveller (ET) to be installed.

The main deck segments are lifted up from the river into position by the ET. Each lift is typically a 16.2m long deck segment complete with the reinforced post-tensioned concrete deck. The transverse in-situ welded joints, cable stay installation, and a short in-situ concrete deck stitch are carried out and the ET is moved along the cantilever for the next lift. The composite decks were adequately cured and the lateral post tensioning applied. Finally, closure segments are installed between the cantilevers.

The first of the cable stays, at Lift 38, was installed once the towers reached Lift 40. The temporary tower leg trestles were installed to provide support to the temporary works platforms and deck works prior to the installation of the stay cables.

In deck erection six erection travellers (ET) were utilised simultaneously, one each positioned at the cantilever ends. With the exception of the starter segments, all cable supported deck segments were erected from a delivery barge.

Deck deflection increased significantly as the deck cantilever lengthens. At its maximum extent, cantilever deflection were 2m requiring four strokes of the 500mm strand jack to overcome this deflection alone.

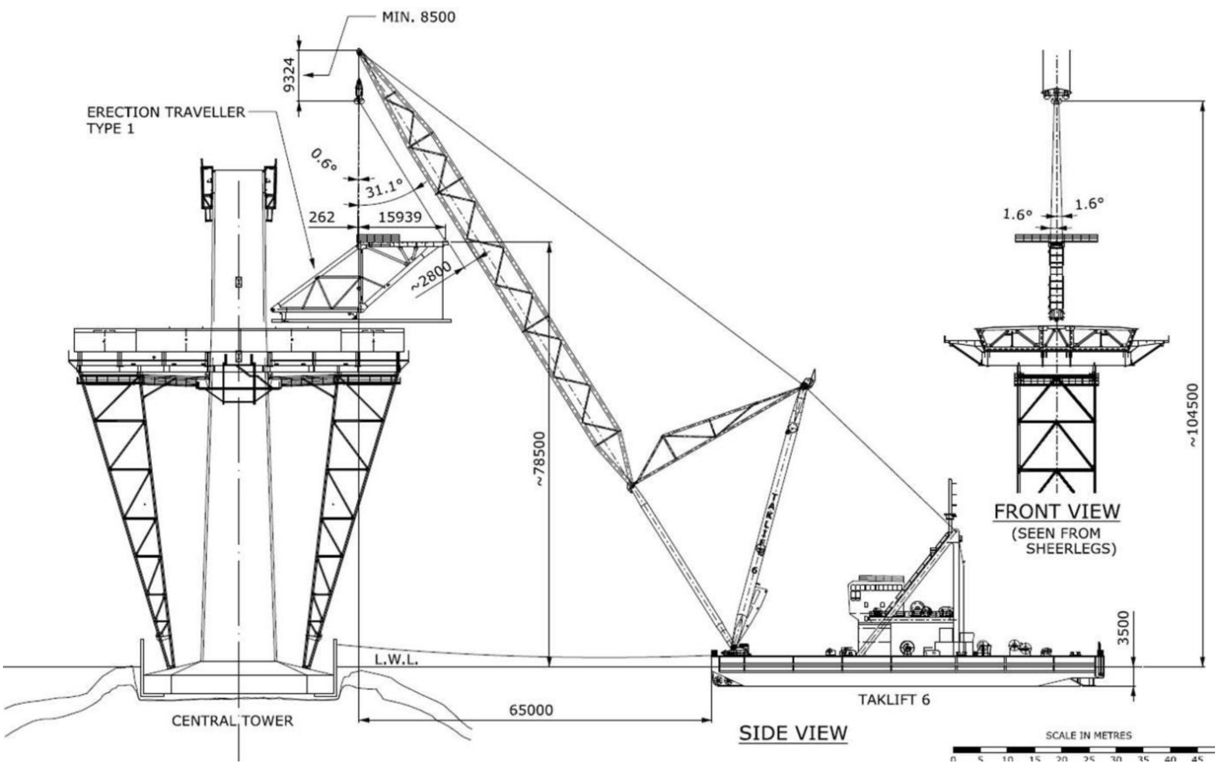


Figure 20: Taklift 6 lifting the Erection Traveller onto Central Tower starter segments

Figure 21: Deck section is lifted up on Central Tower

Figure 22: Segment between SAV and South Tower

5.3 Erection Travellers

The main frame of the ET is a single truss, with primary members consisting of I-section plate girders with flanges in the vertical plane. Two front and rear supports to the main frame are spreader beams on longitudinal beams. The lifting jack carriage tracks backwards and forwards along the top flanges. Movement is achieved with a pinned twin cylinder launching jack system.

Upon completion of segment erection, removal of each ET had to be done piece-meal owing to its location between the permanent stays with adequate protection provided to the cable stays.

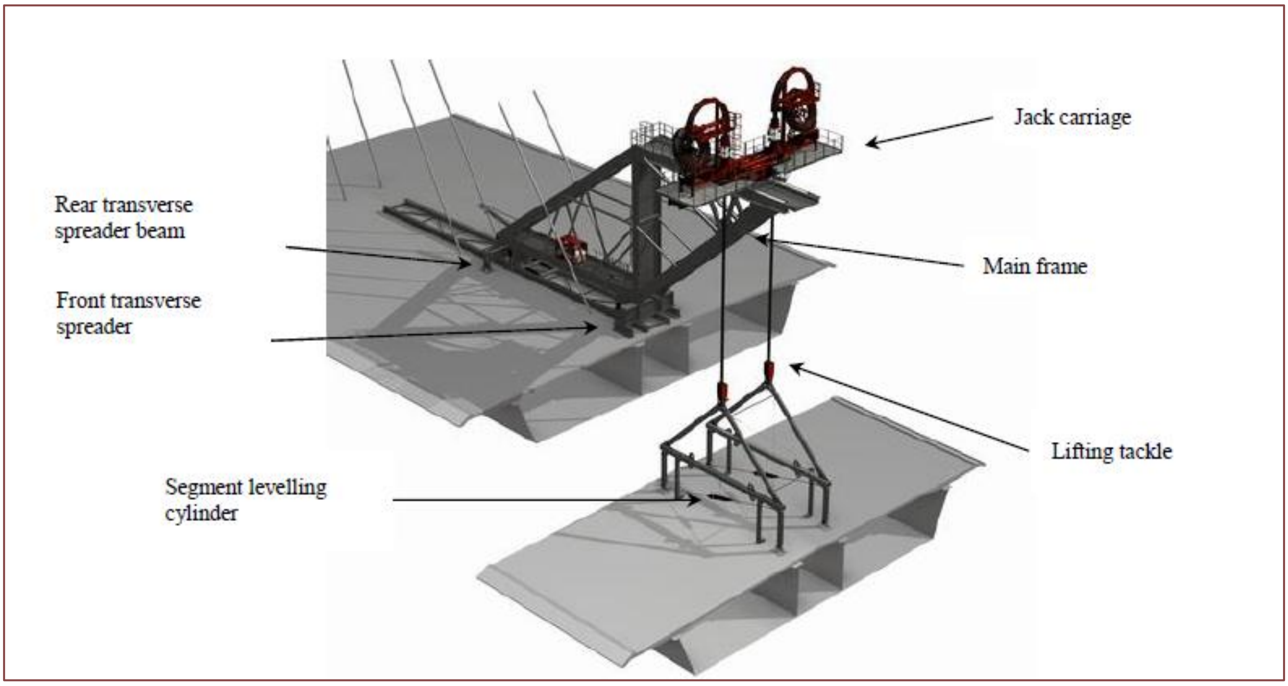


Figure 23: Erection traveller

5.4 Temporary tie-down stays

As the cantilevers extend from the towers, they become vulnerable to wind buffeting resulting in possibly large deflections. The considerable energy of wind in the low frequency range corresponds to the natural frequency of the erection configuration. Temporary tie-down stays are installed as the cantilevers extend to stabilise against buffeting. These are located on the opposite sides of the leading cantilever on each tower and also assist in countering the bending moment in the towers caused by the imbalance from the leading segment. These stays are installed when the cantilevers reach 95m and 160m for the Flanking and Central Towers respectively.

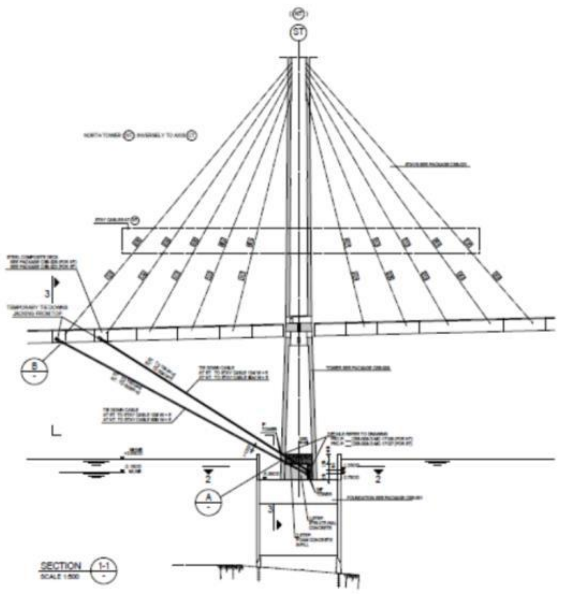


Figure 24: Flanking towers temporary stays

5.5 Geometry control

During construction of the bridge, the *'stay length'* method by which the global geometry can be controlled is adopted. It makes use of the fact that the geometry of the prefabricated members of the bridge can be measured prior to erection under well controlled conditions and with high accuracy.

The elevation of the cantilevers can be controlled by adjusting the length of the sides in a triangle comprising the lengths of the stay, the cantilever and the height of the tower.

During segment erection, the length of the cantilever and the height of the tower are monitored and compared to their theoretical values. If these two dimensions differ at the given construction stage, it can be assumed that this deviation too will be present in the final bridge unless a cable length adjustment is made. Stay cable lengths are only corrected for deviations which are predicted for the final bridge and not for deviations that are temporary.

Differential weight in opposing cantilevers causes deformation in the tower fan. The tower top deflects in the longitudinal direction. The cantilevers tips deflect vertically.

When the cantilevers are short, deformations of this type are relatively small. If an imbalance of more than 1% occurs, temporary ballast (eg concrete blocks or strand coils) is placed on the lighter cantilever. Where the detected imbalance is considerable, permanent ballast may be required.

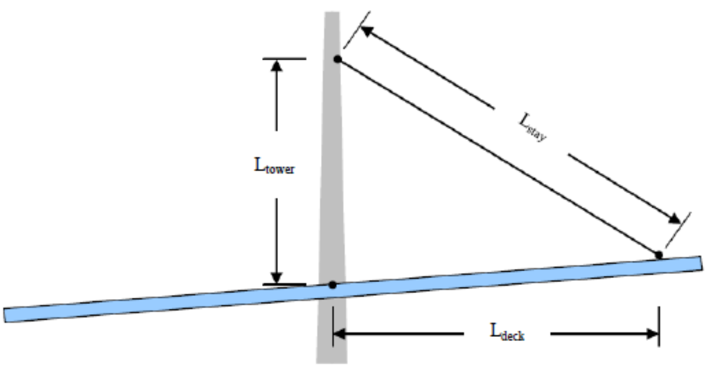


Figure 25: Geometric parameters governing the elevation of the cantilever

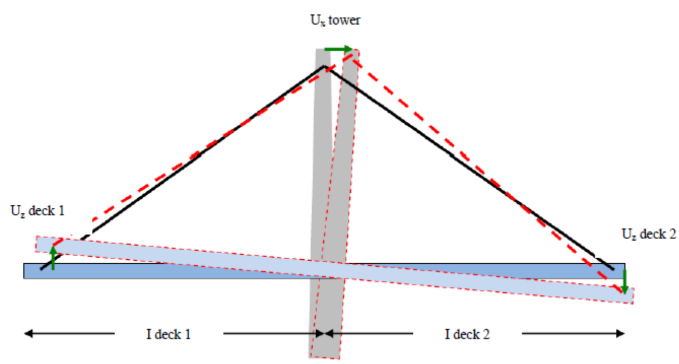


Figure 26: Deformations due to differential weights

6. CABLES

The stay cables used on the Queensferry Crossing consist of a parallel strand system. This system is more expensive than the alternative – a parallel wire system, but was specified for ease of future maintenance. Parallel strands can be replaced individually as a maintenance operation, wheras parallel wires would require the replacement of an entire cable. Stay cables range from 109 strands down to 45 strands depending on their location within the structure and the stay lengths vary from 97 meters to 422 meters in length.

The strands were supplied in reel-less coils thus reducing the waste associated with having to scrap the typical wooden reels. The initial pilot strand and HDPE pipe was pulled up from the deck level with the tower crane and secured to the anchorage inside the tower anchor boxes. The cables were stressed at the live anchorage which was always located within the towers. Strands were pulled up to the tower anchorage one at a time with a typical pulley and shuttle system.

Due to the tight geometry within the tower anchor boxes it was only possible to install one set of stay cables on one face of the tower at a time. One set of stay cables was typically installed over a 3 day duration meaning that the opposing balanced segments were always offset from the segment erection by at least the same time frame.

Some of the stays were installed with temporary shims creating higher erection stage tensions in order to deal with the complex closure geometry imposed by the crossing stay feature on the project. The temporary shims were later removed with a large multi-strand jack after the span closures were completed.

After final tuning of the stays was completed in conjunction with calibrating the structural health monitoring system and the bridge's global geometry, friction dampers were supplied to be engaged at a work point approximately 1.5m above the deck level.

In addition to the stay cable installation the permanent tie down cable and anchorages at the first piers were also supplied and installed on the back side spans of the flanking towers.

7. MAINTENANCE

The design and maintenance philosophy are in accordance with Eurocodes, which establishes principles and requirements for safety, serviceability and durability.

Maintenance activities are performed during the working life of the bridge, limited allowance is made in design for corrosion of some components and wearing of moving parts such as bearings and movement joints, over time. These components are designed to be replaceable.

The future access to the deck is provided via underslung gantries from abutment to abutment. Internally there will be a shuttle running the entire length of the structure, 2633m, to transport personnel and small hand tools. Demountable cradles will provide external access to the towers and cables.

Figure 27: Cables

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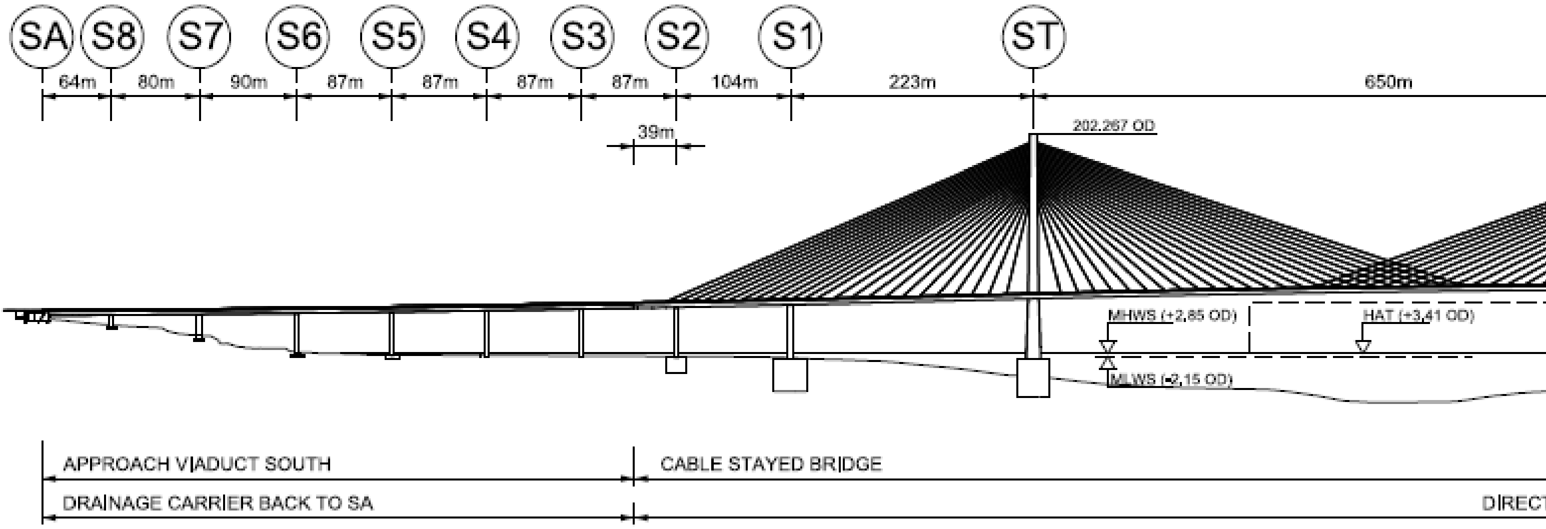

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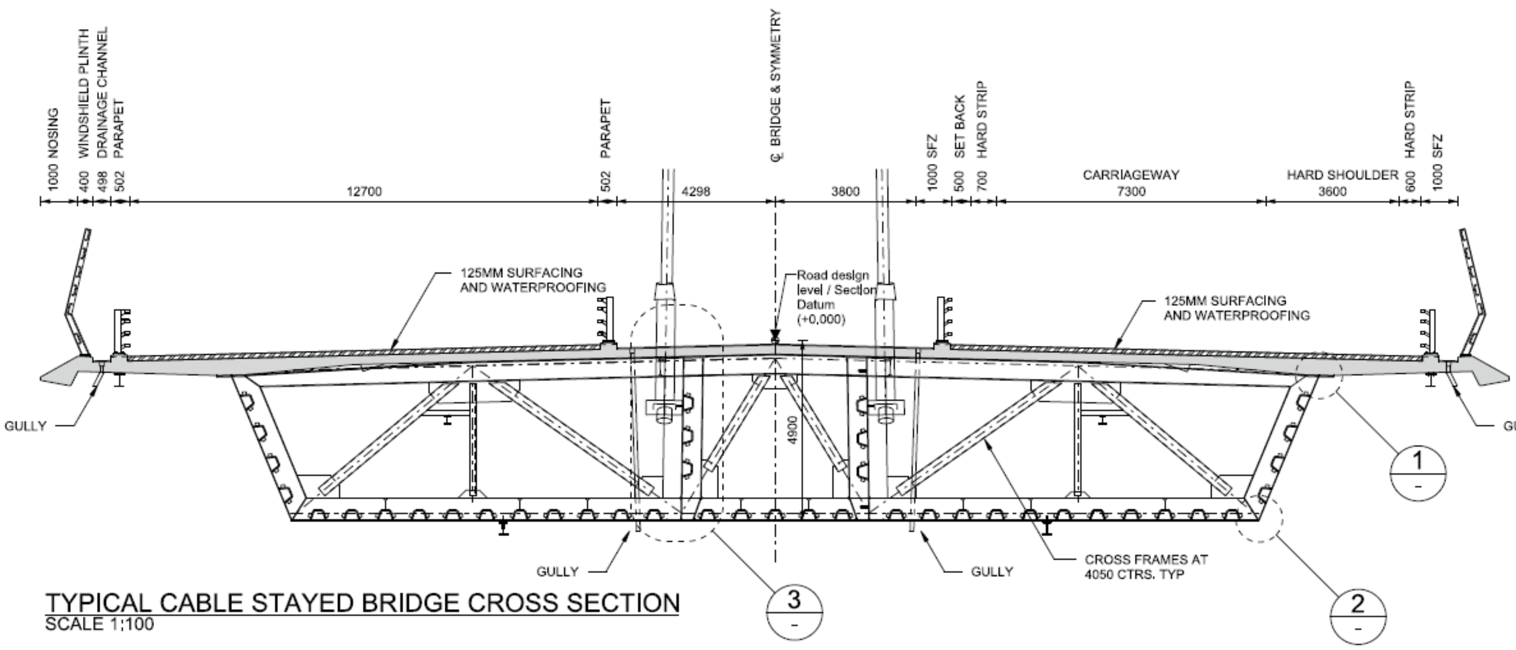

