# A CRICTICAL ANALYSIS OF  NORTH SHORE FOOTBRIDGE, STOCKTON-ON-TEES, UK

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**Abstract:** This paper is a detailed study of the North Shore Footbridge, Stockton-on-Tees, considering the geotechnics, structural design, aesthetics, construction and maintenance. Estimates in the loading and approximate strength calculations following BS5400, give an indication to the capacity of the bridge.

**Keywords:** *Infinity Bridge, North Shore Development, Double bowstring arch, The Merrison Report*
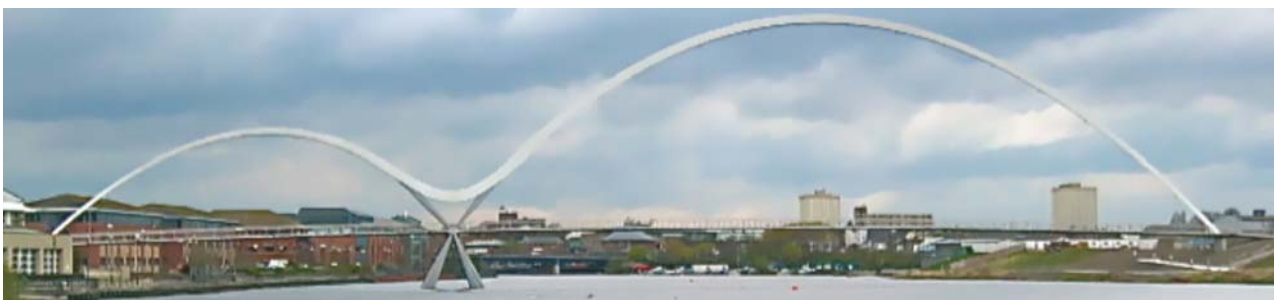


**Figure 1:** North Shore Footbridge

## 1    Introduction

The North Shore Footbridge, named the Infinity Bridge in September 2008, is officially opened from May 2009. The project was launched as an open RIBA design competition in April 2003. The brief was for a prestigious and iconic landmark to span the 125m wide River Tees, linking the Teesdale Business Park with the planned North Shore Development.

In late 2003, Expedition Engineers in association with Spence Associates won the design competition. Expedition continued with the detailed design and the contract to construct the bridge was awarded to Balfour Beatty in March 2007. Work began on site June 2007.

It is anticipated that around 4,000 people per day will use the £15m footbridge improving accessibility to the North Shore Regeneration Development and attracting further business to the area.
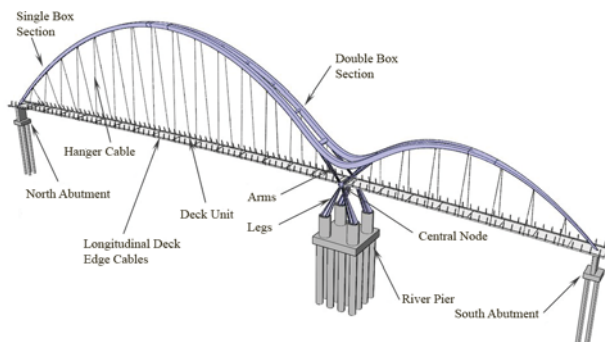


**Figure 2:** Key Elements

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## 1.1   Bridge Superstructure

The superstructure comprises of a continuous asymmetrical 'double arch' with two spans, the north at 120m and the south at 60m.  The steel arches are formed from tapered fabricated structural weathering steel plate box sections and span between abutments and over an off centre river pier. The trapezoidal box section varies from 2500mm deep by 1500mm wide to just 200mm wide by 400mm deep at the end supports. The arch is supported over the river pier by four steel arms connecting into a steel node at deck level with a pair of longitudinal deck edge cables, 90mm diameter on the north and 65mm diameter on the south tying the arches together as well as pre-stressing the deck. The central nodes are supported by steel legs, which sit on 3m diameter concrete legs below the water line.

The deck is made from 7.5m long precast (PC) concrete units, suspended by 30mm diameter high strength steel cables from the arch. The PC deck units are longitudinally pre-stressed to form a stiff in plane structural system, which resists lateral wind and translational effects at deck level. A stainless steel and stressed wire enclosure system is used.

A mild steel fabricated box element incorporated within the end deck units at both riverside supports provides a reaction system to transfer and balance horizontal deck cable forces and horizontal arch thrusts. They also distribute the vertical arch forces to the main bearings on the abutment as well as also providing a thrust frame for both initial stressing of the deck cables and re-stressing during the life of the footbridge.

**Figure 3:** Reaction Frame

## 1.2 Articulation arrangement

The river pier group is designed to resist ship impact and therefore provides much of the horizontal stiffness. As such, the support elements at the river pier cantilever from the pile cap and provide vertical, lateral and longitudinal restraint to the bridge.

The connection at this junction of the elements forming the legs and arms is a welded node. Rotational flexibility about an axis orthogonal to the plane of the arches is achieved through element local bending flexibility.

