A CRITICAL ANALYSIS OF THE FORTH ROAD BRIDGE, QUEENSFERRY, SCOTLAND

J. R. Penistone

Abstract: This conference paper presents a critical analysis of the Forth Road Bridge, addressing factors such as aesthetics, strength, construction methods and geotechnical design, in addition to discussing the bridge’s durability issues and future. Loading and strength analyses are carried out to BS5400-2, issued in 2006, providing a pertinent consideration of the impact of the increased vehicle loads the bridge has been subject to since its construction.

Keywords: Forth, Suspension bridge, Steel, Corrosion, Deck-truss

1 Introduction

Figure 1: Forth road bridge

A road link across the River Forth, connecting Edinburgh to the cities to the north such as Perth, had been proposed many years before the completion of the Forth Road Bridge in 1964, which replaced an existing ferry service at the site. At various times, a tunnel and an earlier suspension bridge design (in 1818) were also proposed [1]. The economic and industrial requirements for the bridge were very high, given the bridge’s location at the nexus of Scotland.

Crossing the Forth presented a number of technical challenges, including the requirement to cross a distance of approximately 1.8km, high wind forces and an extreme range of temperatures throughout the year.

The final bridge, designed by architect Sir Giles Gilbert Scott and engineers Messrs Mott, Hay and Anderson was the first long-span suspension bridge constructed in Europe. Following on in certain aspects from American precedents, the bridge is all-steel, with a deck bracing truss and vertical hangers. Steel twin box girder sections, supported on reinforced concrete piers are used on the approach viaducts [2]. The bridge carries two lanes of vehicle traffic in each direction, with pedestrian walkways cantilevering from the main deck on each side.

Although the use of deck trusses was about to become technically obsolete, the bridge encompasses a number of advances over its predecessors, producing a distinctive and landmark structure.

2 Aesthetics

Discussion of the bridge aesthetics can be categorised according to the rules outlined by Fritz Leonhardt in his work “Bridges: Aesthetics and Design”. While the aesthetical criticism of any bridge is highly subjective, this work contains a number of convenient reference areas that can be analysed sequentially.

The first requirement is that the structural functioning of the bridge is clearly displayed. This is achieved rather well on the Forth Road Bridge. The open nature of the bridge gives it an almost skeletal appearance, with the elements on display fulfilling a structural role. The structural form of the suspension bridge is also intuitive to understand, with the cables clearly being loaded in tension to support the deck loads. Furthermore the anchorage cables are visible externally, fully displaying the structure’s load path. The use of diagonal bracing on the tower also highlights the role they play in the bridge’s stability.

The reading of the functionality of the bridge is complicated by the presence of the deck bracing truss. Its presence and depth overemphasises its contribution to vertical load resistance, when it primarily stabilises the deck in wind loading.

The individual deck-truss members also reduce the order of the bridge. When viewed at a distance from certain orientations, the elements appear quite well-ordered, but at closer distances, the truss appears dense and complex, giving a cluttered appearance. The longitudinal vertical and horizontal trusses also appear...
rather boxy, as shown in Fig. 2, making the bridge appear less elegant. The order of the approach viaduct supports is also reduced as the supports overlap when viewed from a shallow angle. An obvious option would have been to increase the spacing of the supports (currently about 40m), however it may not have been possible to achieve a longer span structurally or lift longer sections into place. More positively, the use of vertical hangers precludes the problem of intersecting cables when viewed obliquely. 

![Figure 2: Deck underside](image)

Good proportion is also vital. Viewed perpendicular to the deck, the bridge is very well-proportioned, featuring side spans that are exactly equal. This was done despite the different founding conditions present on each side, which suggests it was a key aim of the architect.

However, the bridge is less well proportioned in closer detail. For example, the tower bracing elements are far more slender than the tower columns they are attached to, which, combined with the square plan section of the columns, gives the columns a rather crude appearance.

This problem is an example of where the design could have been refined. The deck truss bracing is another such area. Indeed, the bridge was the last bridge built in the United Kingdom to feature such bracing, being built two years before aerodynamically stable deck sections were introduced on the First Severn Crossing, [3]. While the same consultant designed both bridges, as the Forth Road Bridge was the first long-span suspension bridge to be constructed in the United Kingdom, a truss deck was probably chosen out of conservativeness.

However, although the truss has been described as boxy, compared to previous structures the truss is relatively restrained in the size of its members.

The cross bracing used on the support towers also increases the slenderness of the bridge, even if it adversely affects the bridge’s order. With the use of narrower tower sections, the bracing would have seemed far more suited and given a more elegant structure.