TYPICAL CABLE STAYED BRIDGE CROSS SECTION
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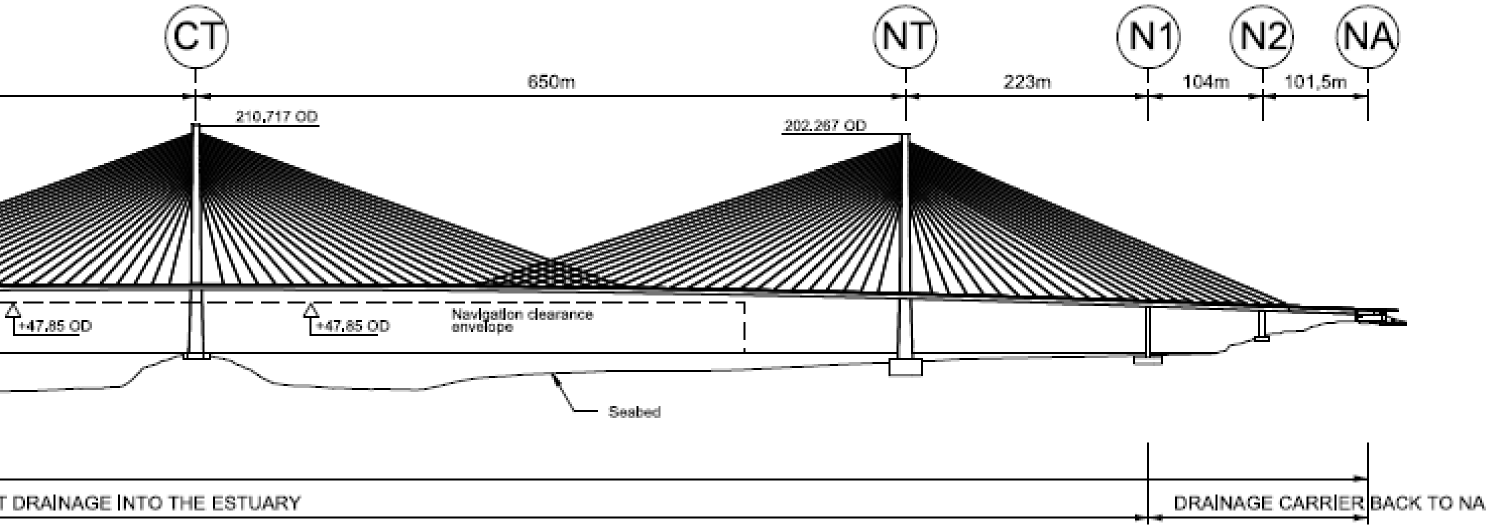
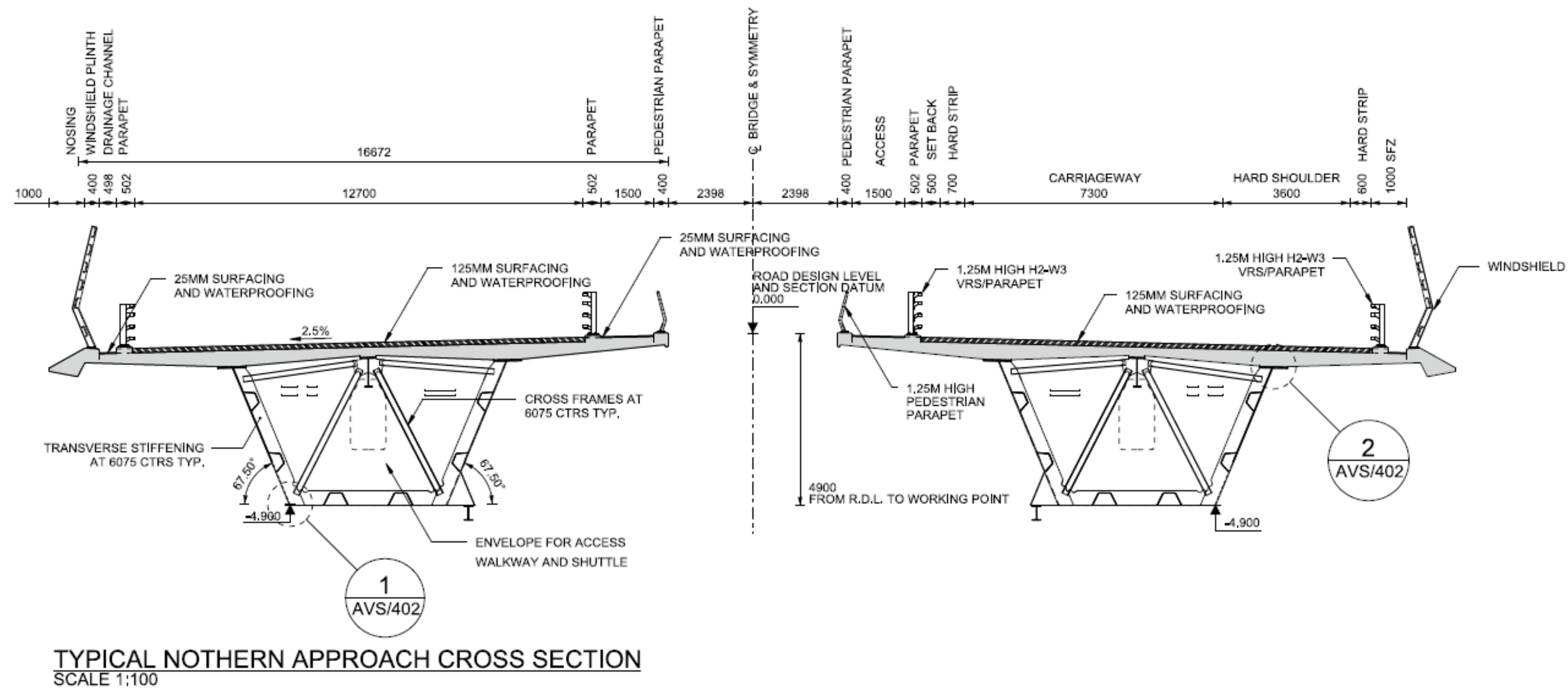
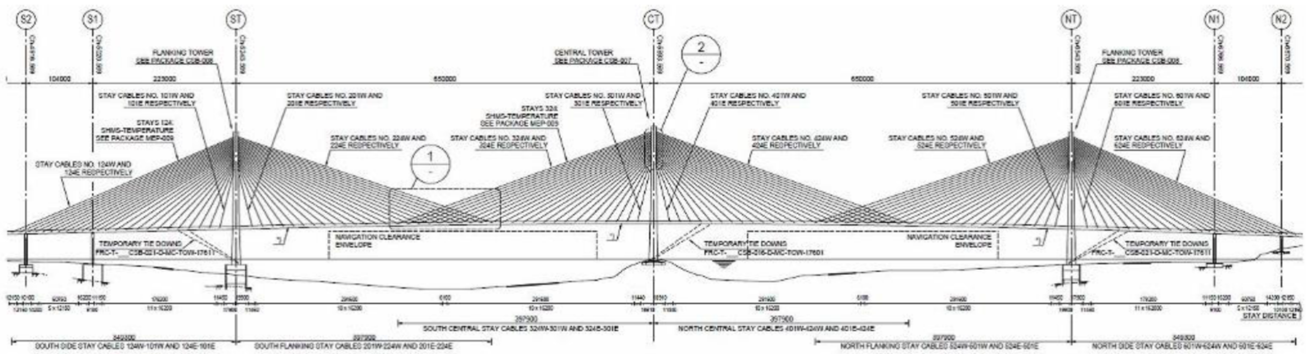




Courtesy of Transport Scotland


S2
S1
ST
CT
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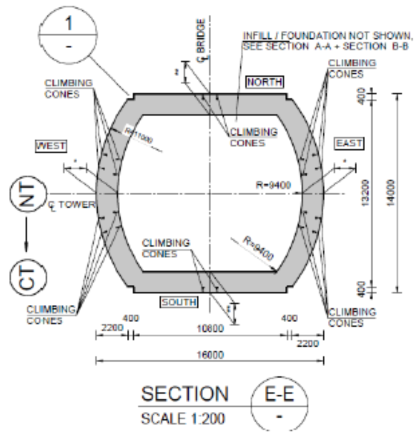
CENTRAL TOWER
NAVIGATION CLEARANCE ENVELOPE
TEMPORARY TIE DOWNS
STAY DISTANCE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CLIMBING CONES
NORTH
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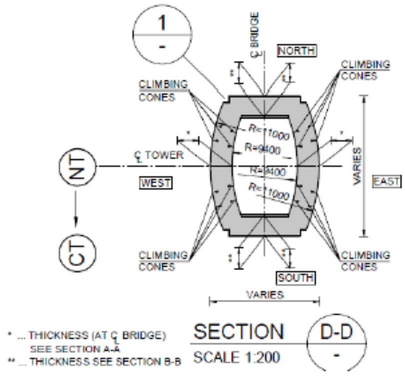
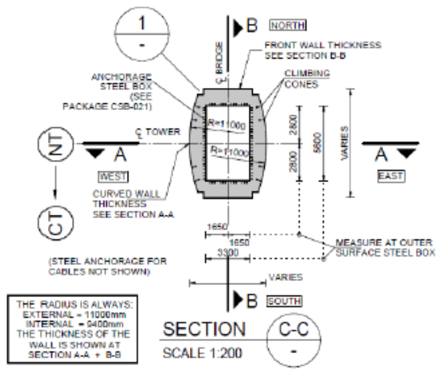
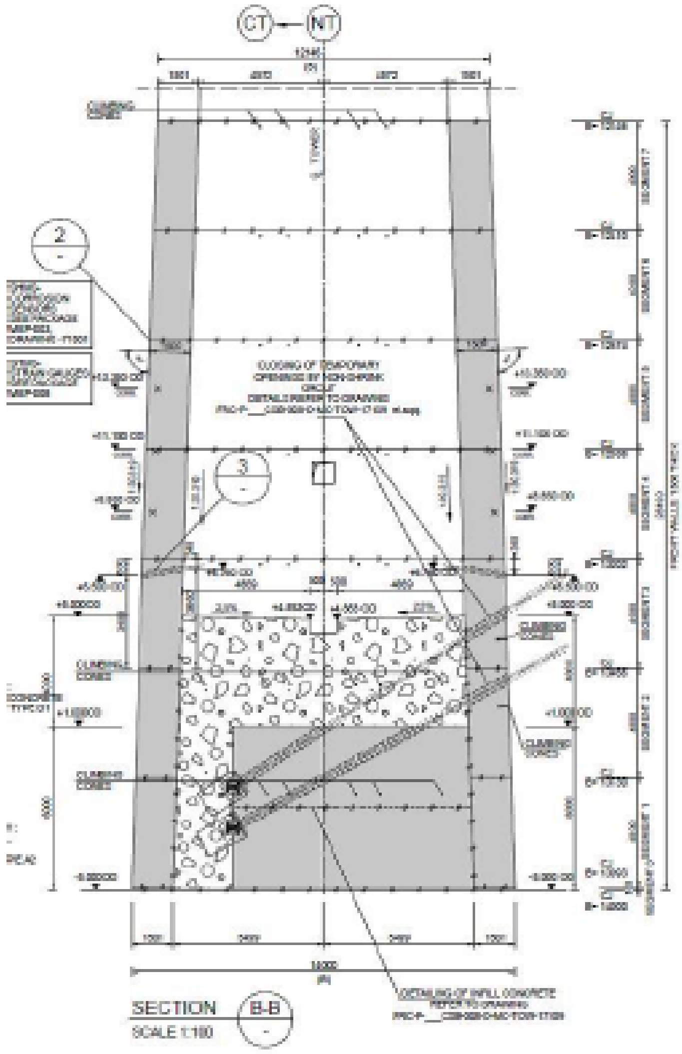
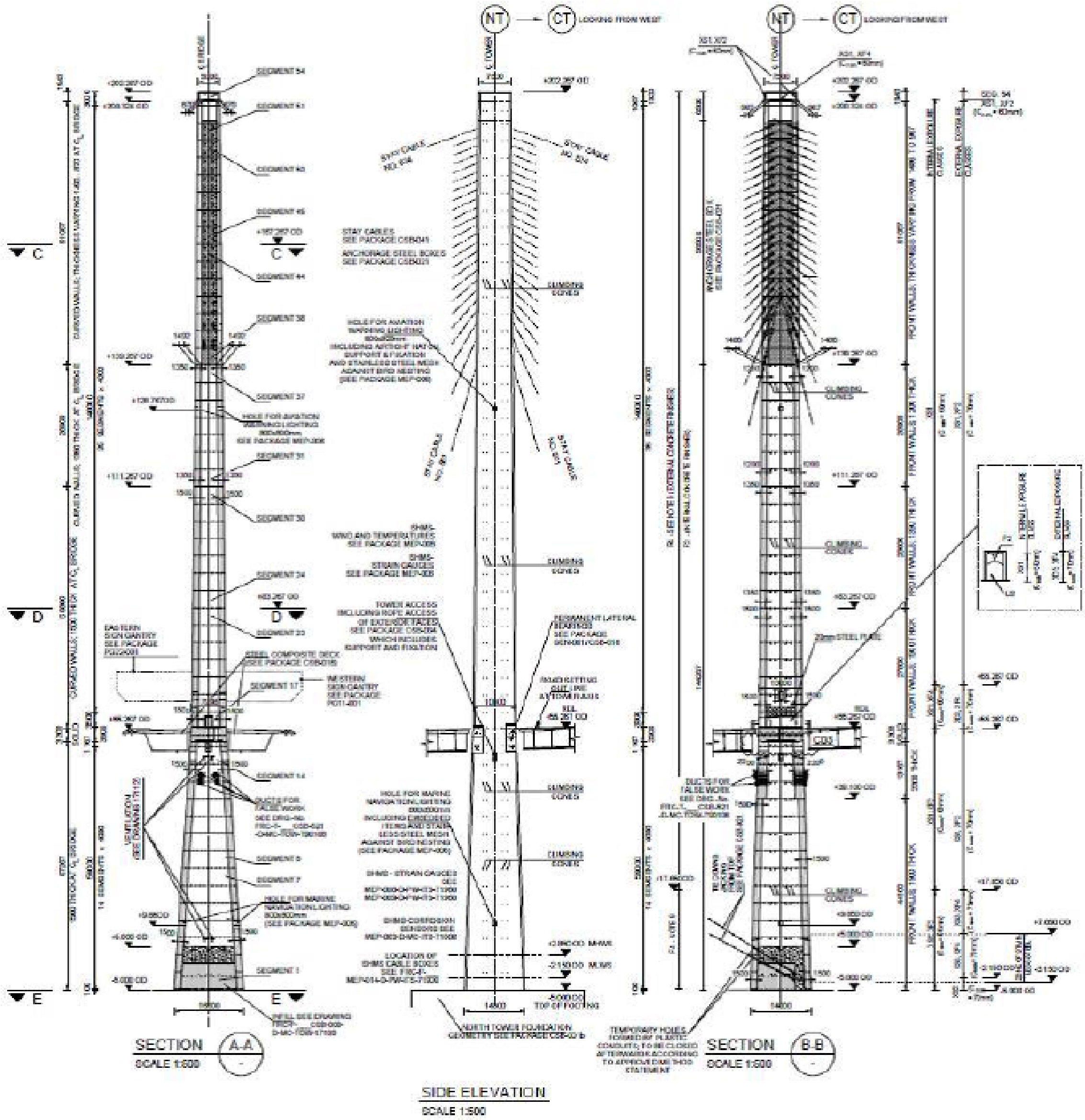
SECTION E-E
SCALE 1:200


CLIMBING CONES
NORTH
WEST
EAST
SOUTH
℄ TOWER
℄ BRIDGE
NT
CT
R=11000
R=9400
VARIES
* ... THICKNESS (AT ℄ BRIDGE) SEE SECTION A-A
** ... THICKNESS SEE SECTION B-B
SECTION D-D
SCALE 1:200

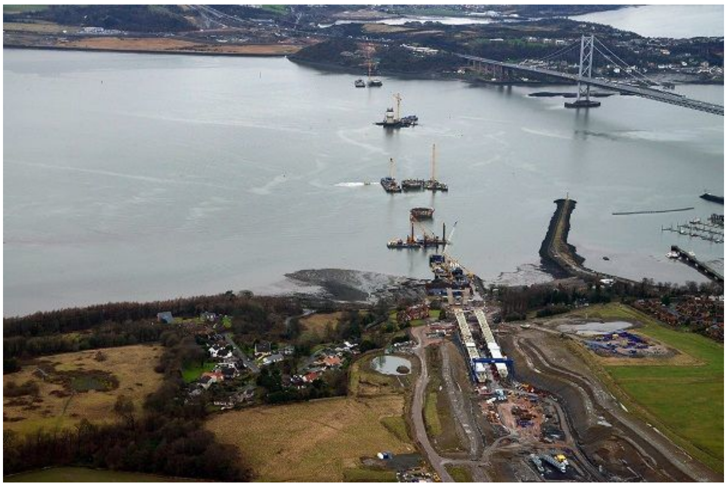

ANCHORAGE STEEL BOX (SEE PACKAGE CSB-021)
FRONT WALL THICKNESS SEE SECTION B-B
CLIMBING CONES
CURVED WALL THICKNESS SEE SECTION A-A
℄ TOWER
℄ BRIDGE
NORTH
WEST
EAST
SOUTH
NT
CT
A
B
R=11000
R=11000
2800
2800
5600
VARIES
1650
1650
3300
MEASURE AT OUTER SURFACE STEEL BOX
(STEEL ANCHORAGE FOR CABLES NOT SHOWN)
THE RADIUS IS ALWAYS: EXTERNAL = 11000mm INTERNAL = 9400mm THE THICKNESS OF THE WALL IS SHOWN AT SECTION A-A + B-B
SECTION C-C
SCALE 1:200

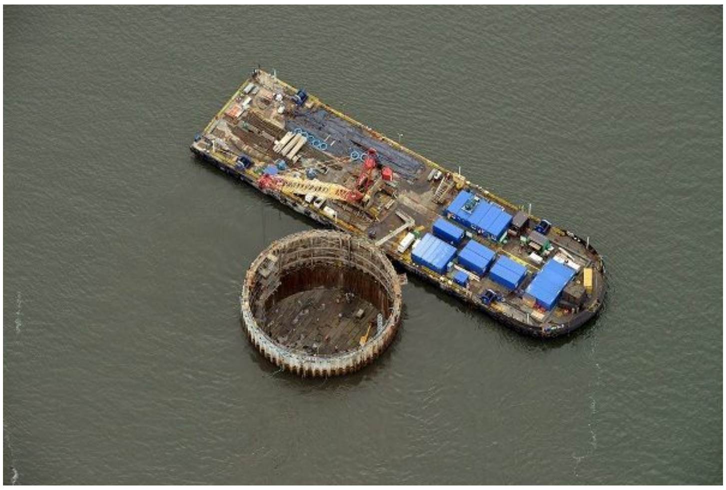

CT
NT
CLIMBING CONES
16000
1500
5400
5400
1500
SECTION B-B
SCALE 1:100

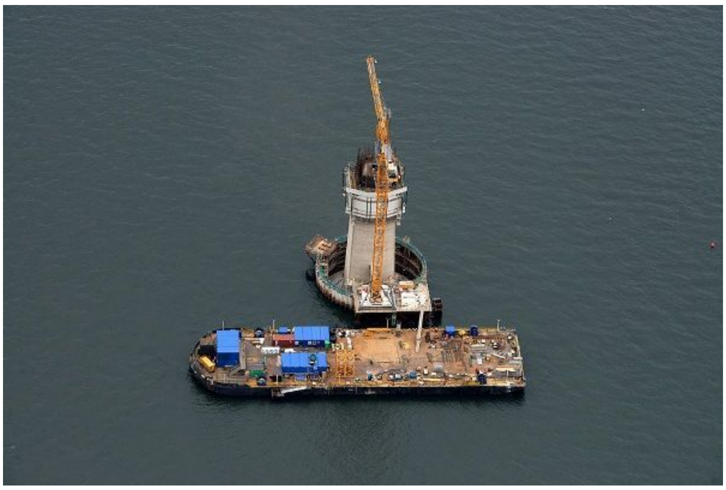


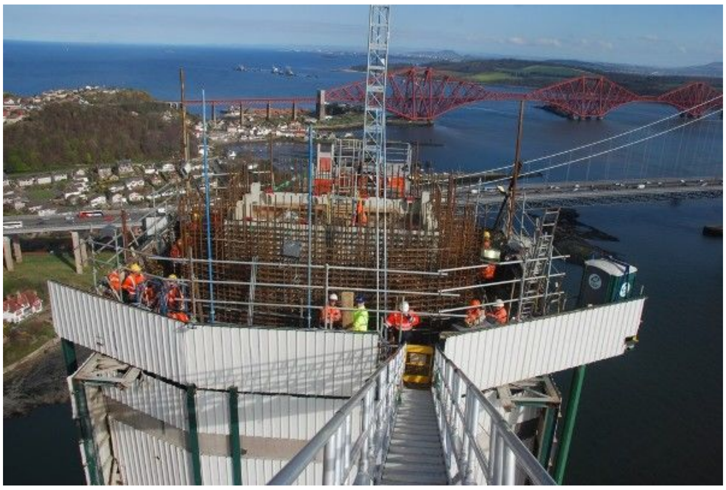
Source: Brunton - Robinson: Tower Construction

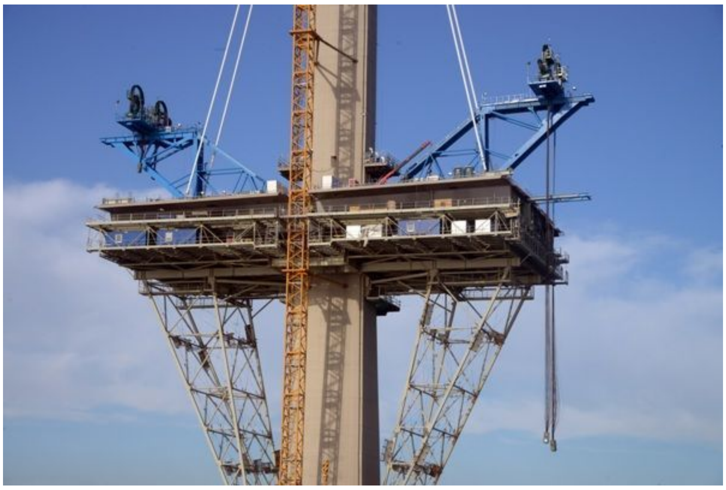
PHOTO GALLERY

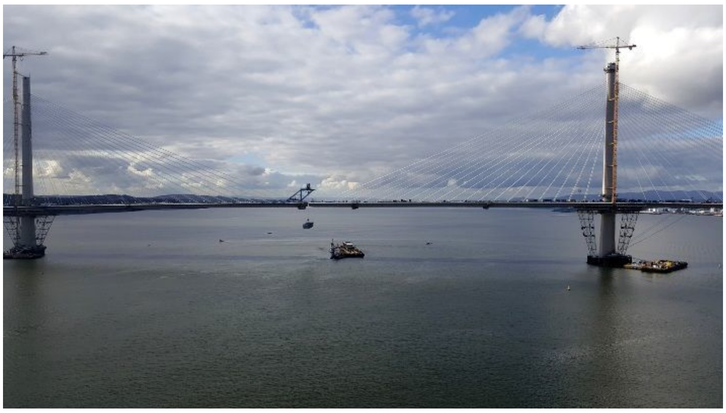


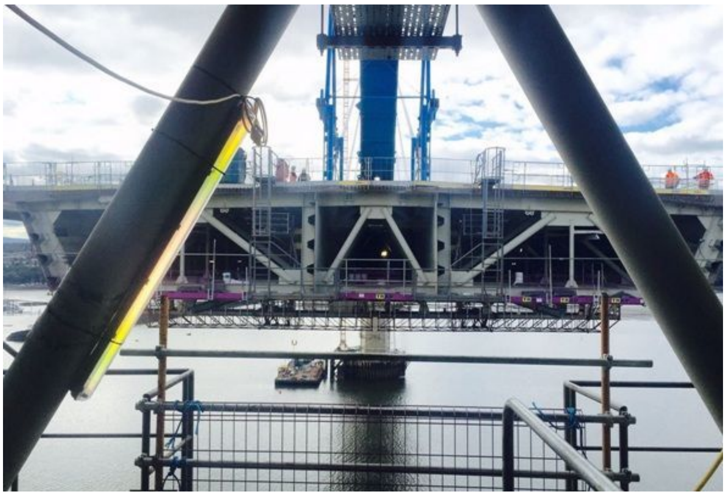


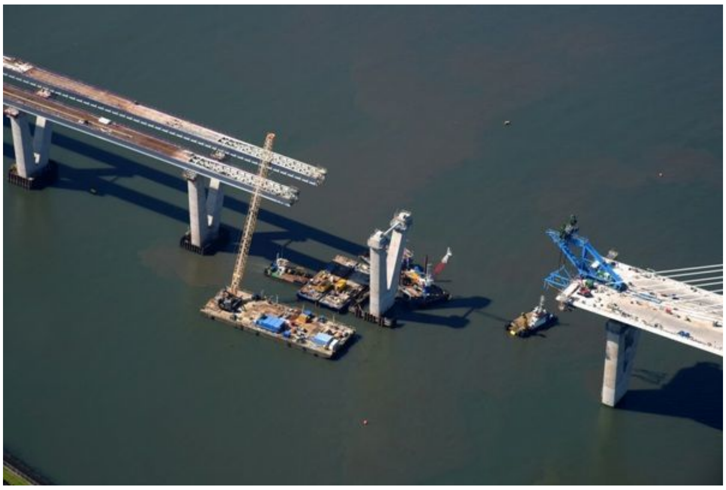






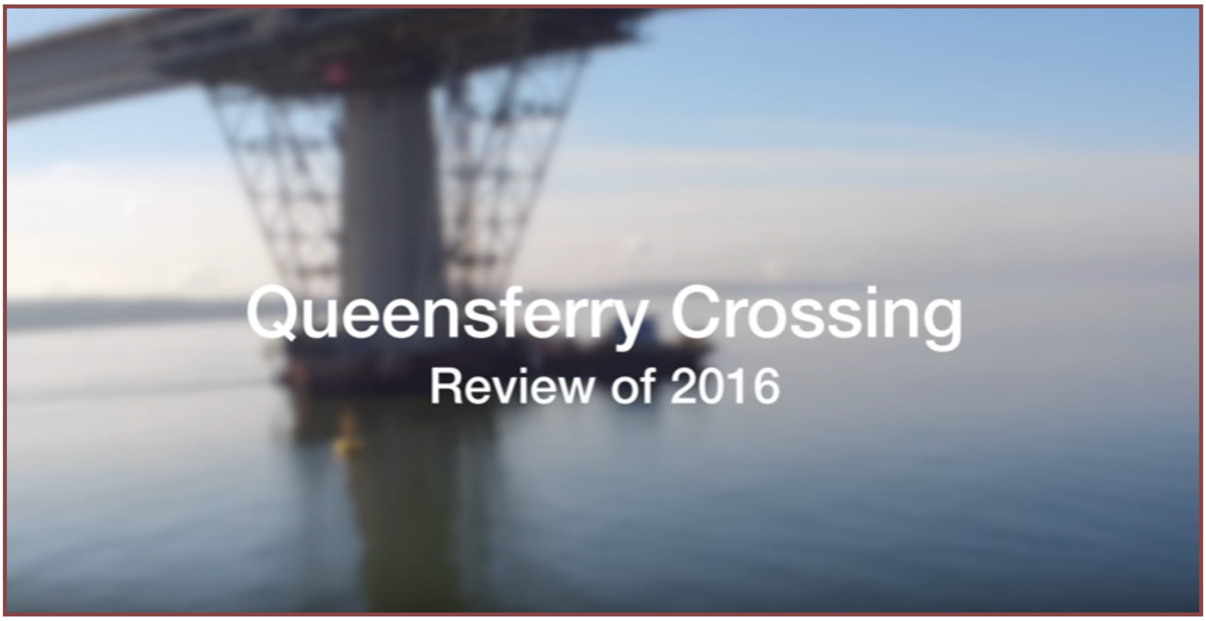






Courtesy of Transport Scotland

VIDEOS



More official videos on Queensferry Crossing:

FORTH ROAD BRIDGE

Magdaléna Sobotková

I. Design and Construction

1. INTRODUCTION

The bridge across the Firth of Forth is a vital link in Scotland´s strategic road network with more than 24 million vehicles every year.