Pin joints connect the deck structure and the arch at the abutments. This allows for rotation in the orthogonal plane of the arch.

The bridge superstructure is supported on bearings, which provide restraint both vertically and laterally, but not longitudinally. Fig. 3. The bearings also allow rotation in the orthogonal plane of the arch.

## 2    Aesthetics

The assessment will be based upon Fritz Leonhardt's ten criteria for the judgement of a bridges' aesthetics. Ref. [1].

The bridge's functionality is fulfilled through use of two asymmetrical bow-string arches that support the lightweight deck through the tension cables; representing a clear hierarchy of structural components.

The arches form much of the aesthetic debate due to the size and intrusiveness set against the surroundings. It could be argued that the large arch has an excessive span and it would have been more economical and possibly aesthetically pleasing for the river pier support to be centralised.

However, British Waterways required a minimum clear navigational waterway of 93m including a distance of 5.5m above mean water level over a 50m rowing course. A 16m wide 8m high corridor is also required for larger vessels. Ref. [2]. This means that a centralised pier wasn't possible.

The two arches are in proportion to each other; in that the larger arch is approximately twice as big as the smaller arch. If the arches were the same height, with different spans, then one arch would have appeared awkward compared to the other and would not appear to

be a continuation. The continuation is achieved through a smooth curved reflex piece located above the river pier. This section is defined by the geometry of the two arches and the curvature was differentiated several times to achieve a maximum smoothing giving the appearance of the arches as a single elegant structure.

Complexity is achieved through the reflex arch that may appear only to be for aesthetic purposes but is crucial in transferring moment across the arches hence making the structure elegant and efficient. The landscape of the surrounding area is relatively flat, and the arches could be criticised for being too high. However the large arch is required in respect to the span and the shape was 'form-found' based on the dead load. The double continuous arches appear to be like hills in the distance. Following this reasoning, arches of the same height would rarely form in nature and therefore the bridge draws inspiration from nature and subsequently becomes integrated into the landscape.

The character is further enhanced by the bifurbication of the arches; this allows a clear view from one end of the deck to the other. Structurally, this is required to resist the wind loads, allowing the ends of the arches to be tapered down where they meet the deck.

The double continuous arch appearance is enhanced with the use of colour with only the uppermost continuous structure painted a glossy white, drawing attention away from the matte grey lower portions of the arch. This is further enhanced through the use of lighting, which highlights the white gloss and the deck.

The supporting arms and legs form a pleasing continuation of the arches, but attention is drawn away from them with the careful control of colour and lighting. Making best use of materials where aesthetics aren't important occurs beneath the water line where thick concrete vertical legs support the steel.

This visual effect of the arch continuation within the substructure isn't replicated at either of the shore abutments where a vertical pier identical to the others that support the approach ramps is used instead. However, this could have created an awkward void shape between the angle pier and adjacent vertical pier, as well as being highly inefficient structurally.

The end piers don't pick up the approach spans, meaning these piers need only to be wide enough for the bearings to sit on. Instead the approach span is supported onto a bearing on the deck of the main span, also facilitating an expansion joint. Fig. 3.

The deck is a single depth that is carried through giving the appearance of a single structural unit throughout both spans and the approach ramps. The depth is very thin and greatly improves the elegance of the bridge. The hangers are evenly spaced which is mirrored in the spacing of the approach ramp piers. The ramps are short making use of the local topography while allowing a clearance of over 4m height for a 7.5m promenade.

The balustrade is a stressed wire system which helps keep the appearance of weightlessness, and

horizontal order of the bridge. Made from stainless steel, it should complement the structural steel and the concrete deck and the materials will signify a modern development.

Overall the bridge is light and elegant, which is evident in the balance between the depth and span of the deck and the height and depth of the two arches; which are equally in proportion to each other.

Since there are only a few structural elements the order is pleasing, and repetition is used effectively and minimally.

Due to the minimal approach and a combination of its relative newness, any imperfection no matter how small as well as decolourisation is stark against the other components. Whether this will worsen or fade will depend on time.

Overall the choice of scaled asymmetric arches, with the refinements has given the bridge a distinctive character and recognisable form, answering the client's original brief.

## 3 Foundation and Geotechnics

### 3.1 Site Investigation data

Geological information indicates the site to be underlain by superficial deposits of Made Ground overlying estuarine and marine alluvium and glacial laminated clays. The underlying solid geology below the site is identified as the Triassic Sherwood Sandstone Group Beds. No faults were identified in the vicinity of the site.

Cable percussion boreholes were drilled on both the north and south shores as well as in the river from floating pontoons. Standard penetration testing was carried out in all boreholes and cone penetration was conducted in the onshore holes. Chemical and geotechnical testing was then carried out in the laboratory.

Sandstone is encountered typically at around 20 metres below river level. Alluvium is absent on the north bank, however it reaches a maximum thickness of 18 meters at the south bank, where it is overlain by up to 4 metres of dense granular fill. Within the region of the river support, there are variable thicknesses of river deposits and transported material overlying the glacial deposits that vary from being predominantly clays to dominantly granular.

