A CRITICAL ANALYSIS OF THE CLIFTON SUSPENSION BRIDGE

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Abstract: This conference paper provides a critical analysis of The Clifton Suspension Bridge, crossing the Avon gorge, Bristol. The paper will cover the design aspects in order to assess the bridge’s current day performance. Topics covered will include aesthetics, structure, construction, loading and maintenance.

Keywords: Brunel, Bristol, Clifton, Suspension Bridge,

1.0 Introduction.

The Clifton Suspension Bridge connects Clifton Down and Leigh Woods either side of the Avon Gorge. The notion for the suspension bridge originated in 1753 upon the death of the highly respected Alderman, William Vick.[1] William Vick bequeathed £1000 to the Society of Merchant Venturers to build a toll free stone bridge[5]. It was expected for the money to accumulate interest until it reached £10,000 and then be used to construct the bridge crossing the gorge. By 1829 it became apparent that the intended £10,000 was not going to be enough to construct a masonry structure. An act of parliament was obtained in 1829 to change Vick’s will that had stipulated the toll free stone bridge and to make the structure out of steel and for tolls to be charged [5]. On October 1st a competition was held in order to find a design for the bridge. Isambard Kingdom Brunel entered four exquisite designs.

However Thomas Telford rejected Brunel’s designs on the grounds that although “pretty and ingenious would certainly tumble down in a high wind” the other designs were dismissed for the reason that “none of the designs were suitable for adoption. Telford then proposed plans for a single short span suspension bridge. Initially thought to be a good solution, latter came under much criticism. In October 1830 a second competition was held. Brunel finally Triumphed, placing the piers on top of the rocks each side of the gorge boldly crossing the vast opening [1]. The design was seen as very ambitious. Brunel pushed hugely for a single span; he believed it was well within the limits of a suspension bridge. These feeling are likely to have been aroused due to Telford’s Runcorn project which had a central span of 30.5m. A “timid solution” in Brunel’s eyes [2, p308]. Work began in 1830 but was soon interrupted due to insufficient availability of load capital due to the Bristol riots in October. It was not until 1836 that the foundation stone was laid and the construction preceded until 1843 when the trustee’s funds were exhausted. The ironwork was sold to settle the debts of the trustees leaving the construction abandoned. Brunel died in 1859. In 1860 members of the ICE formed a company to complete the bridge as a monument to the “late friend and colleague”. [2] Completion was made possible due to the demolition of Hungerford Bridge resulting in the chains being made available for use at Clifton for a reasonable cost. Barlow and Hawkshaw were appointed as the engineers to oversee the completion of the bridge. The bridge was complete and open for traffic in December 1864. [3]

The Clifton Suspension Bridge remains standing today not only as a historical monument to its designer Brunel but also as an essential component to Bristol’s infrastructure and its history.
of the towers, the considerations taken in the proportions between the dip in the chains, the length of the bridge deck and the spacing of the rods. Design factors such as these that were high in Brunel’s priorities have resulted in a structure that doesn’t just deny the norms expected in bridge design but exemplifies the genius of Brunel and in the process created a landmark structure that compliments and now defines its surroundings.

Clifton Suspension Bridge has an industrial feel with its wrought iron chains and girders along with the red sandstone abutments and pennant stone faced towers. [3] The chains, girders and rods are painted white creating the feeling of less bulk and a more considered construction in contrast to that normally associated with an industrial design.

As mentioned previously one of the crowning features of the Clifton Suspension Bridge is its character. This isn’t achieved by use of an intriguing structural approach, but is due to the sheer presence created by the crossing of the suspension structure over the huge Avon gorge. This feature creates undeniable character as it goes against the expected standards such as a suspension/cable stayed structure crossing a wide span of water.

The Clifton Suspension Bridge is structurally honest clearly displaying its true purpose and structural strategy. Although this may be viewed as a result of the simplicity in design and composition it is a result of the combination of this and a certain level of complexity. The complexity is evident in the detailed design of the structural elements. Examples include the formation of the wrought iron cross-braced girders forming the frame of the bridge deck along with the balustrades (providing stiffness). This combination allows the deck to operate in such a way that it spreads loads and that they get passed through the suspension rods in purely tension. Without such features in the detailed design it is likely that the bridge would have ended up being cumbersome in order to function, seriously reducing the aesthetic appeal.

In summary of the aesthetics of the Clifton Suspension Bridge it is clear that they have been carefully considered. Despite not fully applying all of Fritz Leonhardt’s 10 rules of aesthetics, it cannot be argued that the Clifton Suspension Bridge isn’t an incredible example of an aesthetic success.