When opened on 4 September 1964, the Forth Road Bridge was the longest span suspension bridge outside the USA and the fourth longest in the world.

The main span is 1006m with equal side spans of 408m. It carries a two lane dual carriageway, without hard shoulders or strips. There is a separate cantilevered footway/cycle track on either side.

The main span has a steel orthotropic deck and the side spans have a heavier concrete deck. The deck has steel stiffening truss, wire rope hangers and vertically split cast steel cable bands.

The main cables are the primary load-carrying members, each carries almost 14 000t of load. They were aerially spun.

Each cable comprises 11 618 galvanized high-tensile steel wires 4.98mm in diameter, giving a compacted cable size of 590mm.

In 2001 the bridge was classed as a Category A listed structure.

2. DESIGN

2.1 Background

A regular ferry service was instituted about the year 1130 and in 1164 it was granted a charter naming the crossing "Passagium Regina".

Throughout the centuries, tunnel crossing and a chain bridge were under consideration. In 1890 a cantilever railway bridge was opened.

In the years following 1920 the idea of a road link was developed. Various locations were considered including the alignment via the Beamer Rock, where now the Queensferry Crossing is located. Investigations, studies and calculations were carried out and various schemes considered, including the addition of a second deck on the Forth Railway Bridge.

Shortly after the war, a scheme for a long span bridge over the River Severn was also considered, and due to its similarity with Forth Crossing some problems common for both bridges were considered together, particularly aerodynamic research including wind tunnel testing.

From 1947 necessary legal and financial issues were resolved and relevant contracts executed. The works of the preliminary contract commenced during the summer of 1958.

The project, including main bridge and approaches, was substantially completed on the 4 September 1964 when it was opened by H. M. Queen Elizabeth II.

2.2 Evolution of Design

Much of the Forth Road Bridge design was based upon work done originally for the Severn Bridge, which was intended to be built in 1946.

Attention was paid to aerodynamic stability which influenced the arrangement and shape of stiffening girders and deck. The footways and road deck were separated.

The most important decision to be made was the method of forming the cable. Strands would be too heavy for such a long span and would require special plant for erection. As a structural member, strand has a lower modulus of elasticity compared to plain wire, and a group of strands cannot be compacted into as dense a mass as the parallel wires spun in situ. Economical, space and labour aspects were also considered and it was decided to adopt the spinning process.

Another important decision to be made in the early stages of design is the ratio of cable sag to length of span. A ratio of 1/11 was adopted, after laborious investigation including the Severn Bridge estimations.

The originally proposed towers were considerably modified in the course of design. The present system for tower legs is that of welded box sections with interconnecting stiffened plates.

Anchorages were originally contemplated as the gravity ones, however, it was decided to drive tunnels, one tunnel for each cable, filled with concrete.

3. DETAILED DESIGN

3.1 Tower

The section of each tower leg is composed of five cells formed by three prefabricated boxes (the outer boxes with variable width) joined together by four connecting longitudinally stiffened plates. The centre box of each leg forms a point of connection for the transverse bracing.

Correct load and bending moment were checked in sections, the base section was then checked in the free standing condition with various loads. The legs were designed as tapered as the direct load decreases towards the top.

3.2 Tower Foundations

The base sections of the legs are embedded 10 ft (3m) deep in the upper part of a reinforced concrete pier.

South pier foundations were made with two rectangular caissons, capped by concrete slab on which the pier was constructed. For the north pier a similar slab was made.

3.3 Cables

Attention was paid to the choice of wire diameter, especially to economical and practical requirements and to tensile strength of the wire. It was decided to keep to the standard size of 0.196 in (4.98mm).

It was decided to use 37 strands of 304 (19 inner strands) to 324, 326, or 328 (outer 18) wires each, arranged in a hexagon. It was necessary to estimate the final diameter of the cable so that the cable bands could be machined. Percentage of voids was set at 20 % based on American experience. The cable was finally wrapped between the cable bands with steel galvanized wire.

3.4 Saddles

The tower saddle was fabricated in one piece able to keep 32-ton capacity of the erection crane. The cable groove is a single steel casting machined inside and out on the rotary planing machine and they are stepped to suit the vertical hexagon arrangement of the strands.

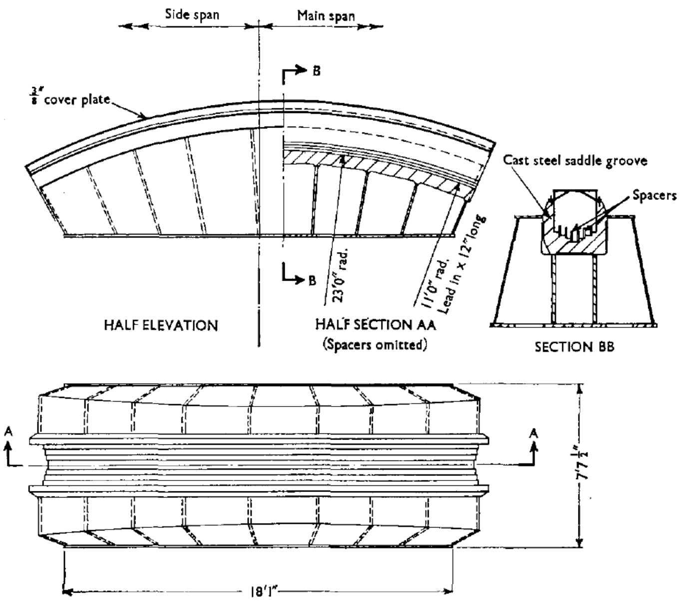


Figure 1: Main Tower Saddle Details

Side tower saddles are mounted on a 13 ft (4m) rocker that allows longitudinal movements of the cable and the force due to the inclination from the vertical is very small. Changing stress in the cable which passes over the saddle is absorbed by friction between the wires and the metal sprayed surfaces of the saddle grooves.

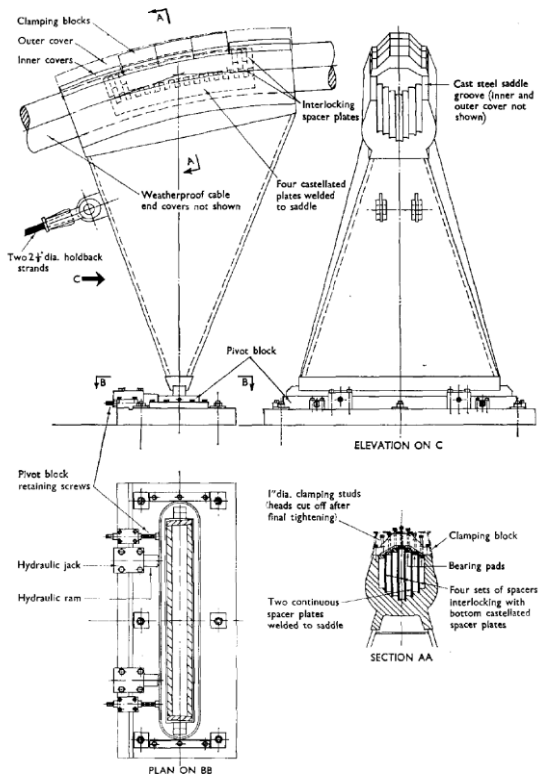


Figure 2: Side Tower Saddle Details

Splay saddles are mounted on rockers. The cable groove is a steel casting. Vertical component of the deflexion is maintained to the rear; the strands do not deflect horizontally without a vertical force to hold them down.

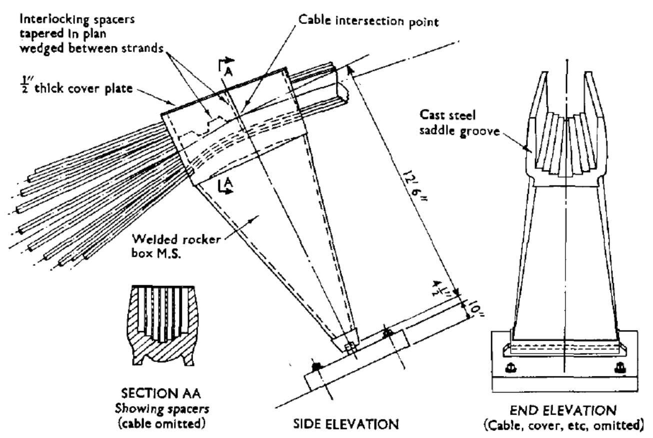


Figure 3: Splay Saddle Details

3.5 Hangers

The cable bands have two grooves on the upper surface for a pair of hanger ropes. The connection to the top chord of the stiffening truss is by a common socket. Where the rope enters the socket, a split cap and rubber gasket is fitted to exclude moisture.

3.6 Suspended Structure

It is an open rectangular framework 78 ft (23.78m) wide and 27 ft 6 in (8.38m) deep braced on all four sides. Its stiffness is designed for torsion (especially for aerodynamic stability) and for vertical and horizontal bending. In the side spans most of the lateral wind force is carried by the suspended structure, but on the main span 60% of the total wind force is transferred to the cables by the lateral inclination of the hangers.

3.7 Side Towers

The two 150-ft high (45.7m) high reinforced concrete side towers carry the rocker saddles over which the cables are deflected from the side span down to the anchorages.

3.8 Design Standards

The live loading adopted was as specified in British Standard 153 Part 3 : 1954. The bridge was designed for a maximum wind speed of 110 mile/h (177 km/h) at deck level.

3.9 Structural Analysis and Calculations

For details, please refer to the original paper (1965 FRB). However, please note that "*It is impossible to record within the compass of this Paper all the calculations leading up to or substantiating the design*" *(2.102, p. 57, 1965 FRB).*

3.10 Tests on Model Members

Seventeen types of model member were tested to destruction as pin-ended struts.

4. CONSTRUCTION

4.1 Conditions

The Forth Road Bridge is situated much further north than any major suspension bridge in the world at the the time. The site is exposed to south-westerly and easterly winds, and speeds of 70 mile/h (113 km/h) during gale conditions are common. The maximum wind speed recorded on the Forth Railway Bridge anemograph during the construction period was 103 mile/h (166 km/h).

As the free standing towers oscillated under the winds during erection, a damping device was devised and installed.

Cable spinning was affected by strong winds as well causing time loss, oscillation and vertical motions and even some damage to the cable strands and bridge equipment.

To check the aerodynamic stability of the suspended steelwork in the main span during erection a series of tests were carried out in the wind tunnel on a model with a scale of 1:48.

4.2 Side Towers

The side towers comprise two reinforced concrete legs and a cross beam, all of box section, with arch ribs stiffening the corners of the structure. The cable saddles, each of which imparts a maximum vertical reaction of 3500 tons, are located on thick capping slabs at the tops of each leg.

The south side tower was originally just off shore, but land has susbsequently be reclaimed so that it now sits on the shore line which is formed by shale and sandstone, at the pier the sandstone is overlaid by 50 ft (15m) of hard boulder clay, 8 ft (2.4m) of gravel and 13 ft (7m) of sand and silt. The foundations were constructed in steel sheet pile cofferdams. The two cofferdams were built consecutively using the same sheet piles and waling frames.

The north side tower is on the shore and excavation in the whinstone rock was carried out by normal blasting methods. A bund wall of rock fill, sand bagging and sheet piling was used to keep tidal water out of the excavations. Foundation blocks are similar to those on the south side.

The walls of the tower legs are generally 2 ft (61cm) thick and were concreted in 6 ft 8 in (2m) lifts. Horizontal diaphragms are at every fourth lift. After completion of the concreting they were post-tensioned.

4.3.Anchorages

Each anchorage had to resist a pull of 28 000 tons (14 000 tons imposed by each main cable), acting at an angle of 30" to the horizontal. The anchorages are tapered concrete blocks, prestressed longitudinally, which fill tunnels driven into the rock. During the final stages of the tunnel concreting, work was commenced on the construction of the reinforced concrete anchorage chambers and splay saddle foundations. The external surfaces of these chambers were tanked with asphalt to ensure watertightness.

At the upper face of each anchorage block there are 19 steel crosshead slabs with the bolts retaining the main cable strand shoes. Each crosshead slab is anchored by six cables consisting of four 19-wire galvanized strands of parallel lay, anchored in a common socket. The cables pass through ducts to the lower end of the concrete blocks where they are stressed. By prestressing the anchorage cables, movement of the steelwork at the anchorage face due to changing loading conditions is eliminated. After stressing, the cables were grouted and the chamber and anchorages backfilled.

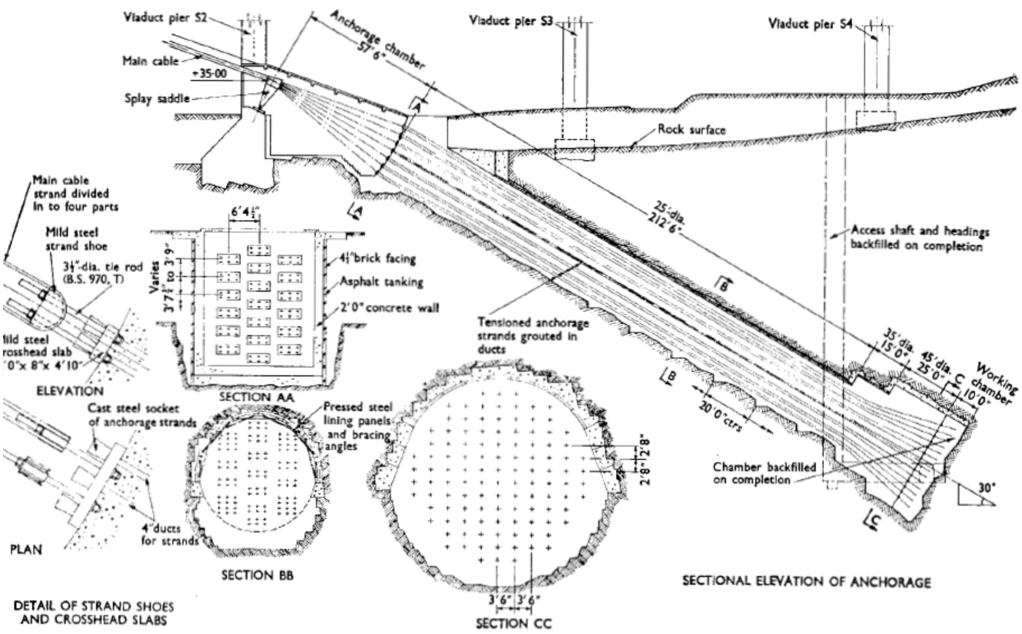


Figure 4: General Sections of South Anchorage Tunnel

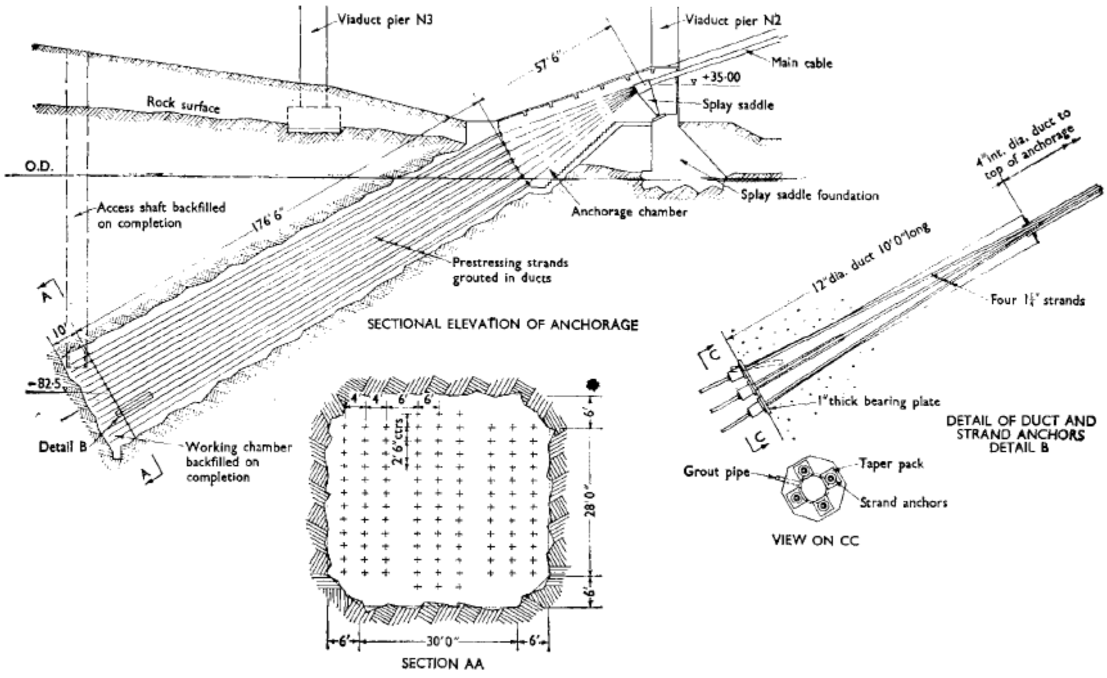


Figure 5: Section and Detail of North Anchorage

4.4 South Pier

Foundation works started with creation of a cofferdam made from sheet piles. After removal of silt and cleaning of the river bed, it was dewatered. Caissons formed of heavy steel were erected. Caisson walls were filled with reinforced concrete. Both caissons were sunk together, in free air. At foundation level both caissons were plumb. The working chambers were filled with concrete, annular space with cement grout and the space between air deck level and -30 O. D. with rock fill and lean concrete.

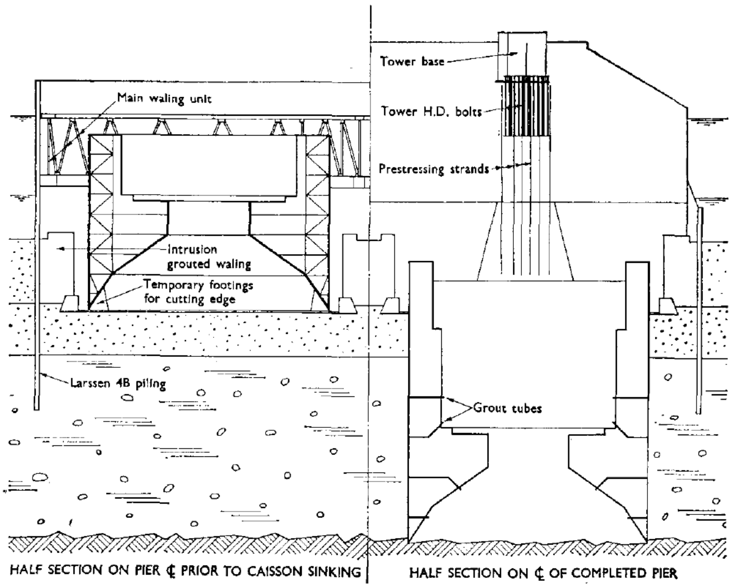


Figure 6: Section of South Pier

The pier takes the form of a 40 ft (12.2m) wide block with cutwater ends having a total length of 157 ft 6 in (48m). The sides have sloping surfaces and plinth enclose the tower bases at the upper level. Each tower base is held down by 28 bolts on both its north and south faces.

4.5 North Pier

It is founded on the Mackintosh Rock. Cofferdams were of steel sheet piles, with tremie concrete waling. The tower base arrangements are identical to those for the south pier only with different prestresing cable details.