Integration with the environment is an important concern for any bridge, not least one the scale of the Forth Road Bridge. Considering the local landscape only, the suspension bridge form of the bridge is appropriate to the bridge’s environment, fitting in well with the open estuary surrounding the bridge and allowing the bridge to stand out well.

The Forth Rail Bridge is also located approximately a kilometre east of the bridge. Given this bridge’s iconic status, it was vital that the road bridge did not compromise its impact. In many ways, this is achieved by the differences between the bridges. The road bridge is the rail bridge’s polar opposite - in the form of transport it carries, its structural form and even its colour. This celebrates the features of each structure without letting one dominate the other. Each bridge also typifies the era in which it was built by representing the cutting edge structural form and form of transport dominant at the time. It is therefore interesting that a new bridge proposed at the site is a cable stayed bridge.

Integration with the surrounding environment is helped somewhat by the grey-blue colour of the steel elements of the bridge. This allows the bridge to compliment the river below it, and gives a particularly good look in sunny weather, as shown to great effect in Fig. 2.

However on the downside, this colour, combined with the use of steel elements, gives a rather cold and dull surface texture when viewed close up. This problem is exacerbated in dull weather, and gives the bridge a rather lifeless appearance. The finish also appears to have aged badly over its relatively short lifespan, resulting in a less than pristine appearance.

Each of these attributes reduces the character of the bridge, with the cold look of the painted steel making the bridge appear rather lifeless.

Complexity is another important characteristic of a bridge. Once again, the deck truss is dominant, being the most complex element on the bridge. From a structural point of view, this complexity adds interest to the bridge, but it also complicates the relatively straightforward appearance of the rest of the bridge and gives a slightly unbalanced appearance.

Finally, given the bridge’s massive scale and height compared to surroundings, the amount that nature can be incorporated with the bridge is rather limited. However, this is another area where the use of colour is helpful, allowing the bridge to blend in with the river well.

**Summary of Aesthetics**

Judged in comparison to later bridges, the Forth Road Bridge performs poorly in certain respects, with a somewhat featureless and cold appearance. The truss deck and towers also make it appear somewhat unrefined. However, this criticism is rather unfair, as the structural form and the material used were limited by the technical knowledge that was available at the time of its construction.
Judged on its individual merits, the bridge can be viewed as a good representation of its time and provides a fitting contrast to the iconic rail bridge it shares the Forth with.

3 Structural Form

The structural form of the bridge follows the American style of suspension bridge construction prevalent when the bridge was constructed, with a deck truss providing rigidity against wind loading. Dead, traffic and wind loads from the deck are taken as tension within the suspender cables, which load the main cables uniformly in tension along the bridge deck’s length. Accordingly, the main cables deflect into a parabolic shape.

The cables rest on the towers with saddles, which transfer compression vertically to the towers. Differences in load between the spans are accommodated by sliding of the cable over the saddle. Tension from the cables is then transferred in the anchorage blocks. Finally, the foundations are moment resisting, providing stability to the structure.

4 Construction

As with any suspension bridge, the Forth Road Bridge was constructed with suspended construction. In this case, after completion of the piers and towers, and the spinning of the main cables; the deck section was bolted together from rectangular hollow sections outwards from the support towers.

The complexity of the truss joints made this a time consuming process. Working at precarious heights was also required, although safety nets were used in this part of the construction process. The use of prefabricated deck sections would have therefore reduced construction time and improved safety.

Safety was also a concern during the construction of the main cables. This required the construction of a walkway between the main cables, which initially did not have safety fencing to protect workers from falling over 150m into the river below.

5 Foundations and Geotechnics

The bridge towers are supported on piers within the river channel. As shown in Fig. 4, the geological conditions vary greatly across the river and required quite different construction methods to be used for each pier.

![Figure 4: Geological section through Forth](image)

The construction of the south side proved much more difficult, as the bedrock was much deeper on this side. The south pier was constructed with the use of 2 cofferdams, formed from driven piles around a circular steel frame, joined in a figure of 8 [4]. The cofferdam was then drained and a steel caisson constructed inside each semi circle. Each caisson was progressively excavated until it sunk under its own self-weight, before being filled with concrete.

On the north side, where the bedrock was practically at the surface, the pier was much more straightforward to construct, consisting of piles driven straight into the bedrock, on which a rectangular steel frame was constructed and encased in concrete.

The anchorages are the final component of the structure’s geotechnical system. The main suspension cables split into 114 strands [4] that are supported within strand shoes on the exterior of the anchorage. This distributes the load evenly to internal pretensioned cables 76m long [4], of which there are present per anchorage, transferring the loads to the anchorage blocks as a shear stress.

6 Loading

Loading is calculated to the latest revision of BS5400 part 2. While the bridge was designed to an older standard (BS 153 [2]), it is very relevant to analyse the bridge’s performance to the current code, considering the bridge’s durability problems and the increase in maximum HGV vehicle size from 24 to 44 tonnes since 1964 [6].