3.0 Refinements – Aesthetic and structural

Although constructed in the mid 1800’s with limitation in the technologies and construction techniques, The Clifton Suspension Bridge demonstrates refinements in design through out the structure. Many of these are as a result of the Engineers Hawkshaw and Barlow who were appointed to the project post Brunels death. They insisted on the addition of a third chain. Not only did this have the structural benefit of eliminating bending of the links but also it had the aesthetic benefit of the rods and chains appearing as a unit rather than just two members

<table>
<thead>
<tr>
<th>Table 1: Key Statistics</th>
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<tbody>
<tr>
<td>Deck Length</td>
</tr>
<tr>
<td>Loaded Length</td>
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<tr>
<td>Overall Width</td>
</tr>
<tr>
<td>Tower Height</td>
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<tr>
<td>Height of Saddles</td>
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<tr>
<td>Clearance above water</td>
</tr>
<tr>
<td>Height of piers</td>
</tr>
<tr>
<td>Dip of Chains</td>
</tr>
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</table>

2.0 Aesthetics

Clifton Suspension Bridge is one of England’s most famous bridges crossing the 300ft deep Avon gorge. Whether it is viewed as a beautiful or not is an entirely subjective matter. The aesthetics will be assessed on the 10 principles set out in Fritz Leonhardt’s book “Bridges”.

The Clifton Suspension Bridge is clear in its structural Form. The two large masonry towers sitting on substantial abutments either side of the gorge instil a sense of strength. It is evident that the towers are capable of supporting the wrought iron chains from which the rods support the deck. The structure as a whole imparts a feeling of stability to its users both from a close and from a far.

The proportions of the bridge are an extremely interesting aspect and one that Brunel has clearly identified. There is a geometric balance between size, spans and heights of the different members. The seemingly dominant masonry towers are balanced by the width and depth of the deck and the suspension chains. Whilst the deck and suspension cables compliment one another meeting at mid-span and connect either side via regularly spaced suspension rods. However all of these proportions are completely contrasted with the huge 250ft gorge between the bridge deck and the water below. It is unusual for a suspension bridge to have such a large space beneath the deck. As a result of careful placement of the tower right on the cliff face, a balance is achieved. This feature in the design is undoubtedly a crowning feature of the structure and is a huge factor in the bridge achieving the presence, charm and character that it does.

Despite being constructed from many individual members such as the links in the chains, the different plates with in the deck girders. The Clifton Suspension Bridge achieves a feeling of unity through a combination of fine detailing, repetition and strong lines through out the structure.

Already mentioned, is the unusual feature of a suspension bridge crossing a deep valley. However this has not caused the Clifton Suspension Bridge to appear unfitting with its surroundings of urban Clifton and the rural Leigh Woods. Brunel identified in the early stages of design that “such imposing scenery demanded a work that was bold simple and unobtrusive”. [2] These criteria were achieved through features such as the refinement and architectural style

[6]
Hawkshaw and Barlow also applied major changes to the design of the deck. They changed from the original timber beam and chains to a wrought iron frame made of longitudinal and cross girders on which timber baulks sat. This provided a deck with far greater stiffness and structural capacity. Without this change it is likely that the bridge wouldn’t have lasted and been able to withstand the increase loads through out its life, without huge strengthening programs.

Further examples of refinements within the structure are in the design of the towers. The Egyptian influenced towers taper towards the top along with having voids in the plane of the deck. The towers also differ from each side; the Clifton tower has rectangular edges along with the arched openings in the sides whilst the Leigh Wood tower has chamfered edges and no side openings. The arches within both towers vary also. These reasons for these differences are not documented but are likely to be a combination of Brunel’s attention to detail along with the possibility of different contractors [Rowlands. M, pers. coment. 15/04/2011] 

**Figure 01 - Comparison of the two towers.**

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**4.0 Structural Design**

The Clifton Suspension Bridge is clear in its structural approach. It is a traditional suspension bridge. The substantial masonry towers on either side of the gorge provide support for the suspension chains. The abutments are substantial masses of masonry faced with red sandstone ashlar on which the masonry towers sit, which are then faced on a local pennant stone. The sizing and design of the towers was not subject to much formal analysis. The height of the tower was based on the height of a block of masonry required that’s weight equated to the vertical force passing through the tower from the chains. From this it is presumed that Brunel deduced that he would be able to spread the chain load through the saddle and across a great enough surface area that the stone would not crush. The only other areas checked for the tower was to evaluate the area at different cross sections to ensure a uniform stress distribution throughout. [2] The towers house the wrought iron saddles placed on cast iron roller frames with the rollers being made of cast steel [1] This allows for horizontal movements of the chains resulting in the towers theoretically only experiencing vertical compressive forces.