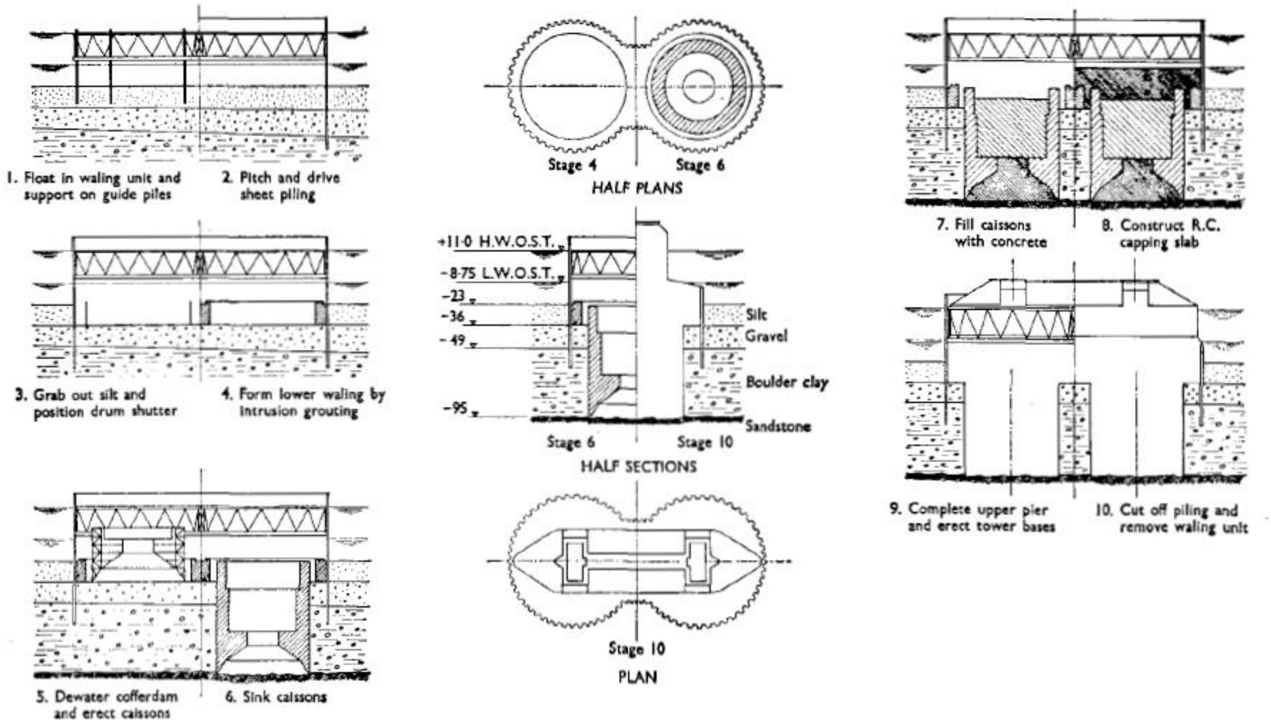


Figure 7: South Pier Construction Sequence

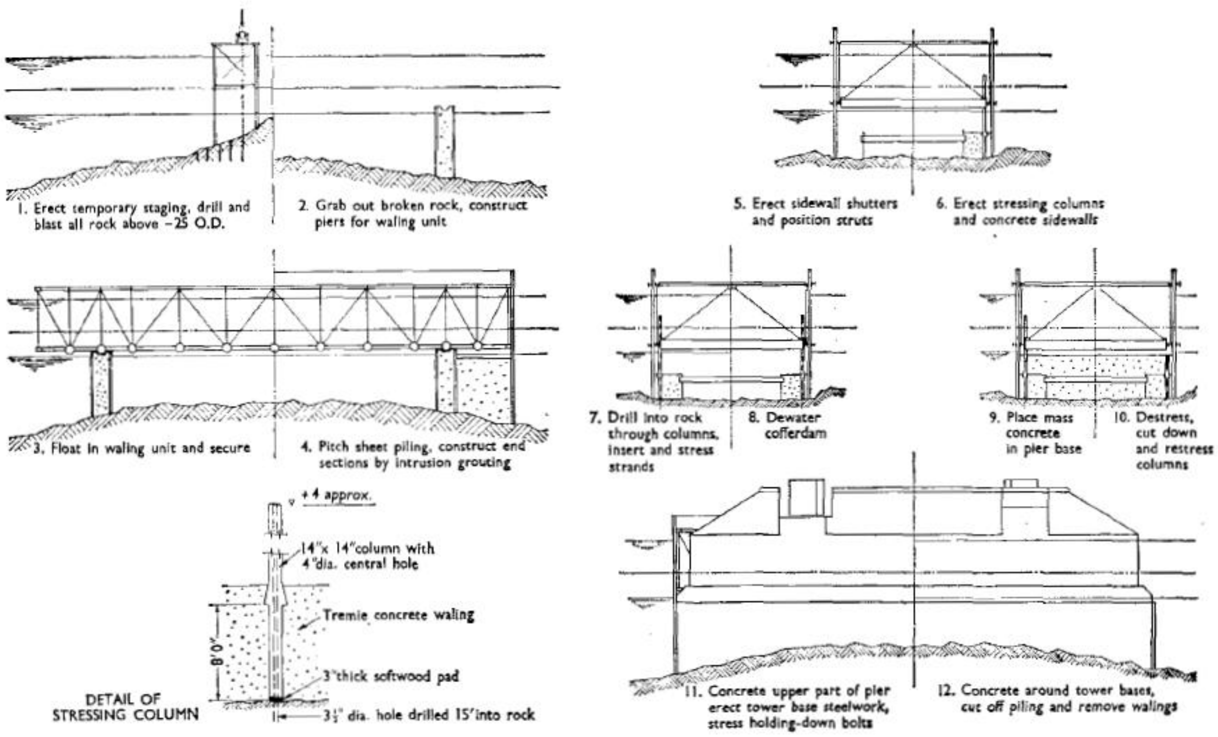


Figure 8: North Pier Construction Sequence

4.6 Approach Viaducts

The north and south viaducts support the carriageways, footpaths, and cycle tracks. The south viaduct of eleven spans has a total length of 1420 ft (433m) and the north viaduct of six spans has a total length of 820 ft (250m). Reinforced concrete piers support the deck structure in spans varying from 110 ft to 16O ft (33.5m to 51.5m). The deck structure is continuous and consists of two steel box beams with transverse beams and outriggers having a reinforced concrete slab working in composite action with the steelwork. The piers consist of two vertical shafts of T-section with the tapering legs of the Ts joining at the top of the pier over an arch of 17 ft (5.2m) radius.

The first four spans of the south viaduct were erected by mobile cranes operating at ground level. The S7-S6 sections were lifted by an erection mast. The main beams were cantilevered forward from pier S7. Span S6-S5 was erected similarly, lifted by mobile cranes.

Erection of the North viaduct started the same way as the south, using falsework supports for the first two spans. After that a 20-ton capacity crane was built on the main beams. Its wheels moved on the top flanges. All remaining steelwork was erected with it by cantilevering forward from each pier in turn. The continuous slab was concreted between expansion joints. It is bonded to the steelwork by T-section shear connectors welded on top flanges.

4.7 Superstructure

4.7.1 Towers

Access to the piers to both towers was provided by rock-filled causeways from the land. For the bases, shop welded boxes were assembled, connected by high strength bolts, and concreted. All except the bases and lowest sections of each tower were erected by means of a climbing structure.

As each box section was landed it was immediately connected to the one below by high tensile steel tie rods and 3 in (7.6cm) dia. high strength grip bolts. The three boxes forming each tower leg were interconnected by means of four vertical cover plates secured with 4 in (10.2cm) dia. close tolerance bolts. Internal diaphragms, access ladders and hoist supports were assembled and the diagonal bracing or cross girders erected and grip bolted. Weather hoods were put on the cross bracing connexions and final painting of bolt heads, butt joints, etc, completed.

When the climbing structure reached its eleventh and final position, the upper cross girder of the tower, which up to then had formed part of the climbing structure, was unshipped and re-erected in its permanent position.

The working platform with two special 10-ton derricks, which were required for assembly of catwalks and cable spinning, were erected. These cranes then dismantled the climbing structure and its 32-ton derrick.

4.7.2 Supply and Erection of Cables

The two main cables of the bridge were each specified to be made of 11 618 galvanized hard drawn wires, erected by the process of cable spinning. It is a highly complex process which involves the use of much specialized plant, such as reeling and unreeling machines, tramway drives, compacting and wrapping machines and also expert knowledge. At that time it had never been used outside the United States so the Contractors sought advice on all details there.

7 450 tons of galvanized high tensile steel wire 0,196 in (4.98mm) in dia. were delivered by rail to the site. 1 500 tons (i. e. about 3 300 coils) were stored directly at site, the remaining 6 000 tons were stored under cover in Middlesbrough.

Over the top of the side towers and main towers temporary footbridges / catwalks were built. They enabled men to adjust the wires and subsequently to compact and wrap them.

Loops of wire were carried over the span on a grooved spinning wheel attached to a tramway drive. The wires were unreeled by eight electrically powered unreeling machines - they help reduce the tension in the wire. Two wheels shuttled to and fro across the river, each trip laying ten miles of wire. The adjustments of individual wires were carried out during spinning. The wires of the last trip completed a strand of 314 wires (average number).

"All the cable spinning was done by men working on two eight-hour shifts from 8:00 a. m. to 4:00 p. m. and from 4:00 p. m. to midnight. In order to permit uninterrupted spinning there were no stops for tea or meal breaks. Tea and hot soup were delivered periodically by means of an urn strapped on the back of a tea boy and meals were eaten between the passing of the wheels. In spite of their exposed position, high over the Forth on bitter, cold nights, little if any shelter could be provided for the cable spinners on account of the necessity for keeping obstructions to wind down to a minimum". (4.52, p. 463, 1965 FRB)

After the first strand had been spun of each cable, the remaining thirty-six strands were spun in nine groups of four - the work being carried out alternately on east and west cables.

When completed, the group of 37 strands in each cable hung in hexagonal shape. All the wires were squeezed throughout their lengths into a compact and approximately circular formation by four purpose-built compacting machines which had hexagonal frames hinged at the tops.

192 cable bands which consisted of two semi-cylindrical steel castings, machined inside to fit the cable, were clamped together at top and bottom, each band by four to eight high tensile steel bolts or screwed rods with washers and nuts on each end.

The last major operation on the cables was wrapping them round from end to end between the cable bands

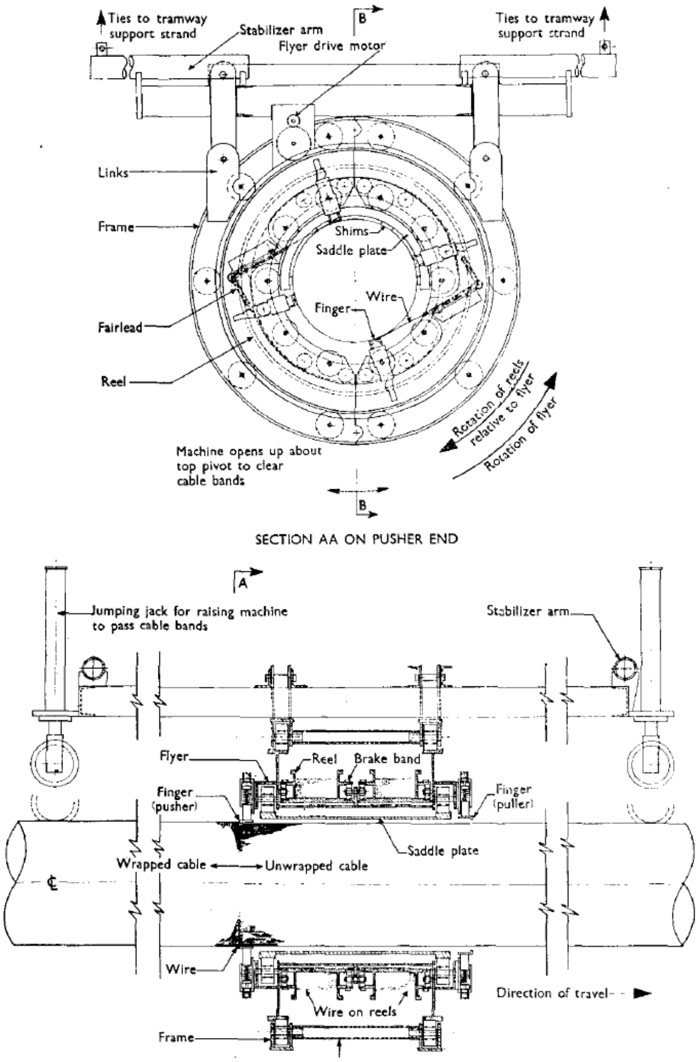


Figure 9: Cable wrapping machine

with galvanized wires under tension. This was done by means of two large wrapping machines which weighed four tons each and were packed off the top of the cable.

"Each machine consisted essentially of two tandem drums on which the wrapping wire was wound, and from which it was pulled off and wrapped round the cable by means of a power-driven revolving flyer. The drums, which encircled the cable, were split at the bottom and hinged at the top to enable them to travel past the cable bands. The machines could be moved up or down the cable by means of their own hauling winches and were capable of wrapping while travelling up or down hill." (4.76, p. 471, 1965 FRB).

The suspenders were each made up of steel wire ropes, which were looped over grooves at the top of the cable bands, and terminated in common sockets at the lower ends. The longest suspenders weighed four tons and were over 300 ft (91.4m) long.

4.7.3 The Suspended Structure

The steelwork of the suspended structure was erected in two phases.

In the first pass four 15-ton derricks, mounted on temporary crane girders in the deck, erected each panel of steelwork in front of them, working out from either side of the span.

The first 90 ft (27.4m) length of deck was cantilevered out from each side of each tower and the derricks assembled on them before the end of cable spinning. When each panel of steelwork had been assembled and connected to the suspenders, the temporary crane tracks were extended, and the derrick moved out to the end of the panel in order to erect the next one.

The work continued in this way until 18 panels had been erected on either side of each tower. Then work on the two fronts in the main span had to be halted until the remaining four panels had been built in each side span and connected to their bearings at the side towers.

There was little difference in the levels of steelwork each side so it was decided to move both cranes forward and to erect the last top chord on the east side with the south derrick and on the west side with the north derrick and by this the bridge gap was closed at mid-span.

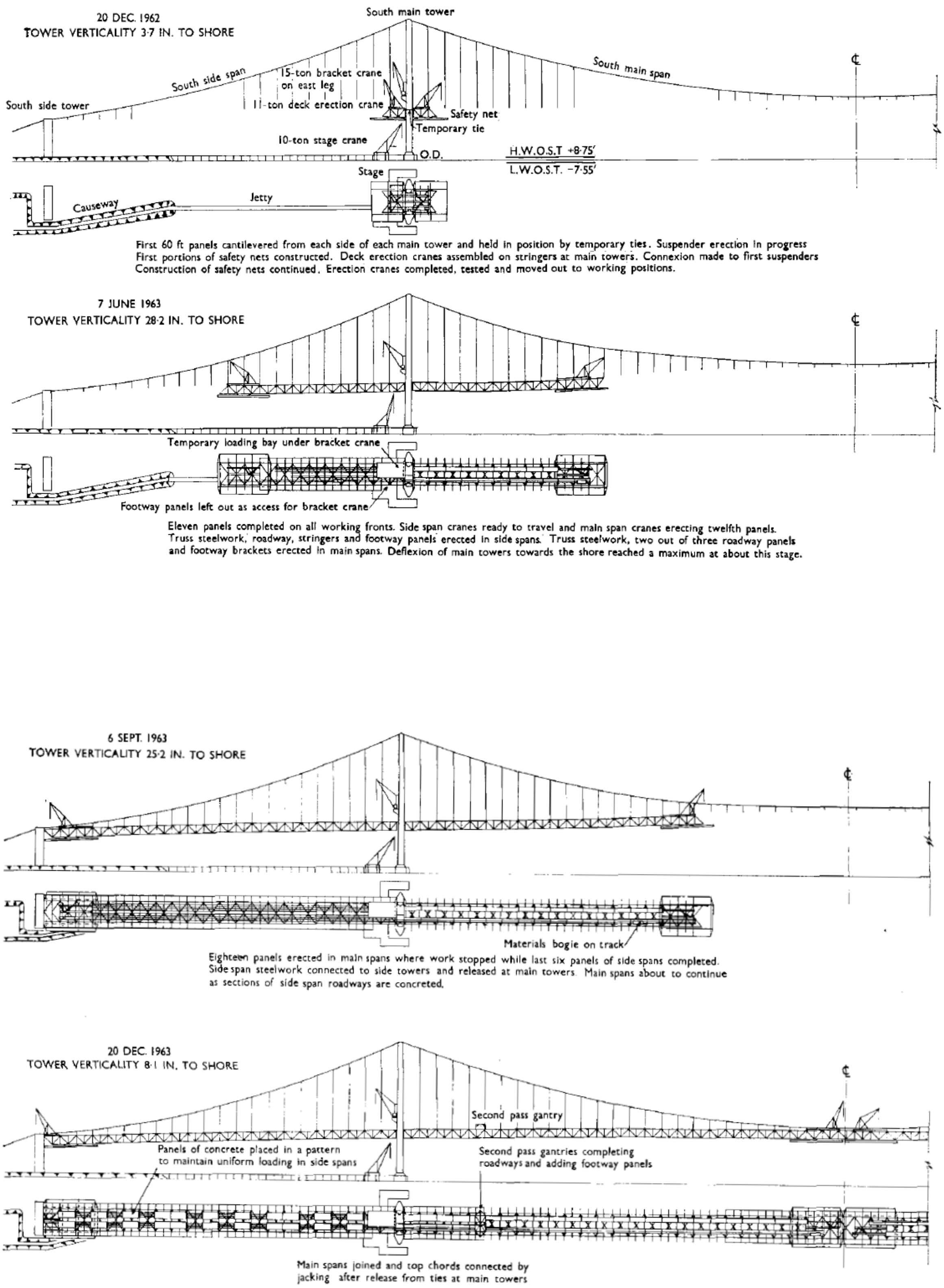


Figure 10: Sequence of deck erection

As soon as the side span steelwork was completed, concreting of the roadway on them was begun.

All the site connections in the suspended structure, with the exception of the joints between the battle deck panels and connections of crash barriers and parapets, were made with high-strength friction-grip bolts, tightened by impact wrenches. The other connections were site welded.

For works on erection of steelwork structure it was decided to use four safety net assemblies. Each assembly fully covered the length and width of three panels. Experience had shown that men work more quickly and freely on structural steelwork if safety nets are provided beneath them.

In the second pass the suspended structure was completed. Two erection gantries, intended to work out from the towers towards mid-span, were designed at site. Thus it was not necessary to wait for the 15-ton derricks to become free and be turned round.

The gantries were assembled on the second panel out from each tower. They completed the erection of the roadway battle decks, the cantilevered footway/cycle tracks, and the crash barriers, panel by panel as they moved towards mid-span.

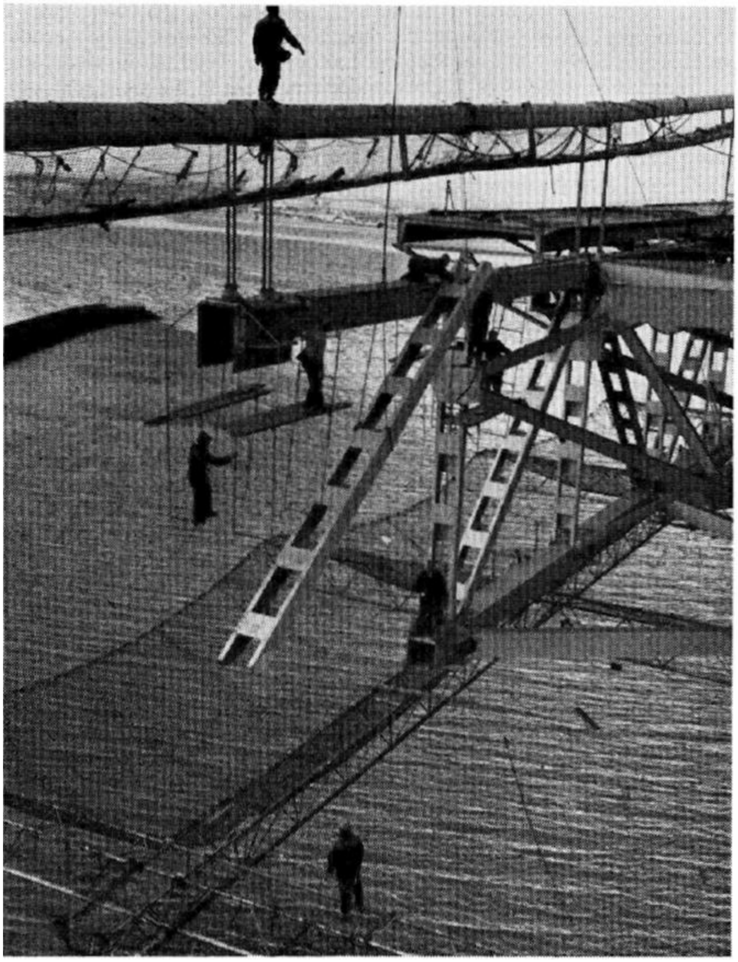


Figure 11: Erection of suspended structure in progress, with safety net

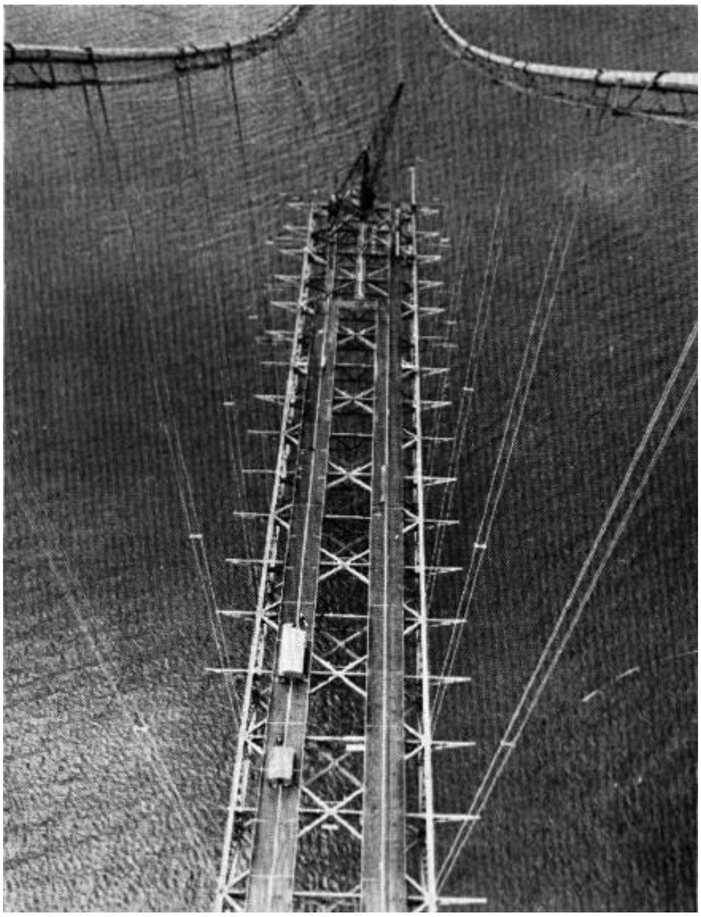
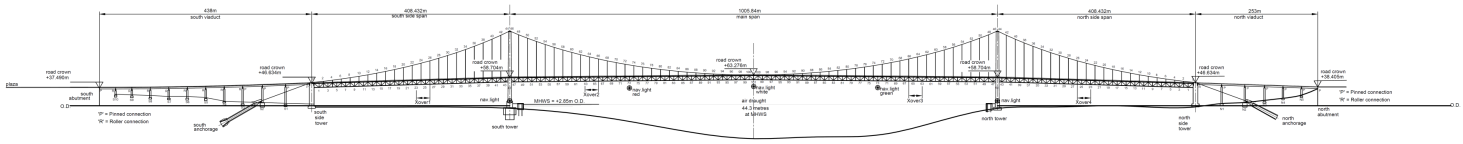
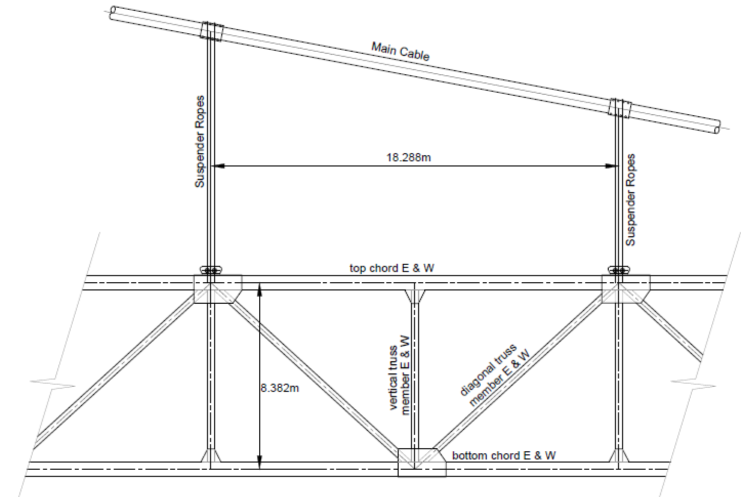