### 3.2 Foundation design

The foundation system for the bridge was driven by a number of factors. Ref. [3]. Including:
- The need for a simple and practical method of construction of the river pier over the water.
- The need to provide adequate resistance to lateral loads induced by wind and the impact load from shipping.
- The need to limit settlements on the south bank where the soils are soft.

The river pier is supported off a 2m thick 11.5 x 11.5 meter square pile cap, with a total of 16 driven 1m diameter hollow steel piles. The piles were designed with the assumption that the latter soils comprise of stiff clays with un-drained shear strength of $150kN/m^2$ over dense granular material. The stiffness and strength of these near surface soils determines the lateral behaviour of these foundations. The required vertical capacity would have been achieved by either driving into the lower dense granular tills or driving onto the sandstone.

The size of the river pier piles under the river was dependent on the construction method needed for the northern arch, even though it was approximately 6 months in between the piles being driven and the lift of the arch. The northern arch was effectively constructed in position by two pieces. The larger piece was determined by the crane's strength. This could not have been increased since it was Britain's largest crane. Therefore the size of crane needed to lift the second smaller piece was then determined. This in turn governed the size of the piles that could be driven by this crane. This crane could only drive a maximum of 1m diameter piles.

The main abutments are supported by four 500mm diameter hollow steel piles tied together with a rectangular pile cap. On the south bank, the piles had to be driven beyond the soft deposits, whereas on the north bank, adequate capacity would have been achieved within the upper glacial deposits.

Shallow pad foundations that terminate within the stiff glacial clays or the dense Made Ground are supporting the piers for the approach ramps and on the south bank these foundations were designed to resist long-term heave.

Four circular concrete legs are built off the pile cap, which remain hidden beneath the water line. Four steel legs are bolted onto the concrete legs. The taper and the inclination of the legs aid the aesthetics, and an apposing leg balances the lateral thrust generated.

### 3.3 Differential Settlement

The structural design of the suspended bridge allows for a differential settlement of any single support pier by ± 50mm with respect to the remaining two piers, according to the 'Approval in Principal' report. Ref. [3].

The structural design of the suspended bridge allows for a rotation of ± 0.25 degrees of the central pier foundation around an axis orthogonal to the plane of the arch. However, the performance specification for the piling will stipulate a maximum rotation of ± 0.1degrees and a maximum settlement of 5mm.

## 4 Idealised Structure

The design is based on a 'double arch' form. Under vertical dead loads and uniform imposed loads the arches support the loads under pure axial compression with the deck edge cables acting as horizontal ties. Under vertical patterned loads the arch acts as a two-

span bending element spanning continuously over the river pier. The bending stiffness of the south arch is used to control the deformation in the north arch. Under horizontal loads the deck and arch act as horizontal beams spanning continuously over the river pier.

## 4.1 Initial Assumptions

During the initial design of structural performance, and the estimate of strength for this investigation there were several assumptions made.

- The steel would behave entirely within the plastic range, with a yield stress of 355N/mm$^2$
- The hanger and deck cables would be made from locked coil cables, with partial safety factors applied to the specified minimum breaking strength.
- All the cables were modelled with their modified properties to match the nominal metallic cross section area of the cable, which is approximately 70% of the equivalent solid area. Equally the density would also be modelled to the solid cross sectional area and the Youngs' modulus for cables would be taken as 150kN/mm$^2$.
- The variability of stiffness for both long-term and short-term loading would have had to have been modelled.
- The longitudinal pre-stressing of the deck would allow continuous mobilisation of uncracked in plane concrete deck stiffness.
- The deflection and dynamic assessment foundation flexibilities were based on appropriate soil properties.

For this investigation, an estimate of the deck geometry gives a cross sectional area, $A$, of $885 \times 10^3$mm$^2$ and a neutral axis 150mm from the top surface of the deck, with a total depth of 430mm. The moment of inertia about the x-axis, $I_{XX}$, was estimated to be $9.61 \times 10^9$ mm$^4$, and about the y-axis, $I_{YY}$, of $2.48 \times 10^{12}$ mm$^4$.

## 5    Loading

The loading for the footbridge was assessed according to BS5400: Part 2 (1978), Specification for loads, which is implemented by the use of BD37/01: Loads for Highway Bridges.

A critical analysis of the loading will be assessed according to BS540: Part 2: 2006; representing the most recent standard as of this investigation.

The five main load cases considered are; dead load, super-imposed dead load, live load, wind loading and temperature effects. Combination of these load cases with the associated load factor, $\gamma_{fL}$, would be used to check for both limit states.

## 5.1  Dead Loading

Nominal dead loads can initially be based on the densities of the materials used. However the value calculated below, would have been checked against the loads actually used in construction, and the discrepancies accounted for.

Taking the density of concrete to be 24kN/m$^3$, this gives a predicted characteristic dead load of 21.3kN/m. The concrete deck also has down stand beams for every deck unit, and considering this additional weight would give an approximate dead load of 23kN/m.

