A STUDY OF BIBLINS FOOTBRIDGE

I D Molesworth[1]

Student - University of Bath

Abstract: This report is a detailed study of Biblins Footbridge. The bridge is an aesthetically appealing footbridge situated over the river Wye near the town of Ross on Wye on the western fringe of the Forest of Dean. The report analyses the aesthetics, loading, construction, strength, serviceability and future requirements of the bridge, with emphasis on the central suspension span.

Keywords: *Biblins Bridge, Suspension, Wood, Steel.*

Figure 1: Biblins footbridge.

1 Introduction

Biblins Footbridge is a small suspension bridge located over the river Wye, to the west of the Royal Forest of Dean, about 5 miles south of the town of Ross on Wye.

[1] *Mr I Molesworth - idm22@bath.ac.uk*

The bridge was built in 1957 and was designed and constructed by the Forestry. Constructed primarily from steel and wood the bridge survived well until 1997, when the decision was made to refurbish the bridge. The original approach steps were replaced by ramps and the towers were replaced entirely. The original design cost £2500 to construct. This report is a critical study of the bridge design and construction.

2 Aesthetics

The analysis of aesthetics is easily broken down by Leonhardt's ten topics of aesthetics, these are analysed in section 2.1 below.

2.1 Analysis of Aesthetics

2.1.1 Fulfilment of Function

The bridge is very simple, it is obvious from its construction the way in which the loads are carried to the ground. Although the main cables are not clearly defined within the main span of the bridge, the shape of the whole structure suggests a bridge under high tension.

2.1.2 Proportions

The bridge was originally designed with relatively short staircases (about 12% the length of the main span) giving a very stunted appearance to the bridge. These staircases are shown in Fig 2, below. The bridge was renovated in 1997, with the towers being rebuilt and ramps built to improve access to the main span. These ramps follow the profile of the cables from the towers down to the ground, with the shorter ramp being just less than 30% the length of the main span. The proportions are greatly improved by this and the slight difference in length of the two ramps is not noticeable.

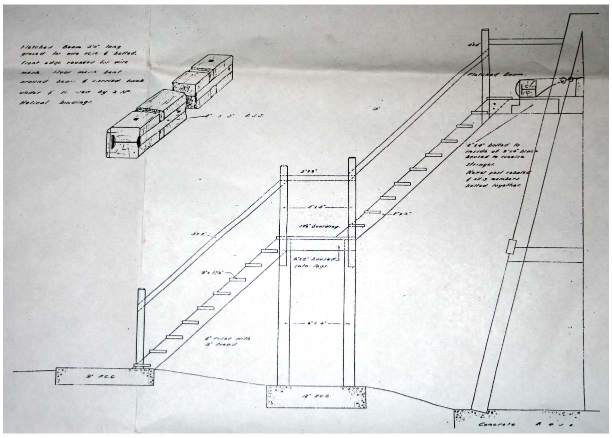

Figure 2: Original drawing of staircase. [1]

2.1.3 Order

The bridge is relatively well ordered; the main span is kept simple, with a metal grating and vertical wooden ties being the only significant objects in view. The perceived depth of the deck is increased by the introduction of a wooden kickboard, increasing the apparent solidity of the bridge.

2.1.4 Refinement of Design

The bridge, while well detailed in places is not particularly refined. Small aesthetic details have been applied to certain parts of the bridge (see Fig. 3) however these are either unnecessary or badly detailed. The pinnacles shown in Fig. 3 and 4 are an example of this. Provided to suggest the vertical supports for the ramps protruded through the handrail, the pinnacles have been inaccurately positioned so as to not be perfectly in-line with the columns. Secondly the handrail also protrudes wider than the columns and it is obvious there is no continuity between pinnacle and column (Fig. 4). The areas that are well resolved are generally the areas where the detailing has simply been left as the required construction, which, for a bridge where the description of the form is plain to see, is the most fitting treatment.

Figure 3: Elevation view of ramp support and pinnacle.

Figure 4: Approach ramp showing pinnacles.

2.1.5 Integration into the environment

The bridge is very well integrated into the environment. Being built by the Forestry Commission local wood was used for the timber parts of the bridge. The steel construction is kept to a minimum, only being used where high tensile capacity is required. From a distance it is very hard to perceive the bridge, partially due to the dense vegetation in the area, and partially due to the lightness of the main span.

2.1.6 Surface Texture

The surfaces have not been specifically treated, but left as they were constructed. The wood has been lightly planed and sanded to remove sharp points, but no treatment has been made as such, which links back into paragraph 2.1.5.