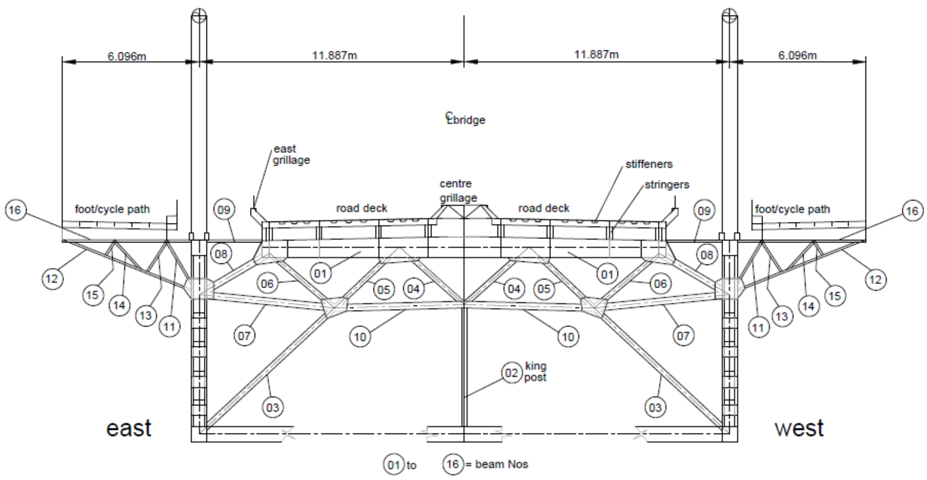
Figure 12: Erection of Steelwork in main span



elevation showing panel points scale
1:4000

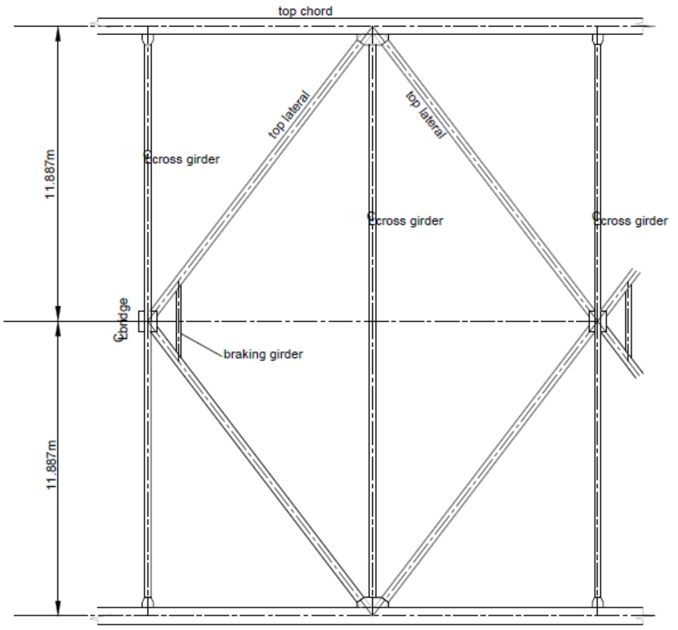


elevation at suspended span deck
scale 1:200

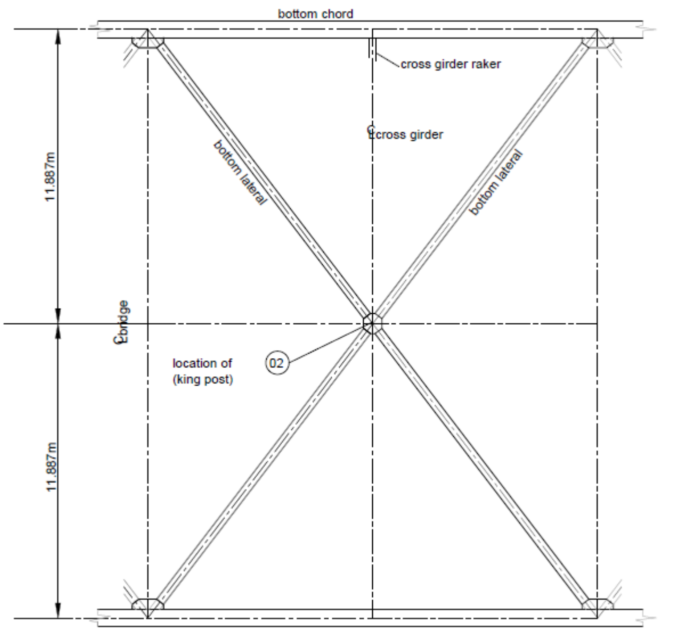


cross girder arrangement
even Nos have king posts

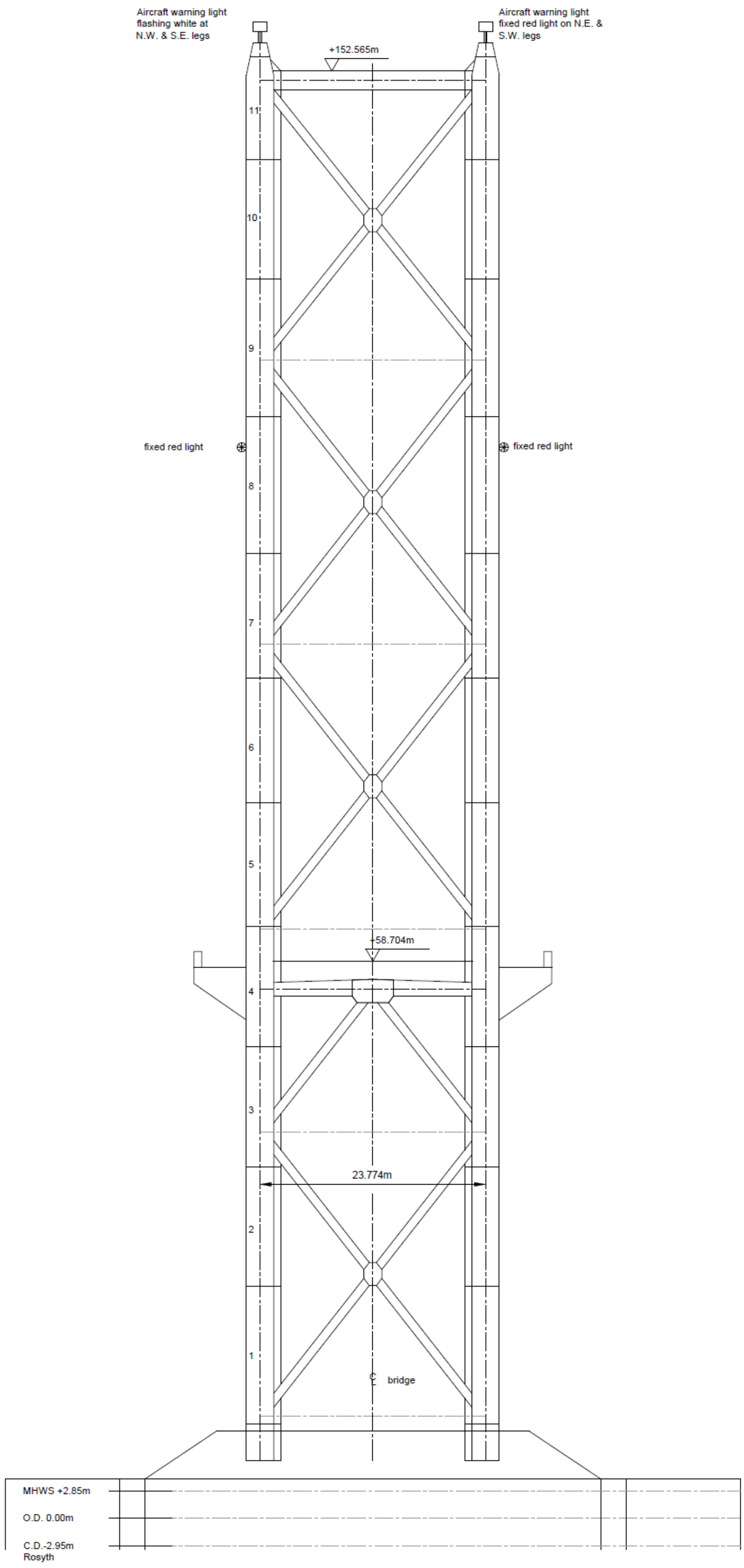
scale 1:200



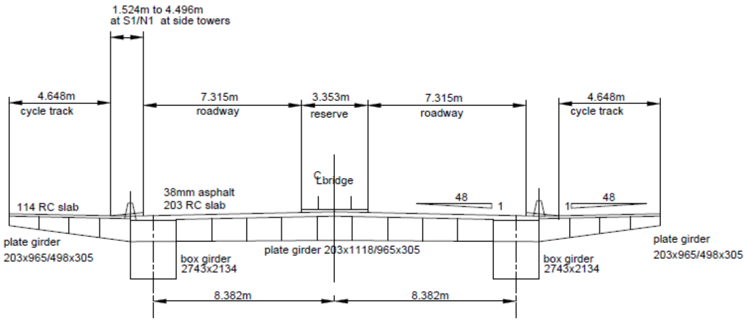
plan on top laterals scale 1:200



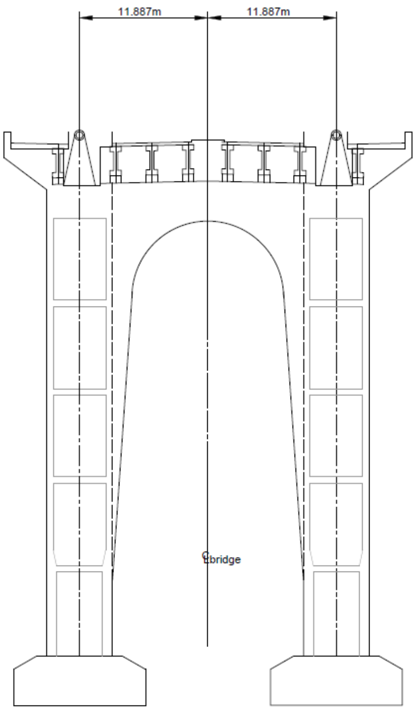
plan on bottom laterals scale 1:200



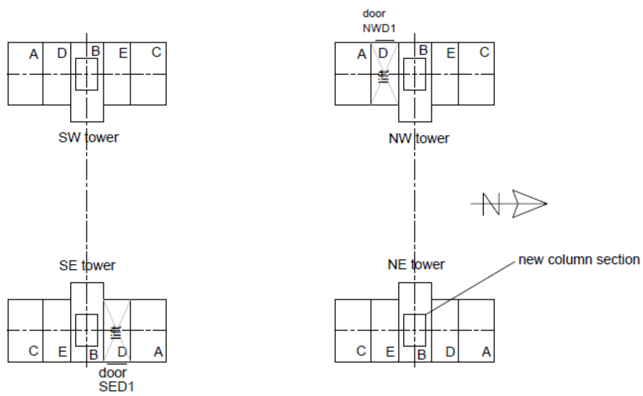
SECTION AT MAIN TOWER
Scale 1:400



typical viaduct section scale
1:200

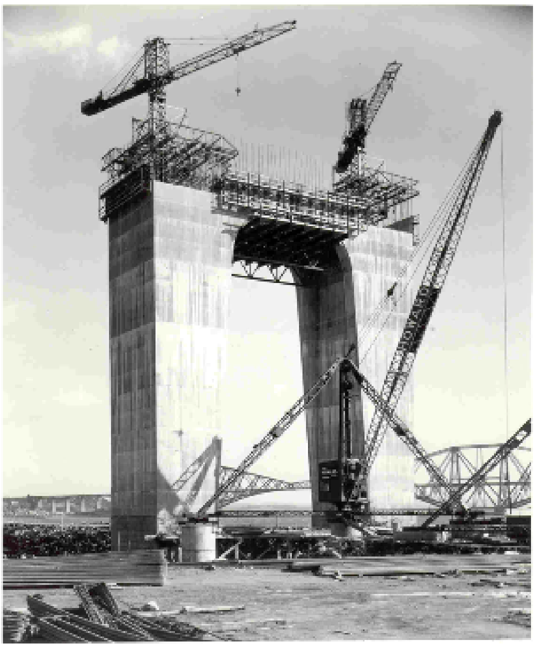


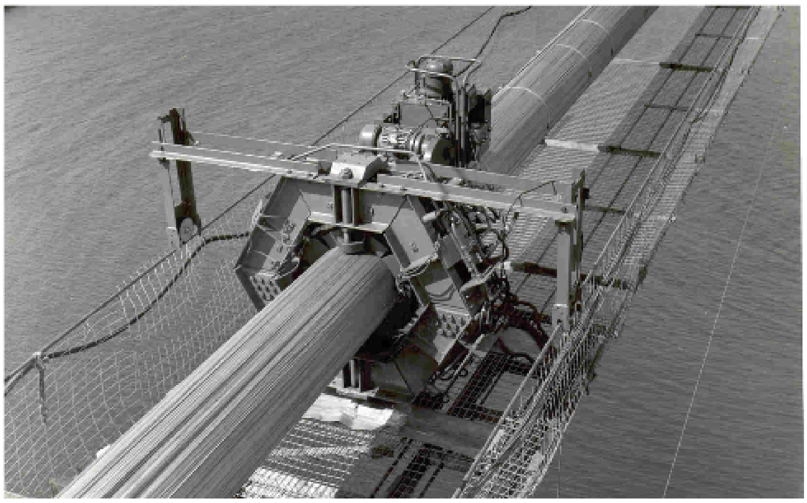
SECTION AT SIDE TOWER
scale 1:400



PLAN SECTION ON TOWERS
Cells A,B, & C are main fabricated boxes cells D & E are formed using cover plates

PHOTO GALLERY





FORTH ROAD BRIDGE

II. Operation and Maintenance

1. INTRODUCTION

The bridge has received a high level of continuous maintenance and retrofitting to enhance the capacity of the structure since it was opened in 1964.

The bridge has a dedicated maintenance unit, which carries out routine inspection and maintenance activities. Major tasks are usually undertaken by external contractors. Over the life of the bridge regular maintenance of the main cable has been undertaken, which has also included periodic repainting. Regular external inspections have been carried out with no significant deterioration or egress of water noted.

The first internal inspection of the main cables took place in 2004. The recommended number of inspection locations was increased from 6 to 8, partly as the bridge was 10 years older than the minimum age of 30 years recommended for carrying out the first inspection, and partly to give a greater representative geographic coverage of different areas of the bridge.

2. HISTORY OF MAJOR WORKS

Viaduct Box Girder Strengthening (1977)

Main Tower Wind Bracing (1992)

Main Tower Strengthening (1997)

Pier Ship Impact Works (1998)

Hanger Replacement (2000)

Resurfacing (2001-2007)

Main Cable Internal Inspections (2004/5, 8 & 12)

Main Cable Acoustic Monitoring (2006)

Main Cable Dehumidification (2009)

Viaduct Bearing Replacement (2011)

Main Tower Painting (2012)

Anchorage Investigation (2013)

Cable Band Bolt Replacement (2013)

Fractured Truss End Link Repair (2015/6)

3. CABLE INSPECTIONS

3.1 First internal inspection in 2004/5

In 2004/5 the first internal inspection of a main cable was undertaken. This inspection revealed significant corrosion in the wires forming the cable. However, the results varied significantly for each of the cables:

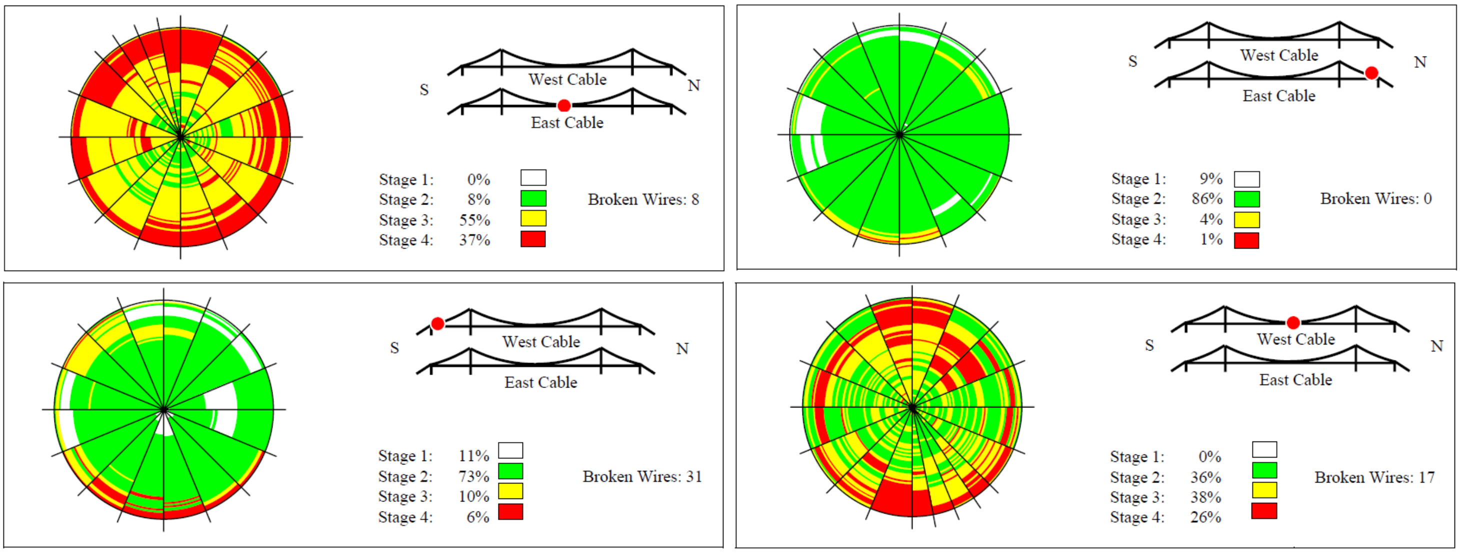


Figure 1: Inspection results for First – Fourth Panels

Various reasons for the variations were considered: cable slope, orientation and prevailing wind direction or the direction the cable was wrapped – uphill wrapping might have held any water in the cable, whereas downhill wrapping would tend to squeeze the water out.

In winter further two panels were inspected but again with variable results. The following spring inspections of the four high level panels were carried out. Conditions ranged from average to good, with no particular findings. For summary see the table below.

Table 1. Summary of Inspection Results

	Cable Panel	High/Low	Corrosion Level				Broken
			Stage 1	Stage 2	Stage 3	Stage 4	
1.	E100S-100N	Low	0 %	8 %	55 %	37 %	8
2.	E00N-02N	Low	9 %	86 %	4 %	1 %	0
3.	W00S-02S	Low	11 %	73 %	10 %	6 %	31
4.	W100S-100N	Low	0 %	36 %	38 %	26 %	17
5.	W00N-02N	Low	11 %	75 %	6 %	8 %	8
6.	E100N-98N	Low	0 %	16 %	58 %	26 %	8
7.	W22N-24N	High	0 %	45 %	42 %	13 %	5
8.	E18S-20S	High	19 %	60 %	19 %	2 %	0
9.	W76S-74S	High	7 %	58 %	25 %	10 %	2
10.	E60N-58N	High	22 %	43 %	26 %	9 %	7

Table1: Summary of Inspection Results

3.1.1 Testing and Evaluation

One 6 metre long sample wire was removed from each groove and was tested in a local laboratory. The majority of the testing was tensile, with each sample providing ten tensile specimens. A total number of tensile tests was 704.

A few wires exceeded the upper specified strength limit. A large proportion (31%) of the test samples lies below the minimum specified tensile strength (100 Imp).

One specimen from each sample wire was measured up for the stress-strain relationship. Zinc coating tests were carried out.

The cable strength was evaluated with focus on the proportion of the wires that have cracks. Of the 46 Stage 4 wires tested, 11 found to have cracks. They all lay within the outer 6 rings of wires. It was recommended that the current strength loss was between 8 and 10 %.

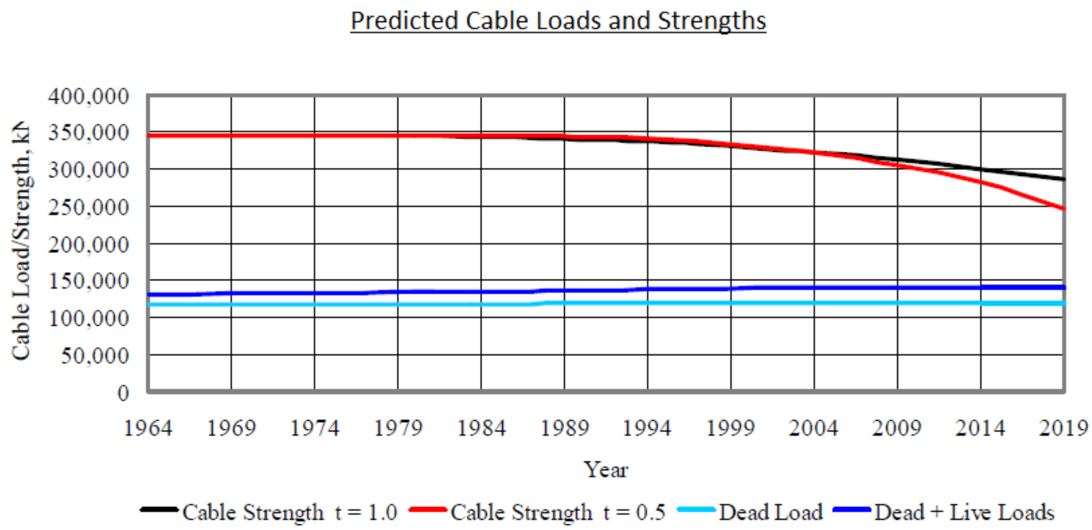
Loads in the main cables were reviewed. It showed that the bridge was somewhat lighter at opening than expected, but subsequent additions and modifications added up the weight near the original intention. The dead load is about 158 kN/m in the main span and 210 kN/m in the side spans. Live load was also assessed. However, the cable force is dominated by the dead load.

3.1.2 Cable Safety

Given the limited nature of a first inspection and the inherent lack of certainty in the results, it was agreed to set the minimum acceptable factor of safety as 2.0.

After confirmation that the cables were safe, further investigation focused on future predictions.