6.1 Load Combinations

BS5400-2, clause 4.4 specifies 5 mandatory load cases:

<table>
<thead>
<tr>
<th>Table 1: Load combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loadcase</td>
</tr>
<tr>
<td>LC 1</td>
</tr>
<tr>
<td>LC 2</td>
</tr>
<tr>
<td>LC 3</td>
</tr>
</tbody>
</table>
temporary erection loads (if under consideration)

LC 4 Permanent loads + secondary live loads + primary live loads. Secondary live loads include skidding, centrifugal, longitudinal and collision loads [3] and may be considered independently of the other loads ([5], clause 4.4.4)

LC 5 Permanent loads + bearing friction loads

Design loads are obtained by factoring the nominal loads with two partial factors. $\gamma_f$ and $\gamma_{fl}$ are the partial load factor (dependant on materials, load case and limit state) and an analysis factor respectively [3]. The partial load factors are given in Ref [5], table 1. As the structural elements of the bridge are steel, $\gamma_{fl3} = 1.10$ at ULS and $1.00$ at SLS [3].

6.2 Permanent loads

The dead weight of the deck truss was estimated by scaling the approximate lengths and depths of the truss elements from Fig. 5.

![Figure 5- Deck truss section [2]](image)

This estimation assumed each element is a rectangular hollow section with a 30mm wall thickness (assumed by scaling from a photograph). In reality, this thickness will not be constant, and certain members, for example the diagonal members of the longitudinal truss, are not solid along their length.

The deck construction also differs between each span. The main span consists of a 12mm thick steel plate deck, supported on transverse I-sections approximately 190mm deep. These elements are supported by 5 longitudinal support beams, assumed to be 600mm x 600mm and again 30mm thick. The side span decks consist of a 203mm reinforced concrete slab [2].

<table>
<thead>
<tr>
<th>Element</th>
<th>Load</th>
<th>ULS Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Truss</td>
<td>85.9KN/m</td>
<td>99.2 KN/m</td>
</tr>
<tr>
<td>Longitudinal Beams (main span)</td>
<td>26.8KN/m</td>
<td>31.0 KN/m</td>
</tr>
<tr>
<td>Steel deck construction &amp; walkways (main span)</td>
<td>27.4KN/m</td>
<td>31.6 KN/m</td>
</tr>
<tr>
<td>R.C. deck &amp; steel walkways (side spans)</td>
<td>76.428KN/m</td>
<td>96.1KN/m</td>
</tr>
</tbody>
</table>

Total (main span) | 140.1KN/m | 161.8KN/m |
Total (side span) | 168.3KN/m | 226.3KN/m |

For the bridge length of 1822m [7], the total mass of the truss is 15651 tonnes. Given that 39,619 tonnes of steel was used in the bridge [7] the assumed steel member sizes seem reasonable, i.e. 39.5% of the total steel weight is within the truss. Furthermore, a study [8] independently calculated a main span dead load of 158KN/m and a side span load of 210KN/m on, again close to the values obtained.

6.3 Superimposed Dead Loads

The bridge deck is surfaced with 38mm of mastic on a waterproofing layer [2], in addition to parapets and lighting. Assumed values are shown below.

<table>
<thead>
<tr>
<th>Element</th>
<th>Nominal Load</th>
<th>Load at ULS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mastic and waterproofing (i.e. surfacing)</td>
<td>19.0 + 6.67KN/m = 25.7KN/m</td>
<td>49.5KN/m</td>
</tr>
<tr>
<td>Parapets</td>
<td>7.5KN/m</td>
<td>9.9KN/m</td>
</tr>
<tr>
<td>Lighting</td>
<td>0.145KN/m</td>
<td>0.191KN/m</td>
</tr>
</tbody>
</table>

Vehicle loading on a bridge is represented with HA and HB loading, representing normal vehicle loading and exceptionally large vehicle loading respectively.

6.4 HA Loading

The HA loading is given by Eq. (1)

$$w = 36\left(\frac{1}{l}\right)^{0.1}. \quad (1) \quad ([5], \text{Clause 6.2.1})$$

The loaded length $l$ is the adverse area of the bridge’s influence line ([5], table 13). As the loading of the side spans will create a relieving effect, the loaded length will be assumed to be the main span length (1006m).

This gives a nominal load of 18.036KN/m per notional lane. Additionally, a knife-edge load of 120KN per notional lane must also be applied ([5], clause 6.2.2). These loads factor to 2574KN (load case 1).