2.1.7 Colour of Components

The colour of the components has been left natural, only the large areas of sheet metal have been painted, and have been painted green in order to reduce their appearance. The bridge was left this way in order to integrate it more into the surrounding forest, with the lighting and shadow being left to define the structure. This means that the bridge fits its environment at most times of day and lighting conditions.

Figure 5: Biblins bridge in winter.

Figure 5 above shows how well the bridge fits its setting, even on an overcast day. Figure 1, a similar view with better weather, shows the bridge on a clear day at a similar time. The colour and surface texture of the bridge allow it to merge reasonably well with the background in both photos. This effect would be unachievable if the bridge were painted as it would be unlikely that the paint would be suitable for all lighting conditions and times of year.

2.1.8 Character

Character is hard to define, however it could be proposed that the bridge has a reasonable amount of it. The bridge is used widely by young people staying atthe campsite, and is quite famous in the area. A large number of people use the bridge as a focus for outings to the area and many will walk across it without need to. It can be suggested this shows the bridge has character; it inspires people to contemplate its construction and brings enjoyment to people using it.

2.1.9 Complexity

The bridge mixes the complex and simple elements well. However no deception is undertaken; the main span is obviously held up by the towers, which are clearly carrying high loads due to their size.

2.1.10 Incorporation of Nature

Linking back to paragraph 2.1.5 the bridge incorporates nature very well, the locally sourced oak gives the bridge an authentic look for the area.

2.2 Summary of Aesthetics

While the bridge is aesthetically pleasing as a whole the details have been somewhat neglected in the design. The bridge is well integrated into the environment but the lack of good detailing means the bridge has a very utilitarian look. The worst part of the bridge for this issue is the ramps, which, as discussed above contain the worst detailing. The ramps are also very ‘heavy compared to the main span. It could be suggested that the handrails are reduced in size and a similar mesh used as a parapet wall in order to lighten this part of the structure. However some degree of solidity in the construction of the ramps is advisable, as the contrast between the heavy, land based construction and the lightweight construction over the water provides a good order to the bridge.

3 Loading

The live pedestrian loading for the bridge is described below.

$$\begin{aligned}
&\textit{Nominal loading} = 5kN/m^2. \qquad (1)\\
&\textit{Reduction factor } k = \frac{W}{30}.\\
&W = 151\left(\frac{1}{L}\right)^{0.475}\\
&W = 151\left(\frac{1}{57.9}\right)^{0.475}.\\
&W = 22kN
\end{aligned}$$

$$\textit{Reduced loading} = 3.66kN/m^2$$

The reduced loading produced in Eq. (1) above, is the loading for the bridge, however for the width of bridge this reduces down to give a uniformly distributed live loading of:

$$w = 3.07kN/m$$

A bridge report from 1962, however, describes the design loading as being 30 people standing in the centre of the bridge. Taking the centre of the bridge to mean the main span, this equates to a loading of about 0.5kN/m assuming a loading of 1kN per person.

A second interpretation of this loading is that it is the serviceability limit state, this being the limit on loading where the amplitude of oscillation of the bridge is becoming dangerous.

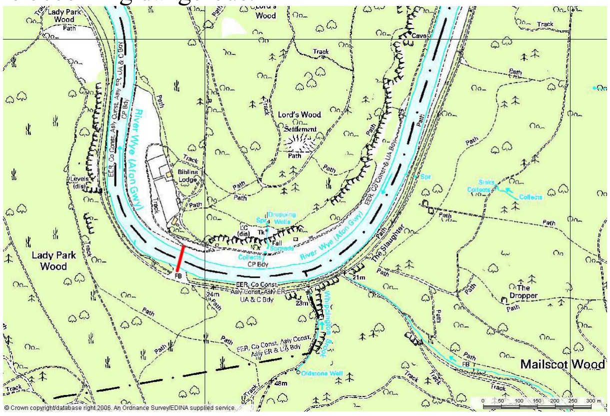

Figure 6: Location of Biblins bridge (in red). [2]

As can be seen from Fig. 6 the bridge is positioned well away from any roads, with the only access being forest rides. This means that the towers and foundations require little resistance to impact loadings. The dimensions of the bridge also allow for accidental HB loading to be ignored as it would be impossible to have a vehicle drive onto the bridge by accident.