The predicted changes in cable loading and strength with the corresponding factor of safety are given below:



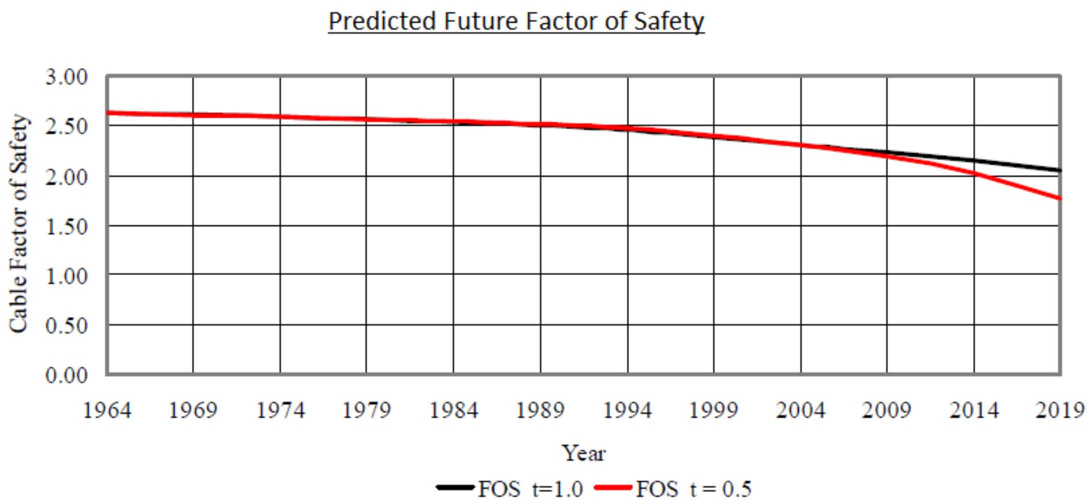


Figure2: Predicted changes in cable loading and strenths, with the corresponding factor of safety

3.2 Subsequent internal inspections

In 2008 a second internal inspection of the main cables was undertaken with the aim to set a benchmark from which the effectiveness of remedial measures could be measured. Estimation of the strength loss in the main cables was of the order of 10%. The inspection confirmed the 2004/5 inspection findings.

A third inspection of the cable was carried out in 2011/2012 to determine:

- Effectiveness of the dehumidification
- Another point on the loss of strength curve

The third internal inspection of the main cables confirmed the findings from the earlier inspections that the external and internal wires have suffered deterioration in a non-uniform manner in all eight panels inspected.

The report concluded that the cables should continue to be inspected in accordance with the NCHRP guidelines with the next internal inspection programmed for 2017.

4. ACCOUSTIC MONITORING

The rate at which wires break can be measured using acoustic monitoring techniques. It also enables to monitor the entire length of the cables.

Fully wired monitoring system was implemented in 2006. This system utilised 15 No sensors on each main cable.

In 2015, a replacement acoustic monitoring system was commissioned which utilises 58 No sensors on each main cable to provide more detailed information.

5. DEHUMIDIFICATION

Dehumidification of the main suspension cables based on recent developments elsewhere was one of the possibilities investigated to prolong the life of the cables.

The system was installed to slow down the rate of corrosion. The process involves pumping dried air into the cable at various points, having first wrapped it in an airtight neoprene.

6. REPLACEMENT OR AUGMENTATION OF THE MAIN CABLES

The first inspection of the main suspension cables opened the discussion whether and how it was possible to replace the main cables. As a precautionary measure, a study into the feasibility of augmenting or replacing the bridge´s main cables was commissioned.

This study examined the most appropriate construction methods taking into account structural options (installation of new cables and hangers on the existing structure) including the integrity of the existing anchorages, the impact of the works on traffic and the effects on the surrounding area.

The following options were considered:

a) Replacement above:

A new cable to be constructed above the existing cable with subsequent replacement of the existing main cables.

The following was to be considered:

- Aerial spinning or pulling prefabricated formed from 91 PPWSs with each strand made up of 127 wires
- Pulling the new cable was preferred option as its installation could be more controlled. The final level would be 5.8m above the level of the existing cable.
- Forming new (it would most likely be adopted) or reuse the existing anchorages
- Alterations to the footways at the side towers as the new cable will be diverted down to the anchorages over the side towers
- The stability of the side towers is maintained and would not require any additional works

The main challenge in supporting the new cable on the main towers is how to transfer the loads into the highly stressed existing structure. Moreover, there is limited access and space.

The preferred option involved supporting the new structural steelwork at the existing saddle by a combination of strengthening and augmentation for the existing steelwork at the head of the existing main towers. It involved substantial alteration to the existing saddle.

b) Augmentation above

This would require the same work as with the replacement cable situated above the existing cable. The main difference was that in this case not all load would be transferred into the new cable. The size of it could therefore be smaller; its exact size would depend on assessment of the long-term capacity of the existing cable.

c) Augmentation to side

This would involve construction of a new cable to the side of the existing cable with a percentage of loads in the existing cable being transferred over to the new cable. The cable would be formed from PPWSs, consisting of 37 strands each of 127 wires, with the cable having a compacted diameter of 380mm. Such a cable would carry approx. 30% of the existing main cable loading. The new hangers would be inclined and connected to the existing connection bracket at the top chord of the stiffening trusses.

Anchorage considerations were similar to those described for the replacement option, with new anchorages being provided.

The main towers would carry the off-centre load and the localised moments that any cantilevered support would induce in the tower legs at the connection points.

It would be possible to erect a new leg from pier head level to support the new cable directly. The new leg would be approx. 2.8m x 1.2m in plan with a cross-sectional area of $0.38 m^2$ and its support could be provided by its connection to the existing towers located at the intersection of the tower cross bracing and the legs of the main towers.

Footways would be widened. The new saddles on the side towers would be positioned to angle the cables down to their anchorage points.

7. MAIN CABLE ANCHORAGES INVESTIGATION

Following the 2004/5 inspection of the main cables, when corrosion was found in their wires, the investigation work of the main cable anchorages was carried out in 2011 – 2013 mainly with the aim to determinate their remaining service life.

The main cable anchorages are formed of concrete filled tunnels within rock. The concrete cast within the tunnels creates four large, individual, concrete plugs to which the main suspension cables are attached. The tunnels were pre-tensioned by galvanised steel strands within grouted ducts. In the design no access to inspect, maintain or monitor the system of pre-tensioning was provided.

The main cable, comprising 11618 wires in 37 bundles, is split into 37 separate strands at the splay saddle located within the anchorage chamber.

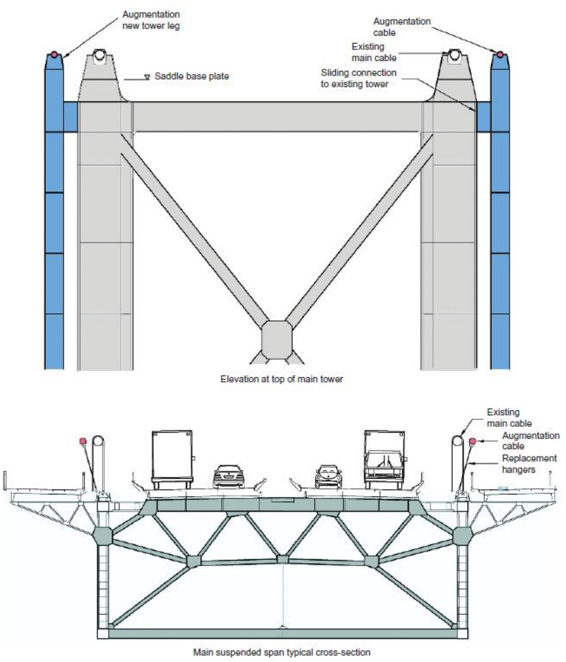


Figure 3: Augmentation to Side

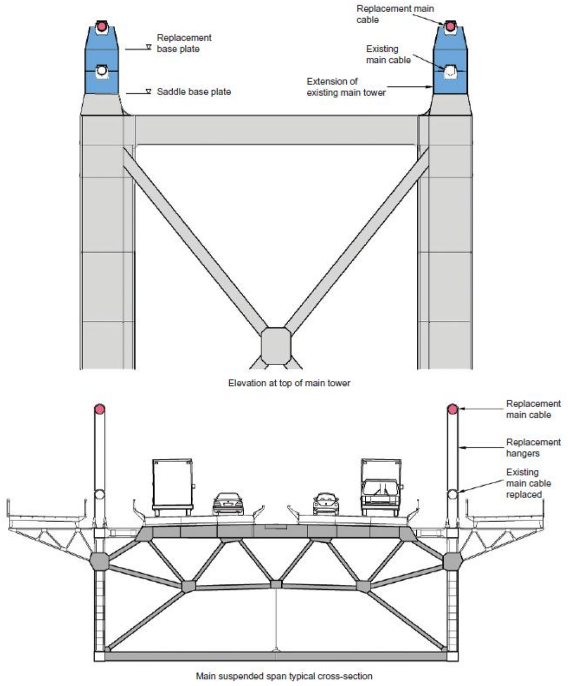


Figure 4: Replacement Above

Figure 5: Cross Head Slabs

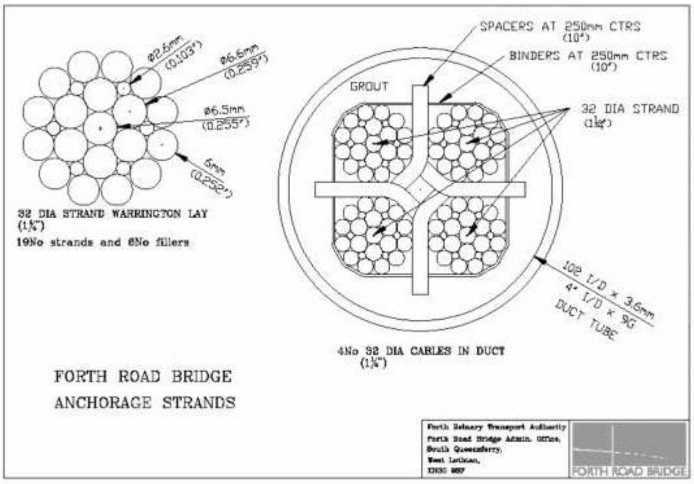


Figure 6: Anchorage Strands

Concerns regarding long term structural integrity of the pretensioning strands arose from the design and construction information, especially:

- Existence of voids in the grout within the ducts.
- Ability to fully grout the ducts and additional forces caused by bends in the ducts.
- All four strands were socketed in a single steel casing, making it difficult to open the wires without misalignment.
- During the socketing the hot metal used for galvanising may have become molten and any corrosion protection beyond the socket would be lost.
- Water ingress reported during the construction may have caused corrosion to the wedges that hold the strands.

In 2005 previously unseen records and papers were acquired. They related to the construction of the anchorages and indicated that especially the southern anchorages might be in potentially worse condition than had been described before.

It was decided, as part of the feasibility study on the options for the replacement or augmentation of the main cables, that investigation into the condition and long term structural integrity of the main cable anchorages would be carried out. A Peer Review Panel was established to audit and review the work.

Three separate methods of investigation were proposed:

- Excavation behind the anchorage chambers down to the top of the tunnel to expose, inspect and test the pre-tensioning strands.
- The full scale load testing of a number of the sockets within the anchorage chamber (which was not accepted).
- Non-destructive testing – e. g. Acoustic monitoring, the use of radar, radiography and magnetostriction scanning.

The main task was to excavate down behind the south west and south east anchorage chambers to expose the top of both the west and east tunnels. The concrete was removed by hydro-demolition. All 18 ducts were opened, grout washed out and wires inspected.

Figure 7: Excavation – overburden, rock and concrete

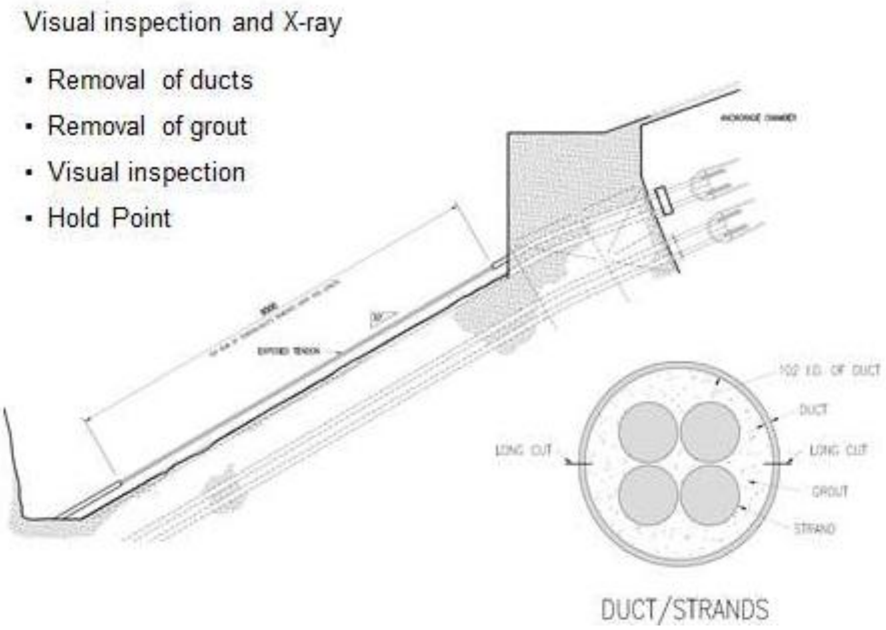


Figure 8: Exposure of tendons at crown of tunnel

The external surfaces of all the exposed ducts were found to be in very good condition. It was found that the ducts were well protected by the tunnel concrete and the anchorages were not allowing the ingress of water.

When the first ducts were cut open, the grout was found to be in remarkably good condition. The wires in the strands were without any visible signs of corrosion. No evidence of a change in diameter was found suggesting no internal corrosion had occurred.

All the strands within the 18 ducts were exposed and all were found to be in a similarly good condition. No significant negative changes were found.

After careful consideration it was decided that enough evidence had been gathered to enable the conclusion that the anchorages on the southern bridgehead were in satisfactory condition and that the risk of having to replace the anchorages during the remaining service life of the bridge was relatively low.

The ducts and grout were reinstated followed by sequenced mass infill concrete casting.

Figure 9: All ducts exposed in south west tunnel

Figure 10: Strands exposed and wedged to enable visual inspection around the circumference

8. CABLE BAND BOLT REPLACEMENT

Cable band bolts form a small but vital part of the suspension bridge. Their primary function is to generate sufficient friction to prevent slippage of the bands down the main suspension cables under loads applied by the hangers.

In the main span there are typically 4 bolts per cable band which increases to 6 near the towers where the cable is steeper. The heavier side spans typically have six bolt bands except near the main and side towers where there are 8 and 4 bolts respectively.

In 1995 a hanger rope was found to be frayed, and tests on some other ropes showed concerning deterioration. It was decided to replace all of the hangers. At the same time the original cable band bolts were replaced as well. The works were completed in 2000.

In 2007 during a routine inspection one nut was found to be cracked and during further checks in 2008 and 2009 nine failed nuts were found.

In March 2012 during inspection of the cable following completion of the dehumidification project a further 17 cracked nuts were found. It was decided to undertake the replacement of all 944 bolt assemblies on the bridge, in order to reinstate the structural integrity of the hanger system and minimise the risks to traffic.

Figure 11: Example cracked nut as found

Figure 12: Example cracked nut after removal

9. MAINTENANCE OF THE MAIN EXPANSION JOINTS

In each carriageway of the suspended spans, at both main towers, there are two joints. Each of these joints (in total eight) is of the roller shutter type whereby a series of plates slide over fixed curved girders.

Roller shutters of this type have a typical design life of 20 to 30 years. In 1975 a detailed inspection was carried out. It was reported that the joints were generally performing well with some evidence of wear.

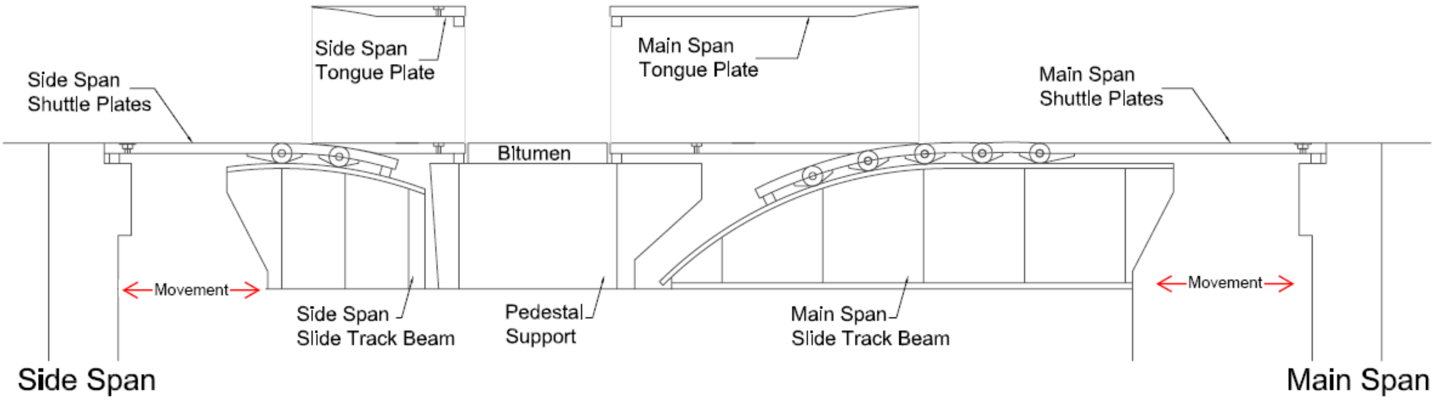


Figure 13: Long section through the joint

Further, more recent inspections had shown further excessive wear in the curved link plate support beams and in the hinges and feet of the link plates. This increases noise and play in the mechanism and wear surfaces, causes rapidly increasing damage to the joint with ultimate failure of the joint resulting.

There are also further joints in both carriageways of the approach viaducts comprising of interlocking combs (or fingers), and there are also corresponding joints in both foot / cycle ways. It was recommended, because their movement is less and they had suffered less wear and tear, that these joints continue to be monitored and inspected and their refurbishment or replacement might be carried out during other future closures.

The replacement or refurbishment of all the expansion joints was recognised as a maintenance priority; the rate of wear was likely to rapidly increase.

In 2007 a feasibility study was commenced, and options to either repair, refurbish, or replace each type of the expansion joint were considered.

The works would cause major disruption to the road users so the timing of the works needed to be coordinated. Timing, periods of closure and economic impact were considered.

To allow traffic flow to continue during replacement work temporary ramps over the expansion joints were proposed. The ramps would be 80 m long with two lanes; the headroom under them for working was a minimum of 1.8m. The existing bridge deck would have to be strengthened at each tower to accommodate these temporary structures.

Tenders were received for the replacement of the main expansion joints during 2008. However, the cost of the temporary works pushed the prices out with the available budget at that time. At the same time, the Scottish Government announced the timescale for the building of a second bridge across the Forth.

In this situation, the main focus was to determine whether or not the replacement of the joints could be deferred until after the opening of the new bridge and the removal of the majority of the traffic off the existing bridge. A review of the project was commenced.

This included undertaking a Failure Mode and Effect Analysis (FMEA).It was seen as the best means of identifying the likelihood and consequences of the failure of the various components that make up the joints. During a weekend closure another inspection was carried out during which one sliding train was removed.

The review team concluded that it would be possible to delay the replacement of the joints subject to the following:

- Significant increase in inspection and monitoring level.
- Installation of the permanent access to aid inspection.
- Replacement of key components such as pins and springs.
- Installation of temporary failsafe devices.
- Annual revision of the decision.

These measures have made it possible to maintain operational safety levels for bridge users.

Figure 14: Virtual reality model of ramps

Figure 15: Plate train being removed

10. FRACTURED TRUSS END LINK AND REOPENING OF THE BRIDGE

The photographs on this page show the location of the failed truss end link, on the south side of the north main tower and a close up of the failed member showing the crack.

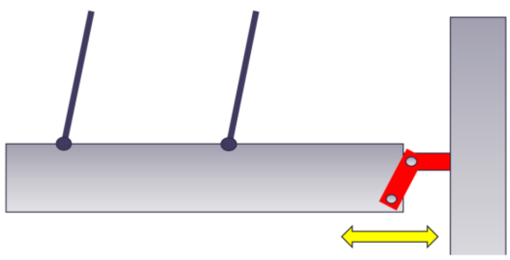
The truss end link carries vertical loading into tower bracket, it accommodates longitudinal truss end movement. Live load is dominated by traffic loading, wind and temperature.

On 1 December 2015 during an inspection a broken link was identified. The failure caused loss of vertical support for a 400t section of the truss and created a cantilever supported by nearest hanger.

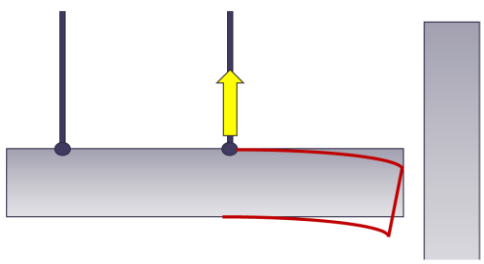
There was the potential for the corner of the truss to drop by approximately 250mm if the remaining section of the damaged link failed. This situation would result in severe damage to the bridge and risk of injury to users so it was decided to close the bridge to traffic on 3 December.

Following the installation of a temporary support at the location of the damaged link, the bridge reopened to cars and light vehicles on 23 December 2015, and to all traffic, including heavy goods vehicles, on 20 February 2016.

Role of Truss End Link



Effect of Truss End Link Failure



The repair had two phases:

Phase 1: Repair – allowed all traffic to return except HGVs (23 December 2015)

Design included specification of main loads, identification of structural requirements on repaired link and construction considerations.

Following repair load testing was carried out, the bridge was monitored using 150 strain gauges and displacement sensors and 4 data units.

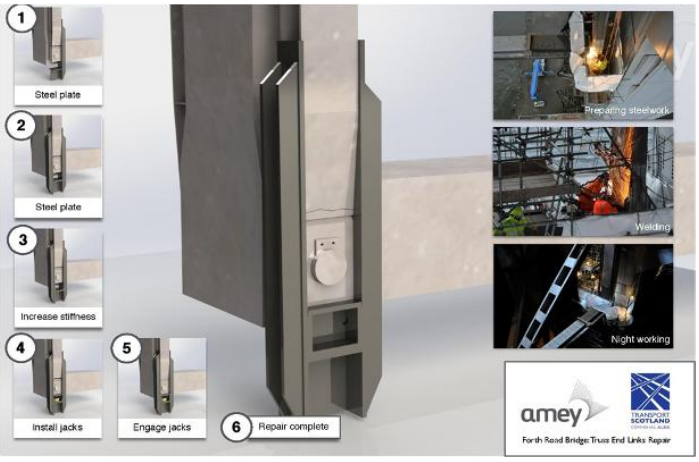
Phase 2: Support – allowed all traffic to return (20 February 2016)

Additional bracket was bolted to tower face above road / carriageway level, spreader beam installed below carriageway level and bridge cables / strand jacks were installed to support the truss.

After opening the bridge, Phase 3: Works follows:

- Existing truss end link member and post are removed.
- New post, support bracket and bearing are installed.
- Concrete is poured to strengthen the tower.

Structural health is constantly monitored by strain gauges, displacement transducers and temperature sensors. WIM data are provided.