The complete arch weighs approximately 304 Tonnes, with the north arch alone contributing 69% of this weight. Averaging over the complete length gives a characteristic dead load of 16.6kN/m.

During the design there was a global allowance of 4% for the weight of additional stiffeners to the arch, support arms and support legs. This would allow the transfer of local hanger cable forces to the arch and provide resistance against local stress effects caused by small curvatures of the arch over the central support reign.

The dead loads were designed to BD37/01.

## 5.2  Superimposed Dead load

Superimposed loads are expected to be a minimum due to the refined design. The only deck furniture is the handrail enclosure system. This would have an approximate load of 2kN/m.

In the dead load assessment for the deck units, an allowance has been made for the possible future addition of a resin bound aggregate surface on top of the deck unit as installed. The allowance is for 0.2kN/m$^2$ and corresponds to a 10mm over-layer of resin-bonded aggregate.

## 5.3  Live Loading

Since the bridge is inaccessible to vehicles due to the arch coming down in the centre of the deck then according to BS5400, the nominal load applied is 5kN/m$^2$, assuming that crowd loading is not to be expected.

However since the bridge is over 36m then under clause 7.1 of BS 5400: 2: 2006, the nominal live load can be reduced by a factor $k$.

$$k = \frac{no\,min\,al\ HA\ UDL\ for\ loaded\ length \times 10}{L + 270} \quad (1)$$

Taking the loaded length, $L$, of the bridge to be 180m

$$k = \frac{21.3 \times 10}{180 + 270}$$
$$= 0.473$$

Therefore the live load is 2.37kN/m$^2$.

Critical patch loading combinations would have been taken into account. Dynamic loading arising from the pedestrian movements would have also been taken into account.

Snow and ice loading would be small in comparison to pedestrian live loading and are unlikely to coincide. Loads from floating ice sheets and debris were conservatively calculated at 1.4MN, however this is less than the ship impact load. Wave loading on the bridge is deemed negligible due to the maximum wave height.

## 5.4 Wind Loading

The effects of wind loading on the structure were considered in accordance with BD37/01 with the drag co-efficient of the deck and the arch based on physical modelling. The force coefficients based on the tests were then combined with the predicted mode shapes and wind information for the site to determine the maximum overall wind loads for the design of the structure as a whole and the maximum wind load acting on the individual panels. The effects on wind loads of ice and rainwater on the hanger and tie cables would also have been considered as well as dynamic effects from wind. There was a possibility of galloping instability caused by wind induced responses of the northern arch. This was discounted after wind tunnel testing. Similarly, further wind tunnel testing discounted instabilities of the deck.

The design wind loads used on the bridge can be estimated using BS5400: 2:2006. Since the span of this footbridge is greater than 30m, British standards states that "consideration should be given to the effects of dynamic response due to turbulence taking due account of lateral, vertical and torsional effects".

Taking a static analytical approach however, the maximum wind gust speed can be calculated using;

$$V_d = S_g V_s \qquad (2)$$

With,

$$S_g = S_b T_g S'_h \qquad (3)$$
$$= (1.60 \times 0.99) \times 0.91 \times 1.0$$
$$= 1.44$$

Hence

$$V_d = 1.44 \times 26.4$$
$$= 38.1 \, m/s$$

This gives a dynamic pressure head, $q$, of 0.89kN/m$^2$ using Eq. (4).

$$q = 0.613 V_d^2 \qquad (4)$$

The dynamic pressure head can then be used to calculate the nominal transverse wind load, $P_t$, as well as the nominal longitudinal wind loads, $P_L$. It should be noted that the dynamic pressure head as calculated can

only be used on adverse areas, where there are relieving areas, the hourly mean wind speed, $V_r$ should be used instead of $V_d$.

### 5.4.1 Nominal Transverse Wind Load

The nominal transverse wind load is calculated considering both with and without the live loading, since the load is calculated in respect to the dynamic pressure, the solid area, and the drag coefficient; both which are dependent on effective depth of the deck. The calculation below will consider the worst-case wind loading;

$$Unloaded \ area = d = d_1 \qquad (5)$$
$$= 0.45m$$

$$Loaded \ area = d = d_3 = d_1 + d_L \qquad (6)$$
$$= 1.7m$$

This gives an effective b/d ratio of 11.3 and 3 respectively. Both give a drag coefficient, $C_D$, of less than 2.0; which is the minimum to be taken for foot and cycle track bridges and will be used in Eq. (7), which gives an indication to the wind load applied to the deck;

$$P_t = qA_t C_D \qquad (7)$$
$$= 0.80kN/m$$
$$= 5.25kM/m \, (with \, live \, loading)$$

### 5.4.2 Nominal Longitudinal wind load

Clause 5.3.4.1 of BS5400: 2:2006 states that the nominal longitudinal wind load, $P_{LS}$, is taken to be 25% of the nominal transverse wind load, hence 1.32kN/m and 50% of the nominal transverse wind load when applied with live loading, $P_{LL}$, is 2.63kN/m.