6.5 HB loading

The HB loading design vehicle is a 16 wheel, 4 axle HGV, weighing 1800KN for full HB loading, which is 112.5 KN per wheel [3]. This vehicle is positioned on the bridge in order to find the most onerous effect, where the inner axle spacing is 6, 11, 16, 21 or 26m depending on which produce the severest effect ([5], Fig. 12). The most onerous factored HB load is 2574KN (load case 1).
6.6 HA & HB loading combinations

Ref. [5], figure 13 defines 3 configurations for combining HA and HB loading. These cases are that the HB vehicle is within the outermost notional lane, that the vehicle is straddling the outer notional lanes and that the vehicle is straddling the inner notional lanes with no loading applied to the opposite carriage way.

These load cases require HA lane factors to be applied, as defined in Ref. [5], table 14. The load factors and corresponding HA loadings per metre length of notional lane are given below.

Table 4: HA loading factors for a 1006m loaded length

<table>
<thead>
<tr>
<th>Loading factor</th>
<th>Corresponding HA load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta_1 = 1.0$</td>
<td>18.036KN/m</td>
</tr>
<tr>
<td>$\beta_2 = 0.67$</td>
<td>12.084KN/m</td>
</tr>
<tr>
<td>$\beta_3 = 0.6$</td>
<td>10.822KN/m</td>
</tr>
<tr>
<td>$\beta_n = 0.6$</td>
<td>10.822KN/m</td>
</tr>
</tbody>
</table>

6.7 Pedestrian Loading

The pedestrian live load is defined by Eq. (2)

$$w_{\text{pedestrian}} = 5\text{KN/m}^2 \times \left(\frac{\text{No min al HA} \times 10}{L + 270}\right), \quad (2)$$

where $L$ is the loaded length (1006m). Hence:

$$w_{\text{pedestrian}} = 0.707\text{KN/m}^2 = 1.061\text{KN/m}^2 \quad \text{(factored)}$$

7 Wind

The performance of the bridge deck was initially verified with wind tunnel testing [4]. The following analysis is carried out to Ref. [5] section 5.3, despite the fact that the bridge exceeds the 200m length limit imposed by the standard ([5], clause 5.3.1), and the fact that the deck section requires wind tunnel analysis ([5], fig. 4).

This analysis gives an approximate indication of the wind loads acting on the structure, and does not consider effects such as turbulence which would be considered in a more rigorous analysis.

3 components of wind load must be considered - the horizontal wind force acting perpendicular to the deck, the vertical force acting perpendicular to the deck and the horizontal force parallel to the deck.

The maximum wind gust speed, $V_d$, is found from Eq. (3)

$$V_d = S_g V_s, \quad (3) \quad ([5], \text{Clause 5.3.2.1})$$

where the gust factor, $S_g$, and the site mean hourly wind speed, $V_s$, are defined by Eqs. (4, 5):

$$V_s = V_b S_p S_a S_d$$

and

$$S_g = S_{b'} K_f S_h T_g$$

Table 5: Wind loading values

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_b$: Basic hourly mean wind speed</td>
<td>23.5m/s</td>
<td>5.3.2.2.1</td>
</tr>
<tr>
<td>$S_p$: Probability factor</td>
<td>1.05</td>
<td>5.3.2.2.2</td>
</tr>
<tr>
<td>$S_a$: Altitude factor</td>
<td>1.05</td>
<td>5.3.2.2.3</td>
</tr>
<tr>
<td>$S_d$: Direction factor</td>
<td>1.00</td>
<td>5.3.2.4</td>
</tr>
<tr>
<td>$S_{b'}$: Bridge and terrain factor</td>
<td>1.73</td>
<td>5.3.2.3.1</td>
</tr>
<tr>
<td>$K_f$: Fetch correction factor</td>
<td>1.00</td>
<td>5.3.2.3.1</td>
</tr>
<tr>
<td>$T_g$: Town reduction factor</td>
<td>1.00</td>
<td>5.3.2.3.2</td>
</tr>
<tr>
<td>$S_h$: Topography factor</td>
<td>1.00</td>
<td>5.3.2.3.3</td>
</tr>
</tbody>
</table>

Hence, $V_d = 44.823 m / s$

The horizontal wind load normal to the deck is found from Eq. (6), with values given in table 6:

$$P_t = q A_t C_d$$

(6), ([5], Clause 5.3.3)

Table 6: Horizontal wind load factors

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q$</td>
<td>Dynamic Pressure Head</td>
<td>$1.232\text{KN/m}^2$</td>
</tr>
<tr>
<td>$A_t$</td>
<td>Solid projected area of the deck, accounting for a 2.5m traffic height ([5], fig.4)</td>
<td>10329.9m$^2$</td>
</tr>
<tr>
<td>$C_d$</td>
<td>Drag coefficient ([5], table 6)</td>
<td>1.7</td>
</tr>
</tbody>
</table>

*Assuming a full truss superstructure ([5], 5.3.3.1.4b)

Hence,

$$P_t = 1.232 \times 10329.9 \times 1.7$$

$P_t = 21634.9 \text{KN}$ Along full bridge length.