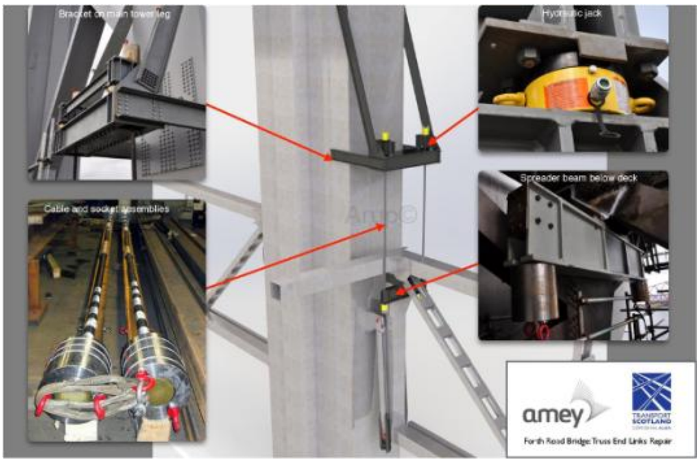


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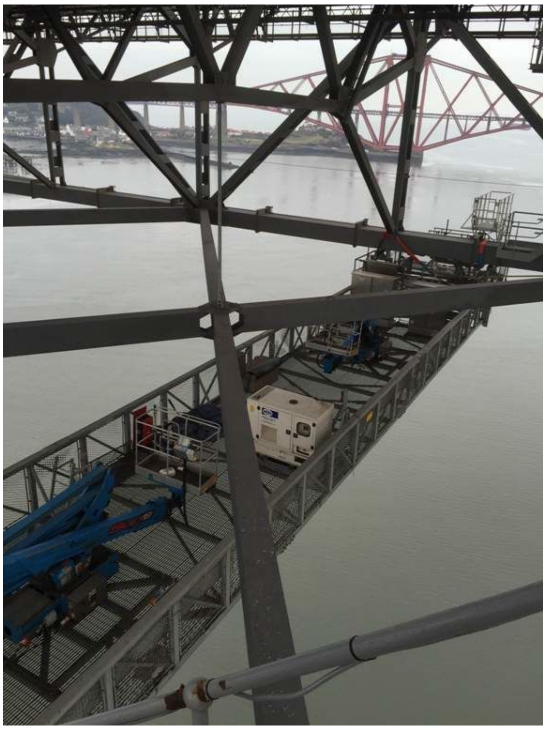
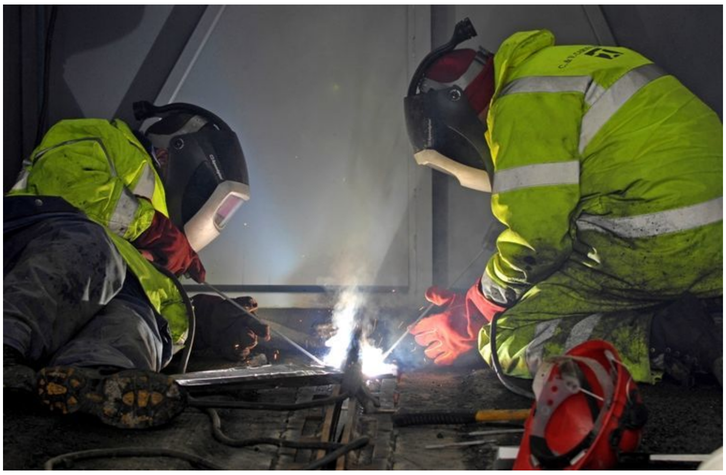
PHOTO GALLERY





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MAINTENANCE


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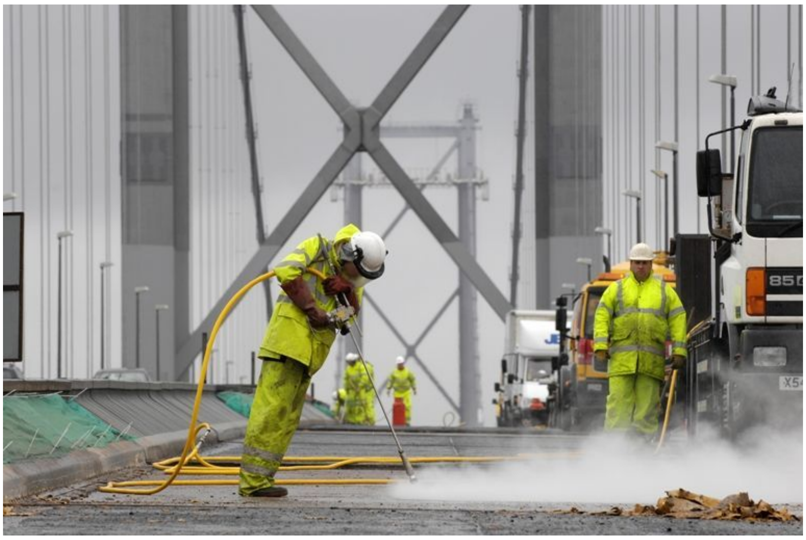
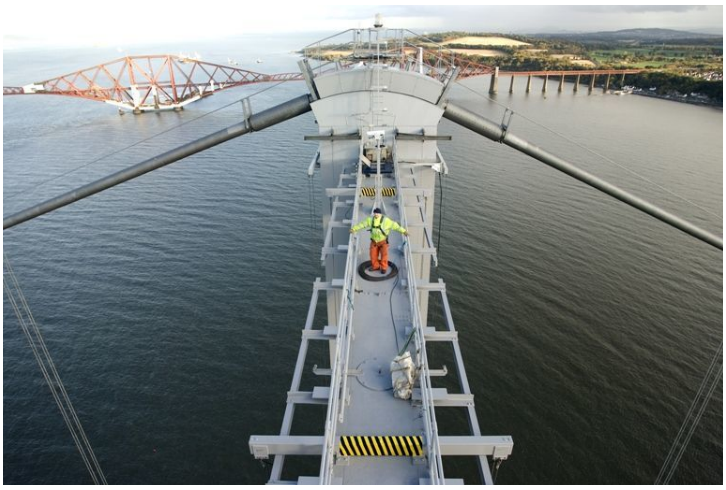
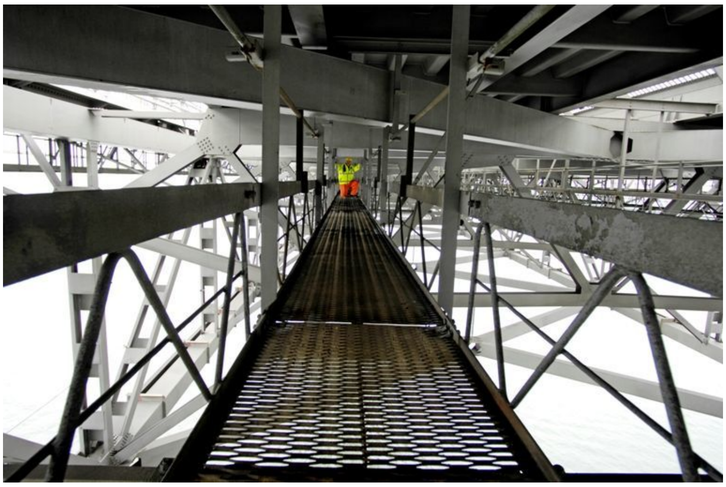
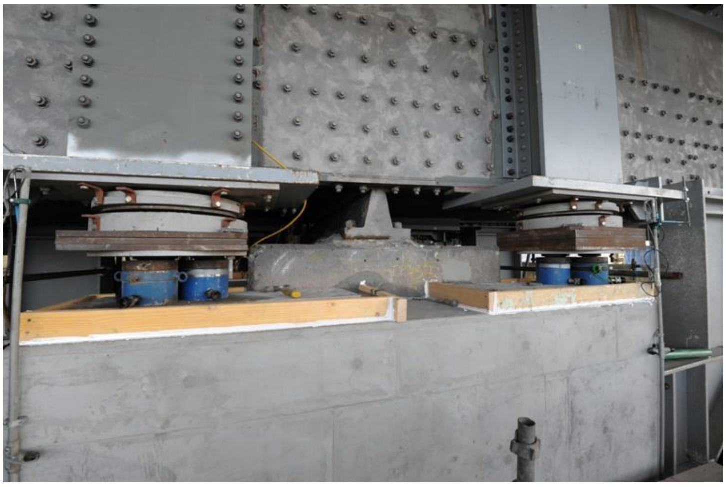
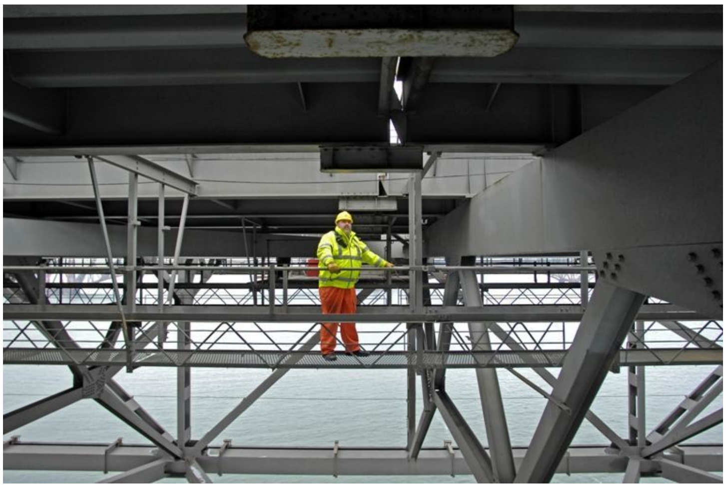


FORTH ROAD BRIDGE - AWARDS

SALTIRE SOCIETY SCOTLAND
COMANN CRANN NA H-ALBA

PEOPLE'S CHOICE AWARD

2016 Civil Engineering Winners

The National Museum of Scotland played host to the 2016 Civil Engineering Awards on the 25th of October. Cabinate Secretary for Rural Economy and Connectivity, Fergus Ewing MSP, was there to see the following awards presented.

Award for Greatest Contribution to Scotland

Forth Road Bridge Truss End Links Repair

ice Institution of Civil Engineers

ICE People´s Choice

Forth Road Bridge reopening

The iconic Forth Road Bridge was closed to traffic in December 2015 following discovery of a fractured truss end link. Engineers reopened the repaired bridge within three weeks.

The ICE People´s Choice is being presented by Nicola Sturgeon MSP First Minister of Scotland

FORTH RAILWAY BRIDGE

Peter Paulík, Slovak Technical University

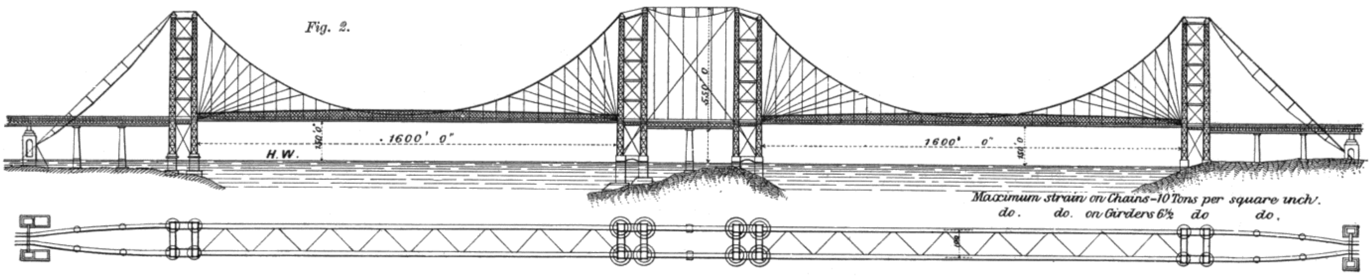
Photo Credit: Dan Crocker of DC Structures Studio (NZ)

The bridge spans the Forth between the villages of South Queensferry and North Queensferry and has a total length of 2,467m. It was the longest single cantilever bridge span in the world until 1917 when the Quebec Bridge in Canada was completed. It continues to be the world's second-longest single cantilever span.

Prior to the construction of the bridge, ferry boats were the only method available to cross the Firth. The first proposal to build a bridge came in 1818 by James Anderson. Anderson proposed to build a three-span suspension bridge with a design which needed approximately 2,500 tons of iron. Wilhelm Westhofen in 1890 in his book "The Forth Bridge" commented on this proposal with sarcasm "this quantity [of iron] distributed over the length would have given it a very light and slender appearance, so light indeed that on a dull day it would hardly have been visible, and after a heavy gale probably no longer to be seen on a clear day either."

Even with the next attempt to design a bridge across the Forth it proved difficult to engineer a suspension bridge which would be able to carry railway traffic. For this reason, engineer Thomas Bouch decided to change the location for the bridge and in 1863 he started work on the design of a single-track girder bridge crossing the Forth near Charlestown, where the river is around two miles wide, but mostly relatively shallow. However by mid-1867 the North British Railway was nearly bankrupt, and all work on the project was stopped.

With the takeover of the ferry service between North and South Queensferry by the North British Railway Company in 1867 and the construction of a connecting line from Ratho, west of Edinburgh, in 1868, interest in the provision of a fixed link across the Forth increased again. In 1871, Bouch proposed a stiffened steel suspension bridge at the site of the present rail bridge.

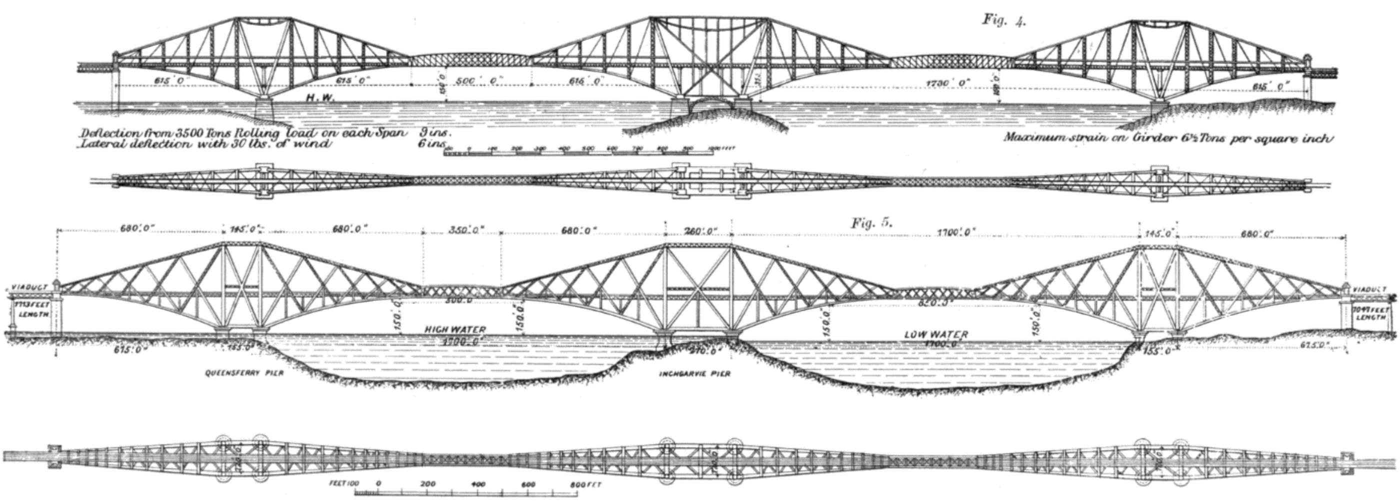


Bouch's proposed bridge in 1871

Works started in 1878 with the construction of one of the piers. But once again circumstances were against this challenge. After the Tay Bridge collapsed in 1879 works were stopped almost immediately. The public inquiry into the disaster, chaired by Henry Cadogan Rothery, found the Tay Bridge to be "badly designed, badly constructed and badly maintained," with Bouch being "mainly to blame" for the defects in construction and maintenance and "entirely responsible" for the defects in design. Bouch's design of the Forth Bridge was formally abandoned on 13 January 1881, and Sir John Fowler, W. H. Barlow and T. E. Harrison were invited to give proposals for a bridge. One of the piers of from the abandoned works for Bouch's bridge remains to this day as the base of a lighthouse.

Design

The new design was made on the principle of the cantilever truss bridge, where a cantilever beam supports a light central girder, The Bridge is 2,467m long, and the double track is elevated 45.72m above the water level at high tide. It consists of two main spans of 518.16m, two side spans of 207.3m, and 15 approach spans of 51.2m. Each main span consists of two 207.3m cantilever arms supporting a central 106.7m span truss. The weight of the bridge superstructure was 51,324 t, including the 6.5 million rivets used. The three great four-tower cantilever structures are 110.03m tall, each tower resting on a separate pier. These were constructed using 21m diameter caissons. The bridge was the first major structure in Britain to be constructed of steel instead of wrought iron.



The original and final design of the bridge

In order to illustrate the use of tension and compression in the bridge, a demonstration in 1887 had the Japanese engineer Kaichi Watanabe supported between Fowler and Baker sitting in chairs. Fowler and Baker represent the cantilevers, with their arms in tension and the sticks under compression, and the bricks the cantilever end piers which are weighted with cast iron.

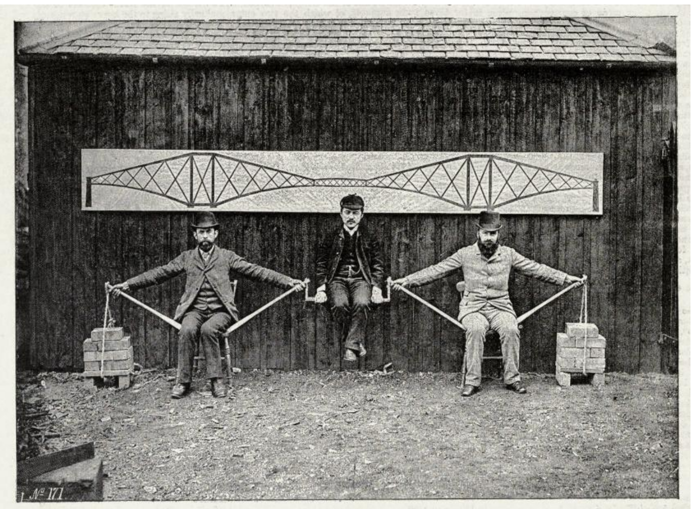


FIG. 5A. LIVING MODEL ILLUSTRATING PRINCIPLE OF THE FORTH BRIDGE.

Construction

The preparations at South Queensferry required the steep hillside to be terraced. Drill roads and workshops were built, as well as a drawing loft (61 by 18 m) to allow full size drawings and templates to be laid out. A cable was laid across the Forth to allow telephone communication between the centres at Queensferry, Inchgarvie, and Fife, and girders from the collapsed Tay Bridge were laid across the railway to the west in order to allow access to the ground there. Many materials, including granite from Aberdeen (18,122 m^3 in total), Arbroath rubble, sand, timber, and sometimes coke and coal, could be taken straight to the centre where they were required. Steel was delivered by train and prepared at the yard at South Queensferry before being painted with boiled linseed oil before being taken to where it was needed by barge.

Near the shore a sawmill and cement store were erected. The cement used was Portland cement manufactured on the Medway.

After preparation works were done the main construction began in 1882.

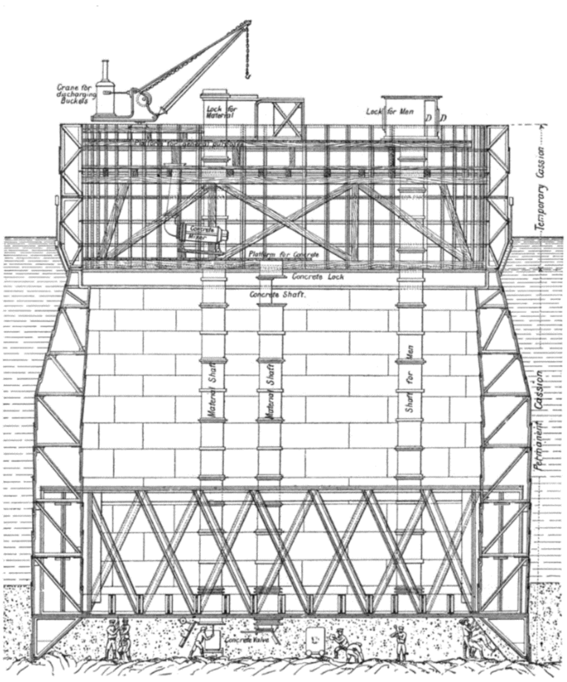
Caissons

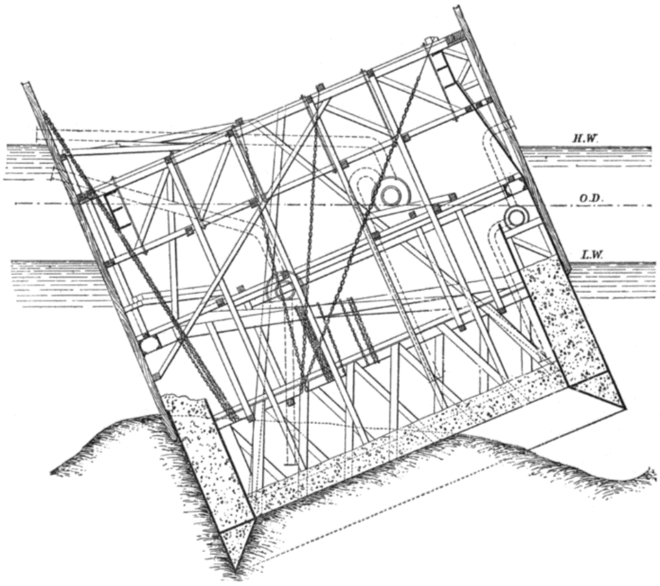
The three towers of the cantilever are each seated on four circular piers. Since the foundations were required to be constructed at or below sea level, they were excavated with the assistance of caissons and cofferdams. Six caissons were excavated by the pneumatic process, which allowed dry working conditions even at depths of up to 27 m. These caissons were constructed and assembled in Glasgow and transported in dismantled state to Queensferry. Then they were reassembled and floated to their final resting-places.

The first caisson, for the south-west pier at Queensferry was launched on 26 May 1884, and the last caisson was launched on 29 May 1885. When the caissons had been launched and moored, they were extended upwards with a temporary portion in order to keep water out and allow the granite pier to be built when in place. Excavation beneath the caissons was generally only carried out at high tide when the caisson was supported by buoyancy, and then when the tide fell the air pressure was reduced in order to allow the caisson to sink down, and digging would begin anew.

However not everything went smoothly. The north-west caisson was towed into place in December 1884, but an exceptionally low tide on New Year's Day 1885 caused the caisson to sink into the mud of the river bed and adopt a slight tilt. When the tide rose, it flooded over the lower edge, filling the caisson with water. Even after the tide fell the situation did not get better since the water did not drain from the caisson but instead, its top-heaviness caused to tilt the caisson further. Plates were bolted on by divers to raise the edge of the caisson above water level, and the caisson was reinforced with wooden struts as water was pumped out, but pumping took place too quickly and the water pressure tore a hole between 7.6 and 9.1 m long.

It was decided to construct a "barrel" of large timbers inside the caisson to reinforce it, and it was ten months before the caisson could be pumped out and dug free. The caisson was refloated on 19 October 1885, and then moved into position and sunk with suitable modifications.