### 5.4.3 Nominal Vertical Wind Load

The effect of wind uplift and down force is also considered with respect to the British Standards. Taking the width of the deck to be 5.1m and the lift coefficient, $C_L$, to be 0.9.

$$P_v = qA_3 C_L \qquad (8)$$
$$= 4.1kN/m$$

## 5.5 Temperature

Daily and seasonal fluctuations in shade air temperature, solar radiation and re-radiation has two effects on a bridge; the effective temperature of the bridge superstructure changes which causes movement, and temperature differential across the depth of the cross section can cause load effects within the superstructure. Ref. [4].

Taking the minimum shade air temperature as -14°C gives a minimum effective bridge temperature of -9°C and a maximum shade air temperature of 33°C

gives a minimum effective bridge temperature of 33°C, according to clause 5.4.3 of BS5400 part 2. Assuming all deck units were cast at a temperature of 12°C, this would give a temperature differential, $\Delta T$, of ±12°C, which has an associated strain, $\varepsilon$, Eq. (9). The temperature co-efficient, $\alpha$, of concrete is taken to be 12 × 10–6/°C according to clause 5.4.6 of BS5400: 2:2006.

$$\varepsilon = \alpha \Delta T \qquad (9)$$
$$= 12 \times 10^{-6} \times \pm 12$$
$$= \pm 144 \mu \varepsilon$$

Assuming that steel was formed at a similar temperature to the concrete, the strain would be the same due to an identical thermal coefficient. This would cause a change in length, $\delta L$, of the north arch according to Eq. (10).

$$\partial L = \varepsilon \times L \qquad (10)$$
$$= (144 \times 10^{-6}) \times (120 \times 10^3)$$
$$= 18mm$$

This movement is facilitated by the bearings at the abutment as well as the expansion joint. However, if the movement were prevented, a stress, $\sigma$, would be induced in the deck.

$$\sigma = E\varepsilon \qquad (11)$$
$$= (30 \times 10^3) \times (144 \times 10^{-6})$$
$$= 4.32 N / mm^2$$

The second effect of temperature on the bridge is the differential temperature. This would induce a moment into the concrete deck.

## 5.6 Dynamic Loading

The dynamic response of the bridge would have been a key aspect in the design process due to the lightweight nature of the bridge. Detailed analysis was undertaken to ensure that the dynamic responses of the bridge in service conditions under wind and pedestrian loading would be below acceptable serviceability criteria for pedestrians.

The vertical and lateral acceleration responses of the bridge were limited. Under normal conditions, the threshold was set at 35 to 50 milli g peak vertically and 15 milli g peak horizontally. Under exceptional loading maximum thresholds of 70 milli g peak vertically and 20 milli g peak horizontally would be used. These thresholds correspond to accelerations that could be felt by the majority of stationary pedestrians, but should not cause concern.

Pedestrian densities of up to 1.5 persons/m$^2$ (walking) and 0.25 persons/m$^2$ (jogging) will be considered. This corresponds to a very dense crowd. Forced vibration, caused by vandalism would have also been considered, based on approximately 20 people running in phase.

The susceptibility of the bridge to 'lock-in' effect under crowd loads was assessed to ensure lock-in would not occur.

The approximate fundamental frequency, $f_0$, of the deck can be calculated using an equation under B.2.3 from Annex B of BS5400: 2:2006;

$$f_0 = \frac{C^2}{2\pi l^2}\left(\sqrt{\frac{EIg}{M}}\right) \qquad (12)$$

The configuration factor, $C$, is based on a continuous span over a fully rigid support. This would provide a simplification when considering that the hangers would provide an elastic directional support.

$$f_0 = \frac{\pi^2}{2\pi 7.5^2}\left(\sqrt{\frac{(30 \times 10^6) \times 0.00961 \times 9.81}{25}}\right)$$
$$= 9.4 Hz$$

Since greater than 5Hz, this is deemed to satisfy the vibration serviceability required by BS5400: 2: 2006. However the horizontal vibration also needs to be considered;

$$f_0 = \frac{3.55^2}{2\pi 120^2}\left(\sqrt{\frac{(30 \times 10^6) \times 2.48 \times 9.81}{25}}\right)$$
$$= 0.753 Hz$$

According to BS5400: 2: 2006, this has exceeded 1.5Hz allowed for horizontal vibration and is unsatisfactory for vibration serviceability.

Seven tuned mass dampeners (TMD) are attached to the underside of the deck, within its depth which address pedestrian induced dynamic effects. These have an associated dead load of approximately 5 tonnes, and also produce a small dynamic force on the bridge, which would have needed to be considered. The design allows for the additional self-weight of two dampeners to be added to the deck and sockets are cast into the underside of the deck, allowing for movement of the current dampeners and easy instalment of any additional TMD. These sockets can then be used for future maintenance.

## 5.7 Other Loading Considerations

Several additional loading considerations had to be taken account of during the initial design process.

The bridge piers and legs were designed for ship impact loading derived from an energy approach with an allowance for hydrodynamic effects. The worst case impact load was taken to be 2MN perpendicular to the bridge and 1MN from other directions. The bridge superstructure was not designed to resist a ship impact as current and predicted navigational usage of the river means that the risk of a significant impact is negligible.