The force on the leeward longitudinal truss can be found by applying a shielding factor to the load already

Figure 6: Wind actions on the deck
calculated. For a spacing ratio between the trusses of 2.667, a shielding ratio of 0.75 is obtained. Hence, the total force on the leeward truss is

\[ P_l = 21634.9 \times 0.75 = 16226.2 \text{KN} \text{.} \]

The vertical wind load acting on the structure is given by:

\[ P_v = q.A_3.C_l. \quad ([5], \text{Clause 5.3.5}) \]

<table>
<thead>
<tr>
<th>Table 7: Vertical wind load factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Symbol</td>
</tr>
<tr>
<td>q</td>
</tr>
<tr>
<td>A₃</td>
</tr>
<tr>
<td>C₇</td>
</tr>
</tbody>
</table>

Accordingly,

\[ P_v \Rightarrow 461675\text{KN} \text{ along the full length of the bridge.} \]

The longitudinal wind load Pₗ consists of the load acting on the superstructure Pₗₘ and that acting on the live load Pₗₗ (BS5400-2, clause 5.3.4), which are defined by:

\[ Pₗₘ = 0.5q.A_1.C_D, \quad ([5], \text{Clause 5.3.4.2}) \]

\[ Pₗₗ = 0.5q.A_1.C_D. \quad ([5], \text{Clause 5.3.4.3}) \]

where the parameters are as defined previously, hence:

\[ Pₗₘ = 0.5 \times 1.232 \times 10329.9 \times 1.7 \Rightarrow 10817.5\text{KN} \text{.} \]

\[ Pₗₗ = 0.5q.A_1.C_D \Rightarrow 2246.4\text{KN} \]

Here, A₁ is the area due to the live load along the bridge. It is taken as the main span length multiplied by the live load depth. C_D is equal to 1.45 (BS5400-2, 5.3.4.3).

Hence

\[ P_{LL} = 0.5 \times 1.232 \times 1006 \times 2.5 \times 1.45 \Rightarrow 2246.4\text{KN} \]

The worst case is then the sum of these loads (5.3.4):

\[ P_{LL} = 10817.5 + 2246.5 = 13064\text{KN} \]

8 Strength
8.1 Cable strength

The mid span horizontal force H in the main cables can be found with Eq. (7), using the values in table 8.

\[ H = \frac{\omega l^2}{8f}. \quad (7) \]

Table 8: Cable loading values

<table>
<thead>
<tr>
<th>Parameter Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ω: Load/ metre length</td>
</tr>
<tr>
<td>l: Main span length</td>
</tr>
<tr>
<td>f: Cable sag</td>
</tr>
</tbody>
</table>

The vehicle load was found by applying full HA loading to all four notional lanes. While this does not correspond to the load cases specified, it allows Eq. (7) to be used and is more onerous than the HA & HB combinations specified by the standard.

Solving Eq. (7), \( H = 241.2\text{MN} \).

Each cable consists of 11618 steel strands 5mm diameter [7], hence:

\[ \text{Area} = 11618 \times \pi (2.5^2) = 2.281 \times 10^5 \text{mm}^2. \]

This gives a cable stress of:

\[ \sigma = \frac{F}{A} = \frac{241.2 \times 10^6}{2.281 \times 10^5} = 1057.4 \text{N} / \text{mm}^2. \quad (8) \]

Given that the cable strength at design was in the range of 1544 and 1774 N/mm², [8] the cable is not overstressed, based the loading assumptions stated. However, the bridge cables have undergone corrosion, reducing their strength. This consideration is discussed in more detail in the durability section.

8.2 Tower strength

The compression in the towers can be found by resolving the free body diagram shown in Fig. 7. In order to prevent translation of the cable across the tower saddles, the horizontal component of the cable tension forces must be equal between the main and side span.

\[ V \times 408 = 241.1 \times 10^3 \times 93.4 + 227.6 \left( \frac{408^2}{2} \right) \Rightarrow V = 78.4\text{MN} \text{.} \]

Given that the tower box sections are 7.25m by 5.486m, 25mm thick [9], the plan area of each section is 0.635m². Solving Eq. (8), the stress in each tower is 147N.mm², which is well below yield for 355 steel.
Euler buckling load can be seen to be prevented with Eq. (9), as the I value of the tower is 3.36m$^4$ and, assuming the tower is a pinned-fixed strut with an effective length,

$$l' = 0.85 \times 156 = 132.6m,$$

$$P_{cri} = \frac{\pi^2 EI}{l'^2} = \frac{\pi^2 \times 200 \times 10^6 \times 3.36}{132.6^2} = 377.2MN$$

(9)

8.3 Deck Torsion

Asymmetric traffic and wind loads induce torsion within the deck. The deck-truss consists of horizontal and vertical trusses longitudinally, braced transversely by steel cross girder sections.