The mode of sinking the Queensferry caissons

The tilted caisson

Approach viaducts

The approaches were constructed and designed by James Carswell under separate contract. The approach viaducts to the north and south had to be carried at 39.78m above the level of high water, and it was decided to build them at a lower level and then raise them in tandem with the construction of the masonry piers.

Two spans were attached together to make a continuous girder, with an expansion joint between each pair of spans. Due to the slope of the hill under the viaducts, the girders were assembled at different heights, and only joined when they had reached the same level. Lifting was done using large hydraulic rams, and took place in increments of around 1.0m every four days.

Building the superstructure

The tubular members were constructed in a workshop further up the hill at Queensferry. To bend plates into the required shape, they were first heated in a gas furnace, and then pressed into the correct curve. The curved plates were then assembled on a mandrel, and holes drilled for rivets, before they were marked individually and moved to the correct location to be added to the structure. Lattice members and other parts were also assembled at South Queensferry, using cranes and highly efficient hydraulic rivetters.

The main compression members are steel tubes ranging up to 3,6m in diameter, the tubular form being adopted for two reasons, firstly, because experiments have shown that inch for inch the tubular form is stronger than any other, and, secondly, because the amount of stiffening and secondary bracing is thereby reduced to the lowest percentage.

Assembly of the cantilevers took 4 years to complete. At the peak, approximately 4,600 workers were employed in the bridge's construction. Wilhelm Westhofen recorded in 1890 that 57 lives were lost.

Opening

The bridge was completed in December 1889, and load testing of the completed bridge was carried out on 21 January 1890. Two trains, each consisting of three heavy locomotives and 50 wagons loaded with coal, totaling 1,880 tons in weight, were driven slowly from South Queensferry to the middle of the north cantilever, stopping frequently to measure the deflection of the bridge. This represented more than twice the design load of the bridge. A few days previously there had been a violent storm, producing the highest wind pressure recorded to date at Inchgarvie, and the deflection of the cantilevers had been less than 25 mm. Thus the bridge was tested even by nature before its final opening.

The first complete crossing took place on 24 February 1890, when a train consisting of two carriages carrying the chairmen of the various railway companies involved made several crossings. The bridge was opened on 4 March 1890 by the Prince of Wales, later King Edward VII, who drove home the last rivet, which was gold plated and suitably inscribed.

The key for the official opening was made by Edinburgh silversmith John Finlayson Bain, commemorated in a plaque on the bridge.

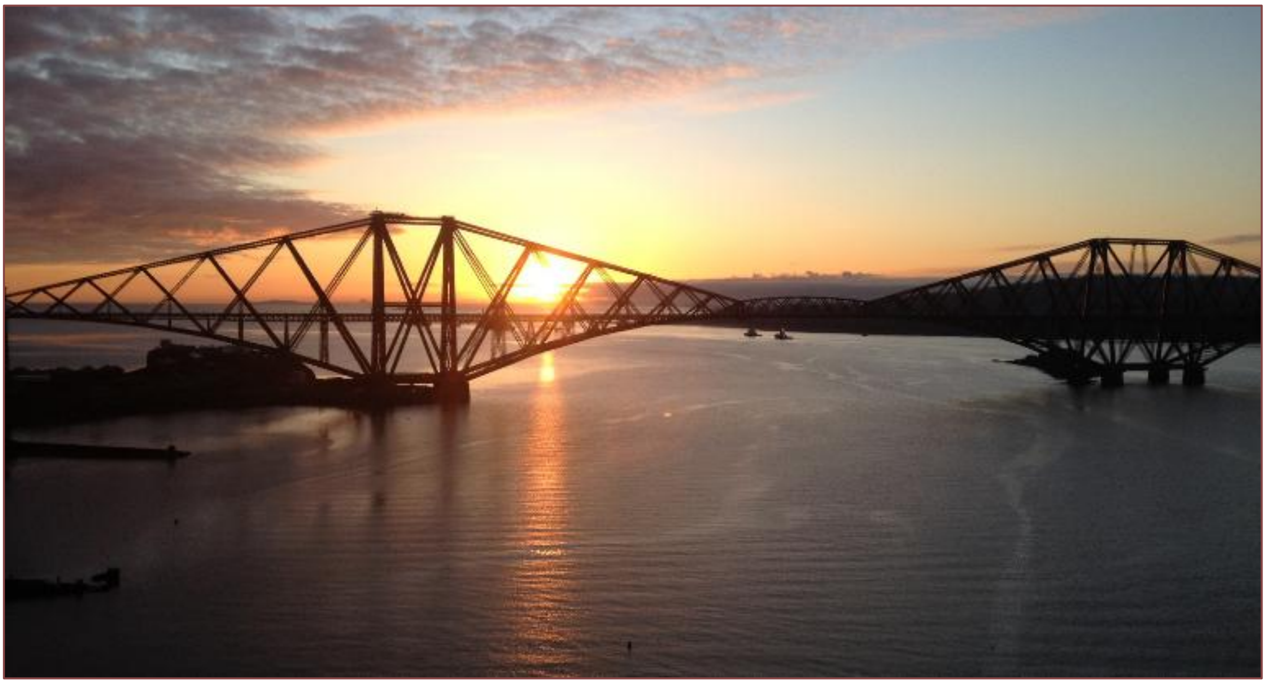
Maintenance

Approximately 190–200 trains per day cross the bridge. The bridge has a speed limit of 80 km/h for high-speed trains, 64 km/h for ordinary passenger trains and 48 km/h for freight trains. Freight trains above a certain size must not pass each other on the bridge. Work started in 2002 to repaint the bridge fully for the first time in its history (before only most weathered parts were repainted when needed).

During the repaint up to 4,000 tons of scaffolding was erected on the bridge at any time, and computer modelling was used to analyze the additional wind load on the structure. All previous layers of paint were removed by blasting using copper slag, exposing the steel and allowing repairs to be made. The scaffold was encapsulated in a climate controlled membrane to give the proper conditions for the application of the paint. Approximately 240,000 litres of specialist glass flake epoxy paint, similar to that used in the offshore oil industry and designed to last 25 years was used. It is, however, expected to last much longer. The top coat can be reapplied indefinitely, minimizing future maintenance work.

The project also involved repair and replacement of walkways including the installation of new walkways and catwalks to allow for access to the works and to assist in the future examination and maintenance of the bridge. Steel repairs involved the replacement of small localised sections of steel where required. The bridge’s architectural lighting system was also refurbished.

UNESCO inscribed the bridge as a World Heritage Site on 5 July 2015, recognizing it as "an extraordinary and impressive milestone in bridge design and construction during the period when railways came to dominate long-distance land travel."

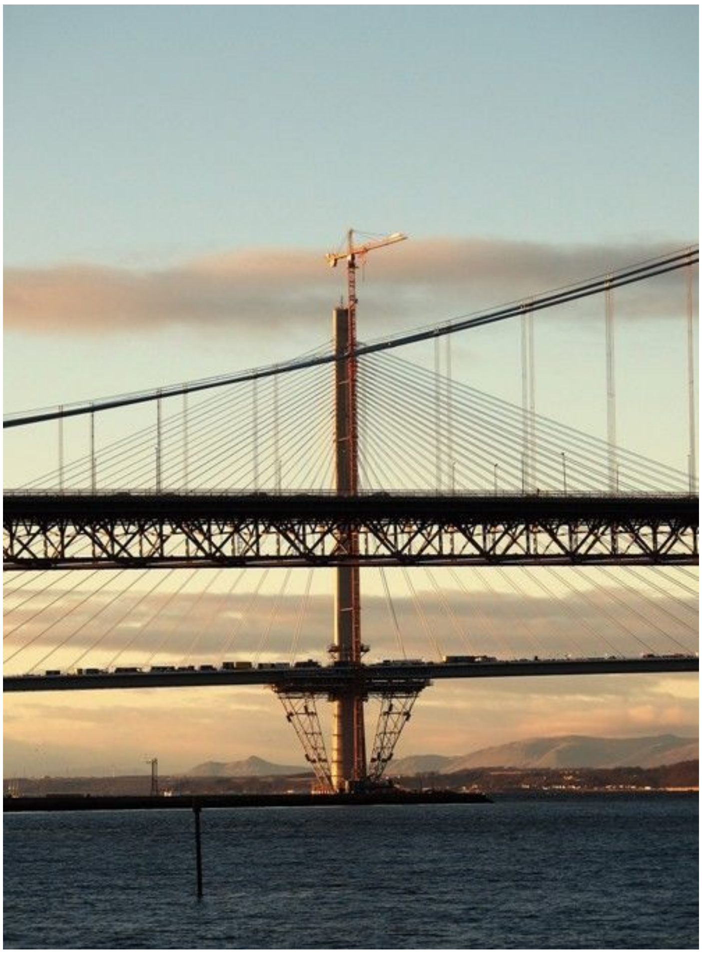


Forth Bridge Sunrise. Photo Credit: Robert McCulloch

PHOTO GALLERY





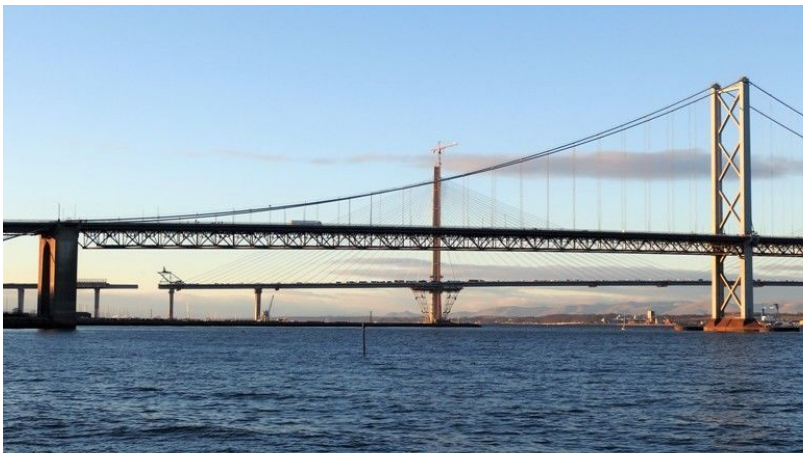
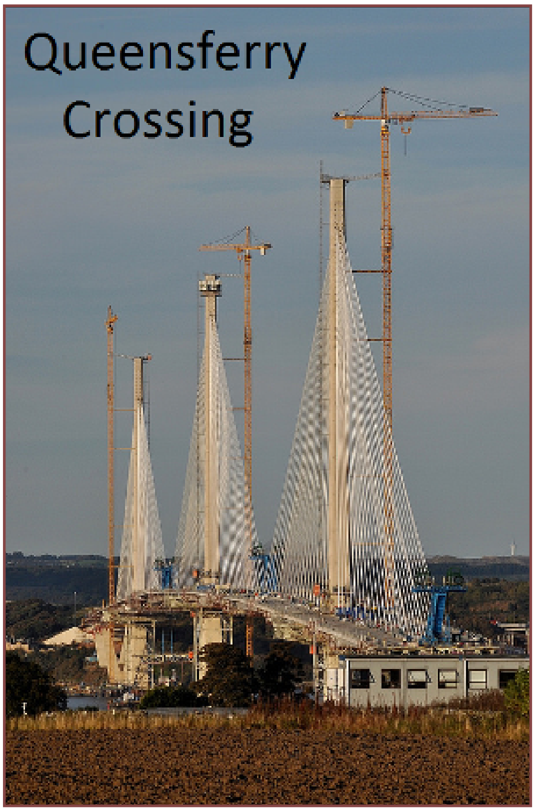


Photo Credit:

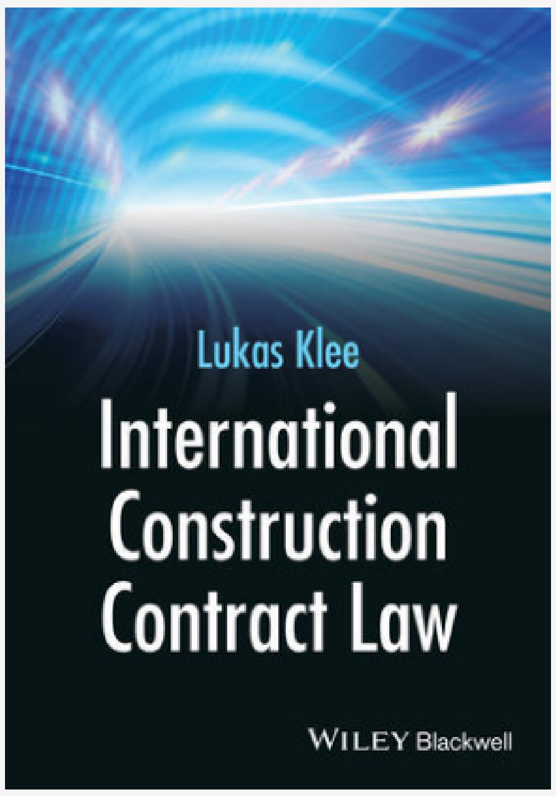
Dan Crocker of DC Structures Studio (NZ) e-mosty

Extra PHOTO GALLERY

Robert McCulloch: Three Bridges - Three Centuries



Click on the photo to open the extra photo gallery on flickr



International Construction Contract Law

Lukas Klee
ISBN: 978-1-118-71790-5
560 pages
January 2015, Wiley-Blackwell

Large international construction projects often have a range of major contractors, subcontractors and consultants based in different parts of the world and working to different legal theories and understandings. This can lead to confusion in the understanding, interpretation and execution of the construction contract, which can result in significant disruption to the construction project.

International Construction Contract Law is written for anyone who needs to understand the legal and managerial aspects of large international construction projects, including consulting engineers, lawyers, clients, developers, contractors and construction managers worldwide. In 18 chapters it provides a thorough overview of civil law /common law interrelationships, delivery methods, standard forms of contract, risk allocation, variations, claims and dispute resolution, all in the context of international construction projects. Highly practical in approach – it introduces legal analysis only when absolutely essential to understanding, the book also contains a range of useful appendices, including a 10-language basic dictionary of terms used in FIDIC forms.

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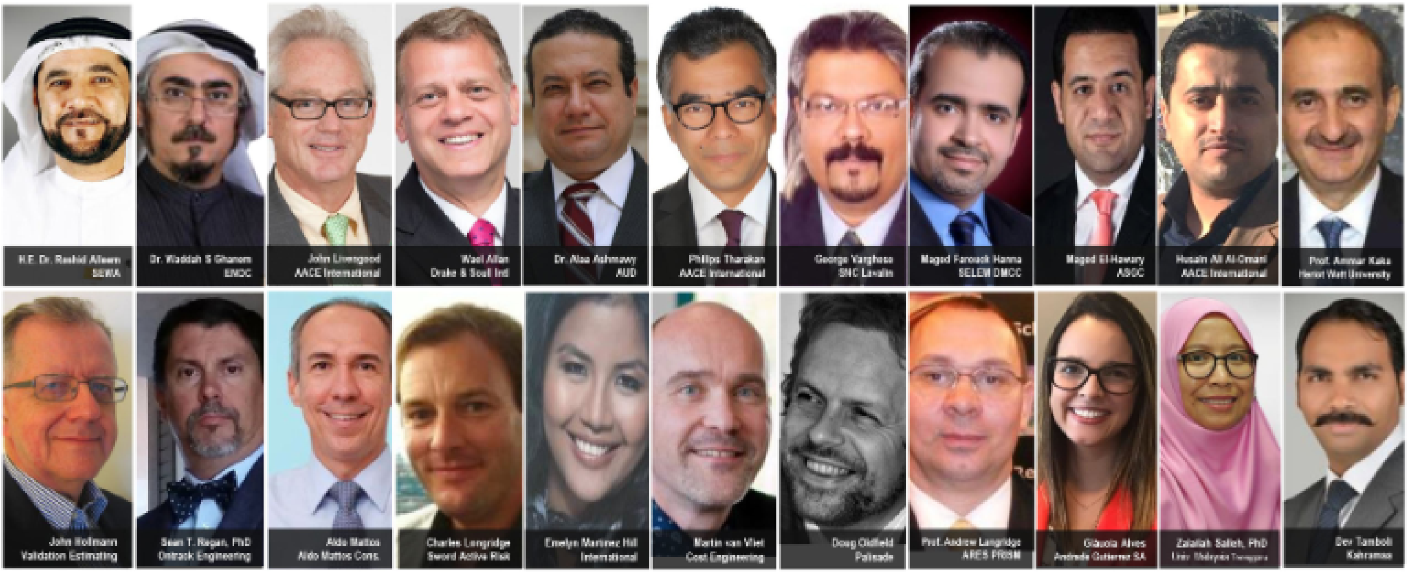

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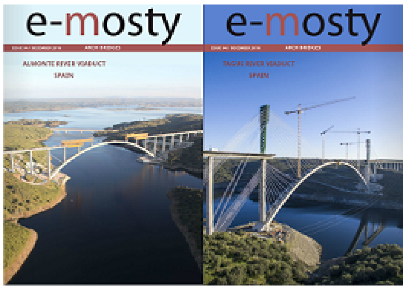
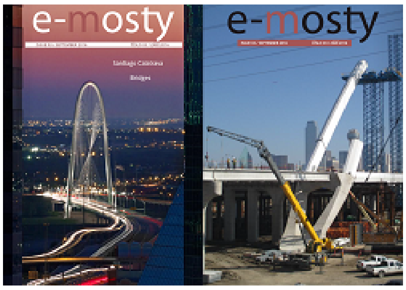


December 2016: Arch Bridges

Almonte Viaduct. Tagus Viaduct.
River Irwell Network Arch Bridge

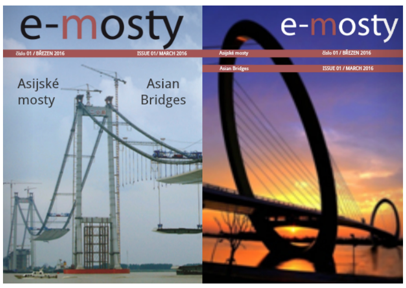


September 2016:

Santiago Calatrava. Bridges.

June 2016

3rd Bosphorus Bridge
Michel Virlogeux
Izmit Bay (Osmangazi) Bridge



March 2016: Asian Bridges
Yangtze River Bridges

Taizhou, Sutong, Runyang (incl. ground freezing method),
Nanjing 2nd, 3th, 4th and many others

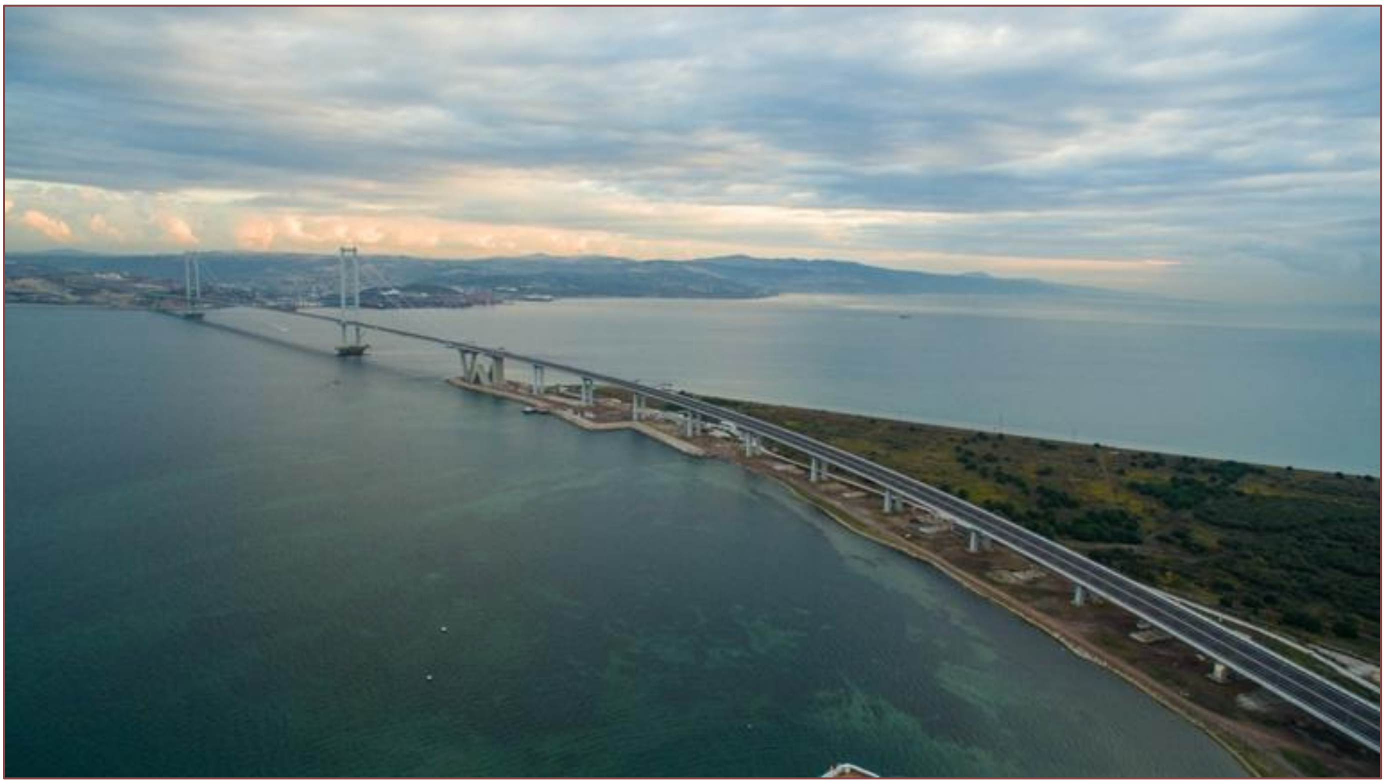
www.e-mosty.cz

e-mosty 2/2017: Suspension Bridges. It will be released on 20 June.

Osmangazi Bridge: One Year in Operation

Fatih Zeybek, Director of Maintenance

OTOYOL YATIRIM ve İŞLETME A.Ş.



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Inspection, preservation and rehabilitation of US Suspension Bridges

Barry R. Colford, Vice President, AECOM
Shane R. Beabes, Associate Vice President, AECOM

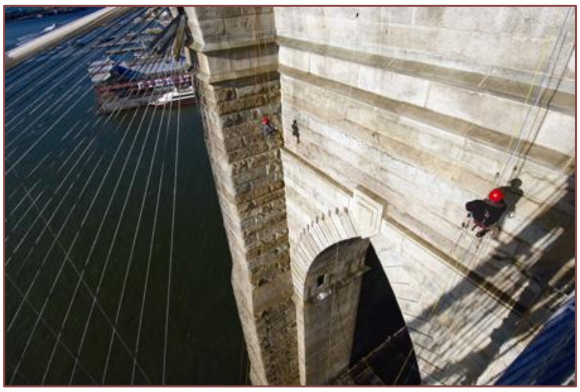
Delaware Memorial Bridge

Suspender Replacement

Delaware Memorial Bridge – Dehumidification of the main cables

Chesapeake Bridge

Walt Whitman Bridge



Brooklyn Bridge - Inspection

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ISSUE 01 / MARCH 2017

Queensferry Crossing. Forth Road and Railway Bridges.