The design has allowed single arch tie cables to be replaced but the rest of the bridge must be closed. Similarly, the deck was designed to accommodate a span of 15m between the hangers either side of a lost hanger. There would be an associated increase in local deformation.

The cables may need re-tensioning to prevent creep and this would have been allowed for.

The bridge deck was designed for a characteristic point load of 10kN over a 150 x 150mm square anywhere on the walking surface for inspection and maintenance equipment. Anchorages are provided in the edge beam of the deck soffit, each designed for a pullout load of 10kN, allowing for inspection purposes.

# 6    Strength

Using the estimated loading values calculated as well as the initial assumptions made, an approximation of the strength of the bridge can be made.

BS5400: 2: 2006 clause 4.1.2 states "Nominal loads shall be multiplied by the approximate value of $\gamma_{fL}$ to derive the design load to be used in the calculations" and hence will be adopted here.

## 6.1  Deck

To calculate the bending in the deck, the deck is considered to span continuously over the river pier, and the hangers provide elastic support. However for approximation of the strength, the cables are assumed to provide a rigid support, hence the calculation can be considered to be that of a continuous beam

$$M = \frac{w_l l^2}{10} \qquad (13)$$

Considering only dead, superimposed dead, and live loading gives;

$$w_l = ( 23 \times 1.15 ) + ( 2 \times 1.20 ) \qquad (14)$$
$$+ (( 0.2 \times 5.1 ) \times 1.75 ) + (( 2.37 \times 5.1 ) \times 1.5 )$$
$$= 48.8 kN / m$$

Hence;

$$M = \frac{48.8 \times 7.5^2}{10} \qquad (15)$$
$$= 274 kNm$$

Giving a maximum stress in the tension face of

$$\sigma_T = \frac{My}{I} \qquad (16)$$
$$= \frac{274 \times 10^6 \times 280}{9.61 \times 10^9}$$
$$= 7.98 N / mm^2$$

Under the assumed conditions this stress would cause cracking in the top surface of the deck. The north span cables were pre-stressed to provide 2100kN under dead load alone; a pre-compression of the north span of approximately 2.4 N/mm², reducing the tension stress. The south cables provided 1550kN of pre-compression, under dead load alone; a stress of 1.75 N/mm² to the deck. Stainless steel reinforcement is used within the deck.

The stress in the compression face can equally be calculated to be 4.28N/mm². The deck concrete is grade C60, giving a strength of 60N/mm²; hence compression is unexpected to be a major concern, even combined with the pre-compression and other loading conditions.

There are two pre-stressed bearings either side of the main bearing at the abutments with the main bearing transferring the vertical load from the arch into the abutment. The pre-stressing of the other bearings removes the tension from the moment generated by the wind load.

## 6.2  Hangers

An estimate of the tension in the hangers can be calculated assuming two hangers on either side of the deck share the design load of 48.8kN/m.

$$R = \frac{wl}{2} \qquad (17)$$
$$= 91.5 kN$$

Considering the maximum inclination of the hangers gives an approximate tension of 100kN, hence

$$\sigma_T = \frac{F}{A} \qquad (18)$$
$$= \frac{100 \times 10^3}{(( \frac{\pi}{4} \times 30^2 ) \times 0.7 )}$$
$$= 202 N / mm^2$$

The steel work has a design strength of 355N/mm², and the system was designed such that a single hanger could be removed, causing an increase in the load.

## 6.3  Arch

Both arch geometries were 'form-found' being optimised to take dead and superimposed dead load in axial compression alone.

It is assumed that through the iterative process a parabolic arch would have been formed. An estimated equation for the arches can be calculated based on their final geometries, which have an approximate span to height ratio of 4:1.

The North arch geometry is given by;

$$y = -\frac{x^2}{120} + x \qquad (19)$$

The South arch geometry is given by;

$$y = -\frac{x^2}{60} + x \qquad (20)$$

The reflex piece over the river pier that is crucial for transferring moments across the arches was differentiated several times to maximise the smoothing of the rate of change of the curvature.

An approximation of the thrust, $H$, by the arches can be calculated using Eq. (21).

$$H = \frac{w_2 l^2}{8f} \qquad (21)$$

Considering the arch has the dead load applied along its length, the total load is that calculated in Eq. (14) with the additional weight of the steel.

$$w_2 = 48.8 + (16.6 \times 1.05)$$
$$= 66.3 kN / m$$

Taking the height of the arch from the deck to be 30m for the northern arch gives;

$$H = \frac{66.3 \times 120^2}{8 \times 30}$$
$$= 3980 kN$$

The vertical reaction can be simply calculated to be 3980kN by Eq. (17).

Therefore the horizontal thrust is equal to the vertical reaction, as is expected from the differential of Eq. (19) which gives a gradient of 1 at the support and as such the maximum axial force taken by the arch is approximately 5630kN.