The suspension chains are constructed of wrought iron links. The suspension chains are made up of three sets of chains that span between the Clifton and the Leigh Woods tower. The anchorages for the chains are constructed of land saddles that guide the chains through a 25m tunnel inclined at \( \approx 45° \) down to the now reinforced anchorages. [3] It was known at the time that the shape of the chains affected the forces within it, for example if the curve was too shallow the forces would be extremely high and if the dip too high the chain would likely to be prone to oscillations. Brunel specified the dip: span ratio as 1 in 10 (economical by modern standards). This value was accepted but recommended by third parties to be increased to 1 in 12 or greater. However “for a given suspended load a chain with a ratio of 1 in 12 is 16% heavier than a chain with a ratio of 1 in 10” [2, p314]. This enabled reductions to be made in the cross sectional areas of the chains resulting in financial savings to a project which progression relied upon efficient budgeting. The suspension rods are fixed to the three separate chains via the bottom, middle and top continuously along its length in order to spread the load evenly [5].

The bridge deck is constructed of two longitudinal girders spanning the length of the deck. These girders were constructed in 5m sections. Lattice cross girders are fitted between the longitudinal girders bracing the structure. Further bracing is added to the deck structure from 127mm longitudinal timber baulks along with further 50mm traverse timber planking [1]. See **Figure 02**

**5.0 Loading**

With the Clifton Suspension Bridge being located in the UK it is logical to assess the load cases in accordance with British Standard BS5400. All bridges experience the same type of loads but with varying magnitudes, these include dead, superimposed dead and combinations of live loading cases. BS 5400
sets out that all loads need to be factored by the partial load factor, γf and a further load factor γm that accounts for possible inaccuracies in the analysis of different bridge types. γf is taken as 1.00 for steel bridges in the serviceability limit state (SLS) and 1.10 for their ultimate limit state (ULS). γm varies depending on the load type. For this assessment a material partial factor of safety, γm will also be used to account for possible inaccuracies in the measurement and data of the now 146 year old structure. Temperature and wind loads need also to be considered.

The bridge is open to vehicles, pedestrians and cyclists. However by 1953 it was recognised the deck was in poor condition and the original 6 ton vehicle weight limit was again reduced to an axle load of 2.5 tons and a vehicle weight of 4 tons [3]. For this reason in our assessment it is reasonable to ignore HB loads.

5.1 Dead Load

The dead load is to be taken as the sum of the weight of the main structural members. This will include the weight of the suspension chains, the suspension roads and the bridge deck longitudinal and cross girders. It is stated in reference [1] “the weight of the chains between the piers is 554 tons.” The weight of the suspension-roads, longitudinal girders and cross girders is roughly 400 tons, giving to a total un-factored dead weight of 9.5MN. To get a value of the dead weight per metre length of bridge, we need to divide by the span between the abutments which is 636ft = 194m. 9.5MN/ 194 = 49kN/m

\[
\text{Dead load (SLS)} = 49 \times \gamma_f \times \gamma_m \times \gamma_m \\
= 49 \times 1.00 \times 1.00 \times 1.2 = 59kN/m
\]

\[
\text{Dead load (ULS)} = 49 \times \gamma_f \times \gamma_m \times \gamma_m \\
= 49 \times 1.05 \times 1.10 \times 1.2 = 68kN/m
\]

5.2 Superimposed dead load

Super imposed dead loads are those that are placed on top/ are attached to the main structure but not structural elements [BS 5400]. These include parapets, road surfaces/ finishes and utilities etc. The load magnitudes of SID’s is open to uncertainty due to limited information of the exact details along with changes made due to maintenance during the bridge life. This results in the factors of safety γf being 1.75 and 1.20 fore the ULS and SLS respectively.

<table>
<thead>
<tr>
<th>Material/ Description</th>
<th>Thickness (mm)</th>
<th>Density (kg/m³)</th>
<th>Weight (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 inch Timber Baulks</td>
<td>127</td>
<td>530</td>
<td>6.7</td>
</tr>
<tr>
<td>2 inch Timber Planks</td>
<td>50.8</td>
<td>530</td>
<td>2.7</td>
</tr>
<tr>
<td>1¼ inch Mastic Asphalt</td>
<td>30.1</td>
<td>2300</td>
<td>8</td>
</tr>
<tr>
<td>Parapet</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>18kN/m</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Superimposed dead load (SLS) = \(18 \times \gamma_f \times \gamma_m \times \gamma_m\)  
\[= 18 \times 1.2 \times 1.00 \times 1.2 = 25.92 \text{kN/m}\]

Superimposed dead load (ULS) = \(18 \times \gamma_f \times \gamma_m \times \gamma_m\)  
\[= 18 \times 1.75 \times 1.00 \times 1.2 = 37.8 \text{kN/m}\]

5.3 Notional Lanes

In order for the assessment of the live loading it is necessary to establish the number of notional lanes. These are lanes that are defined purely for the purpose of applying live loads/ HA loading and are independent of the number of lanes marked on the highway. It is stated in ref [5] clause 3.2.9.3.1 that if the structures carriageway width is greater than 5m and ≤ 7.5m the number of notional lanes is 2. The Clifton Suspension Bridge has a carriageway width of 6.1m and is therefore within these limits.