**Table 9:** Torsion Loads

<table>
<thead>
<tr>
<th>Transverse HA Load</th>
<th>0.71$\times$1.5$\times$1.1$\times$1006$\times$4.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB Load</td>
<td>$1178.5KN/m$</td>
</tr>
<tr>
<td>Knife edge load</td>
<td>$120KN \times 2 \times 1.3 \times 1.1 = 342.3KN$</td>
</tr>
</tbody>
</table>

$T = 182.2MN/m$.

Simplifying the deck truss as 2 flat rectangular strips 30mm thick, the torsion constant J is given by summing Eq. (10) for both strips.

**Figure 8:** Simplified Section

$$J = \sum \frac{1}{3} bt^3 \Rightarrow 2195.3m^4$$

(10)

The shear flow and stress, obtained from Eqs. (11, 12) respectively, (where $A_e$ is the enclosed area) is well below the capacity of steel.

$$q = \frac{T}{2A_e} = \frac{182000}{2 \times 174.8} = 520.6KN/m$$

(11)

$$\tau = \frac{q}{t} = \frac{520.6}{30} = 17.4KN/mm^2.$$  

(12)

8.4 Parapet Impacts

The parapets that are used on the bridge do not satisfy the current parapet design standard, BS6779, as the bridge was built before this standard was developed [2]. The bridge features two parapet types - one on the traffic lanes, with no panel infill and a more conventional arrangement on the pedestrian walkways.

**Figure 10:** Parapet types seen on the bridge [2]

It is obvious that having no infill on the parapet is dangerous, as the containment provided against vehicle collision is reduced. As this parapet is critical, it will be analysed with respect to vehicle impact.

A parapet must resist a 30 tonne vehicle travelling at 64Km/h (17.78ms$^{-1}$), at an angle of 20° to the parapet to provide high containment (BS6779, clause 6.1.3). A specification for high containment is appropriate given that the bridge is open to HGV traffic.
The velocity perpendicular to the parapet is:

\[ V_{\text{perpendicular}} = 17.78 \times \sin 20 = 6.08 \text{ m/s} \]

Assuming the dynamic deflection = 0.5m and that 90% of the collision energy is absorbed in the impact and solving Eqs. (13-15) to find the time for the vehicle to become stationary:

\[ F \cdot \Delta t = m \cdot \Delta v_{\text{perpendicular}} \tag{13} \]

\[ s = ut + \frac{1}{2}at^2 \tag{14} \]

\[ v = u + at \tag{15} \]

\[ \Rightarrow \Delta t = 0.164s, F = 111.2\text{KN} \]

9 Serviceability

Deflection occurs primarily through extension of the main and suspender cables. The nominal total weight acting on these cables is obtained by applying the dead, superimposed dead loads and the full HA loading applied in section 8.1 as well as the pedestrian loading. With a main cable deadweight of 21.2KN/m,

\[ \sigma_{\text{serviceability}} = 134.1\text{KN/m} \]

Solving Eqs. (7,8) gives a horizontal force of 186.4MN and a stress of 817.2N/mm² respectively. For a main span cable length of 1190m (by dividing the total cable weight on the main span [5] by the cable area multiplied by the steel unit weight), the extension of the cable is:

\[ x = \varepsilon \cdot l = \frac{817.2}{200 \times 10^3} = 4.86m \tag{16} \]

With each suspender cable (spaced at 18.3m, with a 1735mm² area) supporting a load of 123.5KN, the strain in each cable is

\[ \varepsilon = \frac{\sigma}{E} = \frac{1301.7}{200 \times 10^3} = 6.509 \times 10^{-3} \]

Which, from Eq. 16 gives an extension of 0.586m in the longest suspenders (90 long [5]) and 0.0156m in the shortest suspenders (2.4m long [5]). The full deck deflection is a result of the combined effect of these individual deflections. While the values may seem high, suspension bridge tend to exhibit large deflection, plus the free cable saddles will cause the side spans to go into tension, sharing the load and reducing deflection. Furthermore, the rigidity of the deck truss will reduce some of the vertical deflection.

10 Temperature

Temperature has a significant effect on the loading capacity of the bridge. Articulation is provided via expansion joints adjacent to the main towers and on the undersides of the approach viaduct supports [2]. The major structural elements in which temperature effects are significant are the deck truss and the hanger cables.