The codes do not cover buckling in necessary detail for the arch structure. The analysis used in the actual design determines all critical buckling modes and shapes up to $\lambda_{crit}$ values of 10. Force, moments and stress effects that are due to buckling were derived from first principals, which accounted for the geometry as well as the stress. An allowance of L/500, equivalent to 250mm, accounts for the imperfections during fabrication and steel, and the effects of residual stress in welds is to BS5400.

The curvature of the top and bottom flanges of the reflex section over the river pier is outside of the limits stated by clause 9.3.5 of BS5400: 3:2000. The Merrison Reports: Interim Deign And Workmanship Rules (IDWR), which formed the basis of BS5400, allowed a method of assessment for the curvatures used.

Assuming the thrust is taken and shared by all four of the 90mm diameter deck edge tie cables, Eq. (18) would give a tension stress of 223N/mm²; similarly the south arch generates a stress in each 60mm diameter cable of 214N/mm². These cables are designed for an additional post-tensioning stress that in turn would pre-stress the deck. Similar to the hangers, the edge cables were designed for one to be replaced.

Each of the central river nodes would experience an approximate vertical load of 2990kN from dead load, superimposed dead load, and live loading. However, since the river pier acts as a portal frame, there would be a considerable additional force due to the moments generated from lateral forces, which have not been considered here. The central pier was modelled using finite elements to help design the node.

## 7    Serviceability

As the bridge deflects downwards under live loading, an additional tension load is put into the hangers. This is transferred to the arch that would similarly deflect downwards. The arch now being shallower increases the tension of the tie cables. This decreases the sag in the deck edge cables having the effect of reducing the deflection of the deck. Hence a full dynamic model was required to ensure that the bridge met serviceability requirements.

## 8    Construction

Ease of construction was a major driver for the bridge design in order to ensure construction was both straightforward and fast.

The contractor designed the actual construction process, and the process was made easier by being able to have a site compound on the south shore as well as the north. The steel fabricators were conveniently located in Darlington, which is about a 20 minutes drive away.

A temporary jetty was constructed on piles that extended out from the south shore allowing access for construction workers and machinery to the river pier.

A sheet pile cofferdam was used in the construction of the river pier allowing foundations and support legs to be constructed. The concrete legs below the water line were constructed by using 3m diameter manhole rings that acted as a permanent formwork. The legs were then positioned and welded to the central node. This central node was machined out of two solid blocks of steel that were welded together with a 30mm fillet weld all the way around.

Temporary sheds were set up on site to allow for deck casting using 3 prefabricated steel moulds; 2 standard with 1 special. The first section of the deck was then lifted and positioned over the river pier, as access later in the construction would be compromised. Steel false work was constructed off of the cofferdam, which would be used to initially support the arches.

The arches were fabricated within the steel factory from 3D models to optimise the pattern cutting of sheets. Factory controlled conditions ensured a high level of quality and reduced onsite erection, welding and painting. The fabrication process ran in parallel to the initial onsite work. The end reaction frame comes down to a pin connection, and so movement should be

limited. This was achieved through using three steel plates that were bored together along with the end splayed arch, rather than boring separately ensuring a very high tolerance.

The south arch is constructed of four separate pieces of fabricated steel that are welded together on site and lifted into place by crane. The first piece positioned was the central reversed curvature piece and this was initially supported off of the steel false work. The continuation of this piece to both the north and south were then positioned, welded to the first piece and still supported off the false work. The final piece of the arch was then lifted into place. Fig 4.

The southern section of the north arch was then welded onto the reflex portion and tied back to the south arch. The final and largest piece of the arch arrived on site in four separate parts and welded together on site. Britain's largest mobile crane was used for a 90m lift of the final and largest section of the north arch. Fig 4.

The proposed construction method involved using a series of temporary trestle piers. This would have allowed smaller arch sections to be lifted, but would require more onsite work and have been more costly.

The bearings were pre-stressed by jacking the pier and deck end reaction frame apart, then installing the bearings. This was released putting a load into the bearing; removing the need for costly tension bearings.

The arch was installed with the hangers attached, and the main tie cables were then fed from the respective shores over to the river pier and attached to the central node. The deck units were then installed individually from the central pier working towards the shores. They were hung off the hangers and were temporarily attached on the topside using the boltholes used for the handrail stanchions. This allowed the deck units to be held apart, allowing the cast-in steel plates to be aligned. Adhesive was applied to the end of the deck, and then they were brought together, allowing the adhesive to set. The steel plates on the underside of the deck units were then welded together to achieve the full tension capacity. The deck units at this stage were still not attached to the edge tie cables.

After the complete deck was installed the edge cables were connected, along with the balustrades, and finally the lighting scheme. The TMD were then installed and tested using accelerometers and several men running and lunging in phase and at set speeds. They were initially tested locked without the dampening effect but with the added mass, and then with the dampening effect unlocked. Only one of the dampeners required tuning.



**Figure 4:** Lifting of Northern Arch

## 9    Maintenance Requirements

There have been features added in to the design of the bridge to enable both inspection and maintenance during its 120 year design life.