5.4 HA loading

HA loading is a combination of uniformly distributed loads across the notional lanes representing the fast moving traffic, together with a knife-edge load (KEL) that is acting within the most adverse position within the notional lane to ensure a representation of the shear and moments that may occur [7 fgg webpage]. Reference [8] states that the un-factored HA load, w for loaded lengths of between 50m and 1600 m can be calculated using equation (2).

\[w = 36 \left(\frac{1}{L}\right)^{0.1}\]  

The loaded length of the bridge is 194m giving a value of \(w = 21.3\text{kN/m}\). This value along with the 120kN KEL needs to be multiplied by the factors of safety γf and γm for the different load combinations to give the factored HB loads for SLS and ULS.

<table>
<thead>
<tr>
<th>Load</th>
<th>SLS</th>
<th>ULS</th>
</tr>
</thead>
<tbody>
<tr>
<td>HA</td>
<td>29.4kN/m</td>
<td>36.75kN/m</td>
</tr>
<tr>
<td>KEL</td>
<td>166kN</td>
<td>207kN</td>
</tr>
</tbody>
</table>

Table 3: Factored HA loads: Combination 1  
\(\gamma_f = 1.20\) (SLS) & 1.50 (ULS)  
\(\gamma_m = 1.15\)

It is stated in reference [8] 6.4.1.1 that the lane loadings are “interchangeable between the notional lanes and a notional lane or lanes may be left unloaded if this causes the most severe effect on the member or element under consideration” [8, p47]. The relevant lane factors, β also need to be applied. With only two notional lanes these values are \(\beta_1 = 1.00\) and \(\beta_2 = 1.00\) for lanes 1 and 2 respectively [8, Table 14].

Figure 04 - Combination 1 HA loading

<table>
<thead>
<tr>
<th>Lane</th>
<th>Material/ Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Central Reservation</td>
</tr>
<tr>
<td>1.0</td>
<td>KEL</td>
</tr>
<tr>
<td>1.0</td>
<td>HA</td>
</tr>
</tbody>
</table>
The cases that will be considered are those that induce the worst moments in the deck and those that induce the greatest forces within the suspension chains.

5.5 Pedestrian Loading

As stated, the Bridge has pedestrian walkways either side of the carriageway. These loads can easily be assessed following [9] clause 6.5.1 where it states for any bridges greater than 36m the nominal pedestrian load of 5.00kN/m² can be factored down by the value $k$.

$$k = \frac{HA (UDL) \times 10}{L + 270}$$

(4)

Therefore the pedestrian load to be used is 2.5kN/m² (unfactored).

5.6 Wind Loading

The fact that the Clifton Suspension Bridge is located in the Avon Gorge is going to have a funnelling affect, and increase the magnitude and likelihood of the wind loads experienced. Due to the lighter dead weight of such a structure in comparison to that of a concrete arch it is far more likely to be affected by wind actions. The wind loads acting on the bridge are likely to produce some of the more severe strains experienced, it is therefore crucial to assess the extent that these loads have.

Gusts in the south-westerly direction are nearly parallel with the direction of the deck. The higher ground on either side of the bridge therefore diminishes these effects, allowing them to be assumed negligible and ignored in this instance. But gales from the northwest or southeast are practically in the direction of the gorge, so need to be assessed.

The wind loads will be calculated in accordance with reference [9] clause 5.3. It requires the calculation of the maximum wind gust speed using equation (5). It is the product of the gust factor and site hourly mean wind speed.

$$V_d = S_p V_s$$

(5)

$$= (S_p K_p S_h') (V_p S_p S_p S_d)$$

$$= (S_p K_p S_p S_p) (V_p S_p S_p S_d)$$

(9,p...)