Within the deck, two temperature effects must be considered, an overall increase in temperature (effective temperature) and a differential temperature profile throughout the deck, which occurs when the deck is heated and cooled at different parts of the day.

The effective temperatures are derived from the air shade temperature, taking the deck construction into account. As the deck is steel on a steel truss, it is defined as group 2 ([5], fig. 9).

<table>
<thead>
<tr>
<th>Air Shade Temp. ((5), fig. 7)</th>
<th>Effective Temp. ([5], tables 10 &amp; 11)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum: -18°C</td>
<td>Minimum: -19°C</td>
</tr>
<tr>
<td>Maximum: 31°C</td>
<td>Maximum: 41°C</td>
</tr>
</tbody>
</table>

Hence, the worst-case effective temperature is 41°C. The temperature expansion, x, which must be accommodated by the main expansion joints is given by Eq. 17:

\[ x = \alpha \cdot \Delta T \cdot l \] \tag{17}

For steel, the linear thermal expansion coefficient \( \alpha = 12 \times 10^{-6} \). \( \Delta T \) is the increase in deck temperature above datum. As the bridge was constructed throughout the year, a datum temperature of 5°C seems appropriately conservative. Hence \( \Delta T = 36°C \). For a main span length of 1006m,

\[ x = 0.435m \]

This expansion is rather large, although the joints are designed to accommodate a large movement [2].

The worst case temperature effects in the main span deck can be found by assuming the expansion joints have seized. This has actually occurred in some cases [2]. If the deck is restrained from expanding longitudinally, stresses will occur within the truss members and reduce their axial load capacity. This effect is relevant to both temperature cases to be considered.

The temperature differentials are shown in Fig. 12.

---

**Figure 11: Vehicle collision angle**

The velocity perpendicular to the parapet is:

\[ V_{\text{perpendicular}} = 17.78 \times \sin 20 = 6.08 \text{ m/s} \]
Considering effective temperature only, the diagonal members (53° to the longitudinal direction) in the horizontal truss planes will be checked as they are the longest and most therefore most critical. Given the element properties shown in table 11,

\[ \text{Table 11: Element properties} \]

<table>
<thead>
<tr>
<th>Length</th>
<th>15100mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth x Width</td>
<td>650mm x 650mm</td>
</tr>
<tr>
<td>Area, A</td>
<td>74400mm²</td>
</tr>
<tr>
<td>Second moment of Area</td>
<td>4.782 x 10⁹</td>
</tr>
</tbody>
</table>

The stress in the direction of expansion is

\[ \sigma = \varepsilon \cdot E = 4.32 \times 10^{-4} \times 200 \times 10^3 = 86.4N/mm² \]  

resolving to 143.6N/mm² along the axis of each element.

Applying γfl=1.05 (for steel) & 1.30 (for temperature expansion) & γf3=1.10 this increases to 215.6N/mm².

With a yield strength of 355N/mm² and applying a material partial factor of 1.15 to obtain a material design strength of 308.7N/mm², 69.8% of the element’s axial capacity is taken by temperature effects, which is a very high value. However, it must also be said that there is a 1 in 120 year probability of this temperature occurring and the assumed datum value is quite low.

Multiplying the element stress by the element area gives an axial force of 16.0MN. Solving Eq. (3), \( P_{\text{crit}} = 41.4MN \), hence, the element will not buckle.

The same temperature increase in the cables will cause a similar loss in capacity. Solving Eqs. (17, 19, 20) for the main, longest and shortest cables, the loss in strength can be seen to be 6.436% in each cable.

\[ \text{Force capacity loss} = \Delta P_{\text{tem}} = E\cdot \varepsilon \cdot A, \]  

\[ \Delta P_{\text{uli}} = \frac{\sigma}{\nu_{\text{steel}}} \times \text{Area} = \frac{355}{1.15} \times \text{Area}. \]  

\[ \text{Table 12: Cable Temperature Effects} \]

<table>
<thead>
<tr>
<th>Cable</th>
<th>x (m)</th>
<th>( P_{\text{tem}} )</th>
<th>( \Delta P_{\text{uli}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Cable</td>
<td>0.514</td>
<td>19.71MN</td>
<td>306.2MN</td>
</tr>
<tr>
<td>Longest suspender</td>
<td>0.0393</td>
<td>149.9KN</td>
<td>2.329MN</td>
</tr>
<tr>
<td>Shortest Suspender</td>
<td>0.001</td>
<td>149.9KN</td>
<td>2.329MN</td>
</tr>
</tbody>
</table>

11 Creep

Creep has not been a documented problem in the bridge. This is likely to be because the structural elements are steel, which has a negligible creep deflection. The elements are also subject to varying loading rather than the constant loading that causes creep to develop. Fatigue may therefore be expected, however this has also yet to be found to be problem [2].