### 9.1    Inspection

The Department of Transports guidelines for bridge inspection indicates there are three types of inspection that are normally carried out. Every two years a general inspection is carried out, which consists of visual observations. Principal inspection is carried out every 6 years and is more comprehensive than a general inspection with observations being made at less than a meter. Special inspection is carried out as required.

However there would be a requirement that the bridge would need specific inspection. The structural arch has been designed with abseil anchor access points, for the principal inspection. These should be checked to take 15kN for 15 seconds before each use. The corrosion system should also be checked on the tie cables as well as the arch. The tie cables should be checked for the tension, relaxation and creep. The hangers, dampeners, bearings and connections should be checked for their performance. Ref. [3].

The piers and abutments, including the river pier would specifically require chloride sampling, a check of cover depth and delamination soundings, to locate areas of steel concrete delamination, with the river pier requiring the removal of accumulated debris following a major flood event. Handrail, stanchions enclosure panels and wires would all be inspected for corrosion, and resistance to horizontal force, with a requirement of a possible load test.

### 9.2    Main phases of deterioration

There are several phases of deterioration that is to be expected. Primary deterioration would be expected to include progressive break down of the protective system such as the paint, waterproofing membranes and expansion joints. Following this, secondary deterioration would occur including physical deterioration of bridge elements that would lead to a reduction of life, which would commence after the protection is lost. Significant damage could then occur with associated possible hazards to users. Substantial damage could occur which may affect the strength of the bridge requiring a special inspection, and a subsequent assessment of load carrying capacity.

### 9.3    Expected Maintenance

The footbridge was designed to be low-maintenance throughout its design life. However, due to certain technical limitations, there would be a small number of scheduled maintenance activities.

There would be a requirement for reapplication of the corrosion protection system every 10 years for

minor maintenance and every 20 years for major maintenance. Further tensioning of the cable would occur almost every year initially, and then every 15 years thereafter. The rip-rap may be required to be repositioned following a special inspection after a major flood event. Similarly, debris around the river pier would also be required to be removed following a major flood event. Ref. [3].

Since there is an included lighting scheme, this has associated maintenance issues. The estimated lifetime of the bulbs is approximately 100000 hours, and the encasing, bulb or cables may need replacing periodically.

### 9.4    Access for Inspection and maintenance

There is an obvious requirement for access for inspection and maintenance. Depending on the method of access, pedestrian, cycle and river management systems may be required.

Access to the arch, hangers, support arms and support legs is assumed to be undertaken by roped access, and as such anchorage points would be located along the steel arch.

Access to the underside of the deck, arch tie cables and arch lighting, service ducts hangers, bearings and support legs would be from ground or river, or possibly under slung scaffolding or from a cradle. Anchorages are provided in the edge beam of the deck soffit, by using sockets originally cast for the TMD. It is assumed that access to the river and the scour protection system will be by a boat. The walking surface, deck lighting and footway enclosure can be accessed from the deck itself. Equipment loading has been allowed for in the design.

### 10    Vandalism

Since the bridge has not been officially opened to the public as of this investigation, then vandalism concerns can only be considered, but the design of the bridge would have taken into account possible acts of vandalism.

The three main types of vandalism that are of concern would be damage (permanent deformation, surface scoring, cutting etc), graffiti and theft.

However, vandalism is largely unexpected. The main deterrent to vandalism is the actual use of the bridge and people in the nearby area. Considering the location of the bridge, and its intended use, there would be little chance for vandalism. However, the North Shore development has not progressed as expected and as such there is an expected increase in the possibility of vandalism. The second most effective deterrent is to continually remove the evidence of vandalism as soon after it occurs. This is made easier for the clients, due to the stripped back approach to the design, limiting the possibility of vandalism in the first place.

The bridge would be extremely difficult to damage and has been designed to allow for the loss of a single hanger cable or single horizontal edge cable if they were to be cut though.

The stressed wire enclosure system is more likely to attract vandalism, however damage is unlikely due to its inherent strength and flexibility. 8mm wire has been specified which effectively means that bolt cutters would be required to cut through the wire.

Graffiti includes painting, scratching and sticking of labels. The ends of the arches could be vandalised through graffiti, but this section can be relatively easily repainted. Since the hand rail and other fixtures are made out of unpainted stainless steel, they are easier to keep clean compared to painted mild steel and they have a comparable resistance to scratching. Using stressed wire reduces the effective exposed area, helping to reduce graffiti.

Theft of the stainless steel due to the market price is probably a greater concern. This is especially true with long lengths of wire that would normally be relatively easily pulled through, however it is limited to 7.5m.

### 11    Conclusion

The North Shore Footbridge fundamentally is deemed a success fulfilling the client's initial brief. The design reflects modern bridge design perfectly; an efficient and elegant structure with each component of the bridge being integral to its function.

Suggesting changes to the design would be to compromise the original reasons behind each decision, whether based on structural performance, ground conditions, construction process or the aesthetic qualities.

The true success of the bridge is now not dependent on its design but its future use.

### 12    Acknowledgements

### References

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