$V_p$ is the basic hourly mean wind speed, this is given for Bristol as ≈ 20.5m/s the other values used are the different coefficients calculated according to reference 9 clause 5.3.2

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Value</th>
<th>Coefficient</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_p$</td>
<td>1.64</td>
<td>$V_p$</td>
<td>20.5 m/s</td>
</tr>
<tr>
<td>$K_p$</td>
<td>0.95</td>
<td>$S_h'$</td>
<td>1.05</td>
</tr>
<tr>
<td>$T_{y}$</td>
<td>1.0</td>
<td>$S_{a}$</td>
<td>1.018</td>
</tr>
<tr>
<td>$S_{h'}$</td>
<td>1.10</td>
<td>$S_{d}$</td>
<td>0.82</td>
</tr>
</tbody>
</table>

The values in the table gave a maximum wind gust value, $V_d = 31.7$m/s. This allows the calculation of the nominal transverse load, $P_t$ (equation (4)) that is taken to be acting on the centroid of the member being assessed. Already mentioned this particular case is likely to be the worst case due to the orientation and location of the bridge.

$$P_t = q A_1 C_d$$

(6)

Where, $q = 0.613 V_d^2$ N/m²

$$= 616N/m^2$$

(7)

$$C_d = 1.3$$

Figure 5 [9.1]

$$P_t = 310kN = 1.6kN/m$$

$A_1$ is the solid surface area (m²) that the wind is hitting; $q$ is the dynamic pressure head stated in equation (7) and $C_d$ is the drag coefficient stated in [9] clause 5.3.3.

To understand the magnitude of these effects that the wind has on the structure, certain assumptions have been made. The value $A_1$, has taken account of the surface area of the chains and the deck in elevation. Treating the deck/chains as a simply support beam we can establish a value for the transverse bending within the deck and hence derive the extent to which it is stressed.

![Figure 05 - Plan view of deck with wind loading $P_t$ and the resultant stress diagram.](image)

The wind load causes a transverse moment ≈ 7.5 MN resulting in longitudinal force within the deck of ±792kN (compression and tension). Assuming the area of the longitudinal cross girders ≈ 25000mm². This force will cause a stress within the wrought iron super structure ≈ 32N/mm². This is the stress caused by the wind alone and needs to be considered. It also needs to be considered in conjunction with dead weight and loading on the structure. Considering the magnitude of the wind load its clear that it is only induces a fraction of the wrought irons capable yield stress of 190N/mm².

The other wind load to be considered is the vertical wind loading $P_v$. This can be calculated using equation (6)

$$P_v = q A_2 C_L$$

(8)

Where $q$ is defined in equation (5), $A_2$ is the area in plan and $C_L$ is the lift coefficient defined in [9] 5.3.5. For the purpose of this paper $C_L$ will be taken as 0.5. Resulting in a value of $P_v = 567$kN. This force is.
The Youngs modulus of wrought iron is 200000N/mm² an increase of the stress would cause increased moments or residual stresses within the structure. The temperature effects on the timbers in the deck will be negligible in comparison to those of the wrought iron chains and deck frame.

It is stated in [2] that Brunel knew that temperature changes would affect the structure. Brunel estimated that the change in length of the chains would cause an increase/ drop of the dip of the chains ≈ 304mm (1ft). This was accounted for by ensuring that the towers were built high enough to permit such movements in the deck caused by the change in dip of the chain along with provision for movement of the chains over the saddles.

From [9] clause 5.4 values were calculated for the minimum and maximum effective bridge temperatures. These were -19°C and 44°C respectively. Giving ΔT=63°C. From this the strain caused by ΔT can be calculated using Equation (9)

\[ \varepsilon_T = \alpha \Delta T \]  

(9)

Where \( \alpha \) is the coefficient of thermal expansion which for wrought iron will be take as \( 12 \times 10^{-6}/°C \)

\[ \varepsilon_T = 756 \mu \varepsilon \]

From here the expansion of the deck ΔL can be calculated.

\[ \Delta T = \varepsilon_T L \]

\[ = 146 \text{mm} \]  

(10)

The movement joints and the saddles are able to accommodate an increase/ decrease of this length over the 214m span of the bridge. However if these were to become damaged and stop working it would result in an increase of the stress within the deck, equation (11).

\[ \sigma_c = \varepsilon_T E \]  

(11)

The Youngs modulus of wrought iron is 200000N/mm²

\[ \therefore \sigma_c = 150 \text{N/mm}^2 \]

Such a large increase such as this could cause buckling with in the deck and failure of the structure, thus highlighting the need for regular maintenance.

6.0 Strength

Analysis will be used to assess the strength of the bridge and to the extent its elements are stressed. This will be in conjunction with loads established to that of modern day standards. Due to limited information regarding sizes and details of members, conservative assumptions have been made in these areas consulting references [2] and [3].