12 Natural Frequency

Ref. [5], Clause 6.13 states that vibrations arising from live loads do not normally need to be considered for highway bridges. However, if the bridge were constructed now, it may be classified as dynamically sensitive and require dynamic modelling, which is beyond the scope of this paper.

The natural frequency \( \omega_n \) for a bridge span of length l with mass m can be approximated with Eq. (21):

\[ \omega_n = \beta \frac{E}{m} \frac{l^2}{I} \]

where the factor \( \beta \) accounts for the fixity of the span under consideration, and the mass m comprises the dead load and superimposed dead load only. The frequency should fall between 5Hz (below which structural damage may occur) and 75Hz (above which psychological effects may affect the users of the bridge) [3].

\[ \text{Table 13: Fixity conditions} \]

<table>
<thead>
<tr>
<th>End conditions</th>
<th>(( \beta l ))^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed- fixed (1st mode of vibration)</td>
<td>22.373</td>
</tr>
<tr>
<td>Fixed- fixed (2nd mode of vibration)</td>
<td>61.670</td>
</tr>
<tr>
<td>Fixed- pinned (1st mode of vibration)</td>
<td>15.420</td>
</tr>
</tbody>
</table>

With the fixity shown in Fig. 13, the centre span 1st mode of vibration is the most critical.

\[ \text{Figure 13: Assumed fixity and modes of vibration} \]

Assuming:

\[ E = 200 \times 10^9 N/m² \]
\[ I = 11.570m^4 \]
\[ l= 1006m \]
\[ M = 17.3 \times 10^3 Kg/m \]

The same I value has been assumed for both spans. The stiffness of the concrete deck on the side spans has been ignored, as this is insignificant compared to the truss stiffness.

Hence,

\[ \omega_{n1} \Rightarrow 0.808Hz, \]

which falls below the ideal range of vibration. However, the deck can be more realistically considered.
as a beam supported by a series of springs representing the cable supports, which will give a lower span length and different stiffnesses (\(\beta_{nl}\)) values. Assuming each cable connection provides complete fixity (\(l = 18.3\)m), the frequency is 772.7Hz, indicating that the actual natural frequency is between these extreme values.

13 Durability, Maintenance & Vandalism

The most significant durability issue affecting the bridge is corrosion of the cable strands. This corrosion is partly due to the order in which the cables were spun, which caused water to be trapped in certain areas [8]. Accordingly, corrosion is most pronounced at the midpoint of each cable, where a number of the strands are actually broken [8]. A number of scenarios have been proposed, to account for the proportion of the strands within the cables that are broken. The most feasible models predict a strength loss between 8 and 15% [8]. If a 15% loss of strength does occur, this will give a lower bound strand strength is about 1310 N/mm². As a stress of 1057 N/mm² was observed in section 8.1, the factor of safety against failure is 1.24, which is rather low. However, this is based on the lowest cable strength predicted and an over conservative loading.

As this loss of strength will increase if corrosion is allowed to continue, an acoustic monitoring system has been implemented to measure the rate of corrosion, and hence predict the working life of the structure [8]. A dehumidification program has also been introduced to prevent further corrosion and improve the lifespan of the structure, which has met with some success [10].

The bridge also requires a large amount of maintenance to continue in operation. Since the bridge’s introduction, the towers have undergone multiple strengthening programs [2], reflecting the increasing vehicle loads that the bridge has been subjected to since its construction.

Vandalism has not been a notable problem on the bridge, which is probably due to its large size and low pedestrian traffic that can be expected.

14 Future

The corrosion problems in the cables mean that it is currently estimated that the bridge will be closed to HGV traffic by 2013 [10]. As this would have a major effect on Scotland’s economy, the construction of a supplementary cable-stayed bridge to the west of the current site was proposed in 2007, at a minimum potential cost of £3.2 billion [10]. This has understandably met with considerable opposition from local residents [10] who feel that this is an unnecessarily expensive option given that the current bridge may stay in operation following the corrosion treatment programs or other repair options such as replacement of the main cables [10].

15 Conclusion

This paper has analysed the aesthetics of the structure and discussed that although it could be improved visually, the aesthetics of the bridge were dominated by design principles in practice at the time of the bridge’s construction.

Strength analyses carried out in this paper have verified that the bridge’s satisfies current design codes, although the bridge has been shown to be sensitive to temperature effects, which demonstrates the importance of the bearings being maintained in future.

However, as the bridge’s future is in such doubt after a relatively short lifespan, whether it can be judged as a true success is open to question.

References