5.1 Suspension Chains

The suspension chains are undoubtedly going to be one of the most highly stressed elements within the structure (along with the towers). For the purpose of this paper it will be assumed that the chains experience only tensile force and that any transverse movement from wind will be ignored. It will also be assumed that deck is stiff and therefore evenly distributes the loads across the chain. The forces within the chains can be easily assessed using:

\[ \text{Figure 06 - Horizontal and vertical forces in suspension cable} \]

Where \( H = \frac{wL^2}{8f} \), crow \( f \) is the dip of the chain and \( L \) is the length of half the chain. \( H=12.1 \text{MN} \) and \( V=8.9 \text{MN} \). From these values the tension in the chain can be calculated by resolving the vertical and horizontal forces. From drawing and sketches in reference [1], it can be deduced that the initial angle of the chains from the piers inwards is ≈36°. This allows us to establish the maximum tension within the chain is 15MN. Knowing the following:

\[ \sigma = \frac{F}{A} \]  

(12)

Reference [1], states that the surface area of the chains is \( 440 \text{inch}^2 = 280900 \text{mm}^2 \)

\[ \sigma = 53.4 \text{ N/mm}^2 \]

This value is only roughly 27% of the wrought irons capable yield stress of 190 N/mm². Despite this value being based on assumptions regarding the behaviour of the deck and some of the member sizes it gives an idea of just how over engineered the bridge is.
5.2 Bridge Deck

In order to assess the forces within the bridge deck the cables have been assumed to be rigid and therefore effectively acting as supports upon which the continuous bridge deck sits. The deck will be treated as a continuous beam with supports at 2.8m intervals. This will enable an assessment of the magnitudes of the maximum hogging and sagging moments that the deck will experience.

Figure 07 - Worst case loading.

This case results in maximum sagging and hogging moments of 158kNm and 152.4kNm respectively. These values can give us an insight into the stress within longitudinal girders due to bending moments. Using equation (13) the stress will be calculated.

\[
\sigma = \frac{My}{I} = 15N/mm^2
\]  

(13)

This value is extremely low and is a fraction of the wrought irons capable yield stress = 190 N/mm²[10]. This may partly result from overly conservative estimates regarding the size of the longitudinal girders. However it does suggest that girders were sized and designed to ensure sufficient stiffness in the deck in order to distribute the load evenly across the chains.

5.3 Inaccuracies

Both of the values for the stress in the chains due to tension and the deck due to bending are uncharacteristically low. There is possibility of inaccuracies in the data obtained for the weight of the dead loads. Conflicting information was found in reference [11] stating the total weight of the structure between the piers to be 30MN rather than that of the 9.5MN used in this assessment. This would cause increases in stress stresses in the region of 3×. If this were the case the bridge would still be deemed structurally safe to modern day standards but would yield figures closer to the magnitudes one would expect. E.g. σ in the deck ≈ 30% of its capacity and σ in the chains ≈ 80-90% of there capacity.

5.4 Buckling

Despite clearly having sufficient strength there may be a possibility of buckling within the bridge deck. Insufficient data would call for far to many assumptions to be made resulting in data of little use. However this has not occurred in the bridges 140 year history so unless there is a serious increase in the degree of loading or greatly adverse weather conditions such an incidence is unlikely.

6.0 Natural Frequency

The assessment of the natural frequency of the bridge is an extremely important aspect of the bridge design. If overlooked could possible failure and collapse of the structure.

At the time of design construction the area of natural frequency and dynamic effects was not considered such an important area in comparison to that of the design loads. The nature of such actions was also a matter of personal opinion. Awareness of these actions was sometimes apparent in the engineer’s detailed design of the chain and decks.

Today’s standards set out that the fundamental natural frequency, \( f_0 \) of the superstructure is required to fall in the limits greater 5Hz and less than 75 Hz. If natural frequency falls below 5Hz then anything from gusts of wind to the effect of pedestrians walking on the bridge can cause excitations creating a range of problems. Natural frequencies of larger than 75Hz can cause large movements of the structure, which again are undesirable causing psychological discomfort to users. The natural frequency can be worked out using:

\[
f_0 = \frac{C^2}{2\pi^2} \sqrt{\frac{Elg}{M}}
\]

(14)

Taking the 1 value of the deck to be \( \approx 5.5 \times 10^9 \) and \( M=106kN/m \) gives the natural frequency \( f_0 \) to be 12.9Hz. This suggests that vibrations should not be a problem for the structure. However it is important that this value is based on several assumptions regarding members sizes and loadings. For a more accurate assessment regarding the natural frequency of the structure it would be necessary to use computer analysis to assess the natural frequencies of the different members, the extent to which they may be effected and the extent to which they may affect others.

7.0 Foundation/Abutments and Geology

The Leigh Woods abutment is built up 33.5m Of red sandstone blocks whilst the Clifton abutment and towers sits on top of the cliff edge. The Avon Gorge is made predominantly of limestone with traces of other carboniferous rocks. It is considered stable and to have sufficient capacity. However there have been events, which have raised concerns. In 1969 there were landslips in the gorge, thus raising concerns about the abutments stabilities[5].. Investigations were carried out in which boreholes were taken. These showed that the rock directly beneath the abutments...
was very hard carboniferous limestone partlyon well-compacted Triassic breccia infilling a gorge or large fissure. Due to no presence of a continuous slip surface such as a clay or soft mudstone it was concluded that there was no present risk [3].

Several other investigations have taken place in the past none of which have raised any cause for concern. Investigations are carried out periodically, January 2010 was the most recent in which 6 × 10cm bores were drilled. This allowed data collection that gave insights into the layering and mass of the rocks beneath. Again there were no reports to raise concern[12].

It was thought until 2002 that the Leigh Woods abutment of the bridge was constructed of solid sandstone block work. In November 2002 it was discovered that the abutments were made up of a number of chambers. Although the original plans are held at Bristol University none document this design feature. The larges chambers have floor areas of up to 17.25m ×5.6m and are up to 10.8m in height [4]. The reason for these chambers is not documented in any of Brunel’s notes or calculation books. The justification that is apparent is the cost saving of such details would create. Brunel, obviously aware of the financial restriction thought it structurally feasible to implement such details.

8.0 Construction

The construction of the bridge took nearly 30 years from 1836 when the foundation stone was first laid through to 1864 when the bridge was complete and opened for traffic.

The construction of the abutments and piers was a comparatively long process in comparison to that of the rest of the bridge. Other than the delays incurred due to lack of funds and interruptions from the riots there are very few documents regarding the construction of the towers themselves.

Six lengths of wire were strung across the gorge side by side. These were bound with planks of wood forming a temporary footbridge with two ropes 3ft 6in above the planking that act as a handrail. A further wire ran above the footbridge to which a small cart was attached shown in Figure 10. This allowed the individual links of the chains to be taken from the piers out to the men constructing them. The links were laid across wooden packing blocks on top of the planks where they were then fitted to each other [5].

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9.0 Maintenance

Being a grade 1 listed structure the bridge is constantly being monitored with work regularly being carried out to maintain and keep the bridge in working order. It currently costs the Trust that overlooks the worok £1m per year in maintenance costs.

The chains were constructed from the anchorage plates upwards and from the piers inwards. Once the chains were joined at the centre the packing was lowered and the chain took its own weight relieving the stage from its action. Following completion of the first chains the second and third was laid in the same way on top the chains below

Next was the attachment of the suspension rods, and the cross girders. This was achieved by the use of apparatus made by the contractors bespoke to the project. It was a moveable crane that travelled on a railway on top of a base frame weighing in excess of 5tons. It was manufactured as such that when at the edge of the abutment it could carry a cross girder with section of a longitudinal attached into its intended position where the rods were fixed to it and the chain. Planks was then laid from the abutment to the cross girder. The railway was then advanced and the process repeated for the second cross girder and so on from both ends until met in the middle. As the planks were removed this allowed the remaining cross girders to be fixed in place [1]. Figure 11 shows various stages of construction

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Figure 10 - Section and elevation of wire rope platform

Figure 11 – Construction Process
maintenance of the bridge can be logged over two main periods. 1864 – 1953 and from 1953 until present.

9.1 Maintenance, 1864 - 1953
Documentation of the maintenance from 1864 – 1953 is very scarce. The ironwork had originally been coated in tar. In the following years this was firstly replace with bituminous compounds and finally treated with specially prepared paints. These measures kept the iron work in surprising good condition considering the harsh industrial environment it was exposed too. During 1884 -1948 the transverse timber planks were replaced three times.

Previously mentioned the initial 6ton vehicle limit. This was introduced as a result of concerns that had developed from the Clifton Suspension Bridge Committee about the increased volume and weight of traffic passing the bridge. During the same period remedial plans were put in place to strengthen the anchorages and organise provisions for the removal/ replacement of the suspension rods in the event of a failure.

In 1918 bolts from every 10th rod were removed to check their condition. The majority were deemed acceptable but a few showed cracks as a result of forging defects. Further inspection of the rods was carried out in 1923 with most highly stressed being removed and replaced. This allowed for testing of full sized rods in which tests revealed the average tensile strength of the rods was 22.3 ton/in².

There were records of concerns about the areas of the suspension chains and the anchorages. There had been severe corrosion to the chains particularly on the Leigh woods side due to the proximity of draining water. The chains were reinforced by an addition of a link where the chains entered the anchorage arch. The anchorage on the Leigh Woods side was reinforced by the introduction of an additional open link chain that passed outwards from beneath the land saddles and was fixed to a new point in the anchorage tunnel. Further strengthening was implemented by the filling of the anchorage tunnel with 9ft of concrete. The same was repeated for the Clifton anchorages several years later. See Figure 13

9.2 Maintenance, 1953 – Present
By 1953 the bridge had not only become part of the national heritage but had become a vital transport link as a result of growth of the surrounding residential areas. It was necessary to ensure the bridge was fit to meet the demand of modern day traffic.

On initial inspection it was discovered that cross girders had been subject to some mild corrosion and that each of the end girders had been severely corroded as a result of being unpainted because of there proximity to the masonry. Much of the timber baulks and deck had succumbed to brown rot. In order for this work to be executed a working platform would be needed, allowing access to all areas of the suspended structure. A 19.5 × 9.7 m cradles was designed and erected. It comprised several frames spanning from each side and which met in the middle, hanging from roller adjacent to the handrails. When not in use the frames are positioned next to the abutments.

Figure 13 - Maintenance Frame

It was decided necessary to: replace the whole of the timbers. Grit blast and zinc spray the under bridge iron-work giving protection against corrosion. Replace the end cross girders and strengthen the junction between the cross and longitudinal girders.

When the deck was completely replaced in 1958 it was discovered that roughly 30% of the timber baulks had been replaced. The deterioration of the deck timbers was a result of small gaps between the planks and the heavy tar coating that resulted in water remained in these gaps rather than passing through. The new Douglas fir deck was constructed with large gaps to promote the free drainage of the water and was incised and pressure creosoted pre installation. In addition the asphalt surface was strengthened with galvanized expanded metal laid on bitumen felt reducing water penetration.

The replacement of the end girders and treatment/ painting of the under bridge was carried out. The junction between the longitudinal and cross girders was strengthened by replacing the existing bolts with high yield steel bolts and any of the cross girders loose rivets were fixed.
9.3 Future maintenance/Changes.

The latter part of 2011 going forward sees many more plans for the maintenance of the Clifton Suspension Bridge. Over the years the abutments have seen much wear due to the elements. The first plan is the cleaning of both the red sandstone abutments beneath the tower, this will be done using gentle steam cleaning techniques that were tested on the Clifton abutment in 2010. 2014 will be the 150\(^{th}\) Anniversary of the opening of the suspension bridge. For this it intended for both the Leigh Woods and Clifton towers to be cleaned and restored. The next structural renewal/repair proposed is the replacement of the bolts connecting the longitudinal and cross girders [Rowlands. M, pers.coment. 15/04/2011]

The Clifton Suspension Bridge is one of the very few remaining examples of an “early chain suspension Bridge” [11, p1]. Being a grade 1 listed building/structure it is unlikely that there will be any changes made that would affect the aesthetics or its original purpose. The only future changes likely to be made are similar to those already planned in order to maintain the structures integrity, function and pristine aesthetics.

10.0 Serviceability

Regarding the serviceability of this type of structure it is likely to be governed by deflections and deformations. It is recorded in reference [1] that heavy winds can cause a rise and fall in the deck of up to ±6inches = 150mm. It has also been stated in [5] that the bridge is prone to deflections at each end due to the weight of vehicles entering/leaving the bridge deck. These factors would certainly compromise the serviceability of the bridge. To assess this movement computer analysis would be need to assess the effect of these dynamic movements.

However on visual inspection of the bridge the above effects stated were not visible. Movements due to the wind would not have been expected though as there was only a light breeze whilst movement due to vehicles would have been expected due to a reasonable volume of traffic.

11.0 Durability and Vandalism

The durability of the bridge is evident that it remains to this day in extremely good condition. However this is a result of the high level of maintenance and repairs that have been carried out during its life span.

The bridge itself is in an exposed position in which it is subject to the corrosive salt bearing southwesterly winds and ever increasing urban pollution. Despite all of this the wrought iron superstructure has shown little corrosion over its 147-year life span. This is the result of the various protective coatings that have been used, ranging from dark grey coal tar though to a modern day epoxy coating.

Vandalism to the bridge has not been recorded as a major problem at any point in the structures history. However, it is inevitable that certain areas such as the abutments have been victim to graffiti in some form or another. Due to the nature of the bridge being a tourist attraction as well as a vital transport link it is of high importance to ensure that these petty incidents are resolved and removed to ensure the structure is preserved to the highest quality.

12.0 Conclusion

This paper has assessed the bridge based on several assumptions and criteria. These have shown the bridges design has been able to withstand the test of time along with accommodating modern day usage requirements. Not only an essential piece of Bristol’s infrastructure but one of its greatest attractions, The Clifton Suspension Bridge will undoubtedly be preserved for as long as viably possible.

References

[12] Ibell T., 2008, University of Bath Bridge Engineering. Bath, University of Bath