The bridge has to be able to withstand both wind loading and low level hydraulic loading from the river. The highest recorded flood level on the 1962 report puts the water at 0.91m below the centre of the span, 1.22m above the level of the foundations. For this reason the stabilising cables that travel from the banks to three points on either side of the central span are removed during the winter months. This is due to an incident that occurred about 5 years ago when, during a high flood, a pontoon from up river caught on one of the bracing cables and caused damage to the bridge. There is no record of the flood level ever reaching above the deck. The cables are re-attached during spring in order to brace the bridge during the summer months when the use of the bridge is very high. As the bracing cables are periodically removed, the bridge must be able to carry wind loading through the main span, as well as the bracing cables. This loading is likely to be low as the elevation presented to the wind is very open, with only small areas of solid material

Dead loading, for the central span, comes to 0.457kN/m

4 Strength and Construction

4.1 Construction

4.1.1 Towers and approach ramps

The towers and approach ramps are constructed from locally sourced oak timbers and are 'A' frame shape in elevation. The joints are simple bolted connections. Metal saddle bearings are provided where the cables pass over the tower from the abutments to the central span. Resistance to lateral loads is provided by two large oak members, as can be seen in Fig. 7. Figure 7 also shows the cross bracing in the tower, this bracing is different to that of the original construction, and was changed when the towers were refurbished in 1997. The original cross bracing can be seen in Fig. 9 showing the extra level of bracing that was provided in the original design.

Figure 7: Tower construction, showing lateral bracing

4.1.2 Central span

The central span of the bridge is constructed mainly from steel. Wooden hangers run under the deck and up to the main cables. The two main cables are 28mm diameter steel wire ropes at hand-rail level. The deck and side panels are constructed of steel mesh 76mmx51mm gauge. A thin wooden board is overlaid on the foot walk for serviceability reasons. Kick boards are provided to protect the edges of the deck and side panels. I beams are attached to the underside of the deck at the quarter points, to provide connections for the bracing cables. Fig 8: Shows the additional steel that connects from the I beam to the main cables to provide some lateral stability to the main cables as well.

Figure 8: Attachment point for bracing cables

The small cross section of this member, suggests that little restraint is provided to the main cables directly from the I beam, with more restraint being provided by the wooden hangers. The bracing cables travel back to 4 fixing points on the river bank, one each side of each end of the bridge.

As far as can be gauged the bridge was originally designed to carry loading both through the main cables via the wooden hangers and through the mesh of the deck. This theory is supported by the above paragraph, where the majority of lateral restraint could be provided by the deck if it is also being held under tension. The mesh was originally designed to attach to a fletched steel I beam to which more cables also attached carrying the load to ground (Fig. 9). The cables fixed directly to the steel, with the mesh being carried by the wooden fletching

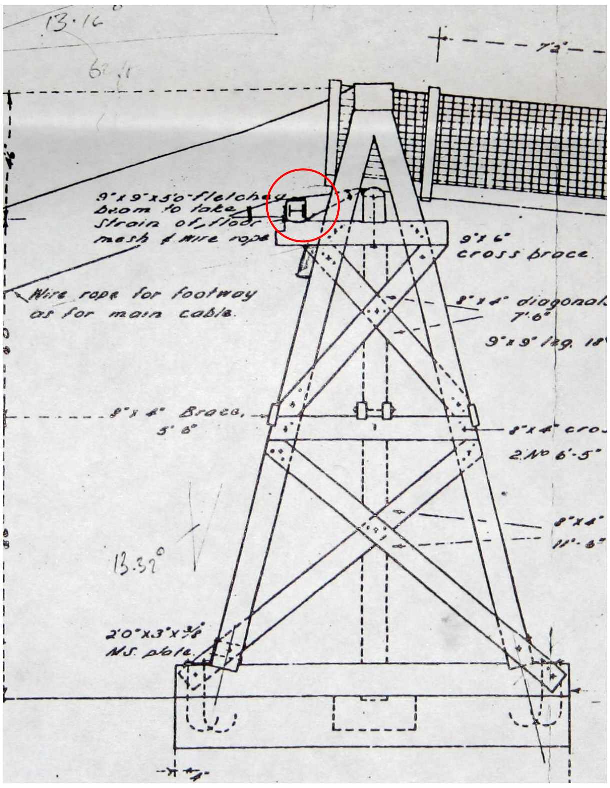

Figure 9: Original drawing showing tower and fletched beam (circled). [1]

This beam was replaced in 1997 with a circular hollow section (CHS) steel beam but as the bridge currently stands today, the deck cannot be carrying any high tensile loads. The tie back cables (labelled wire rope for footway in Fig 9 above) at either end of the bridge are slack, with noticeable a noticeable sag along their length. For a relatively light and short section of cable compared to the central span this supports the theory they are carrying no load. The attachment of the mesh to the beam is lacking at both ends and damaged at the northern (campsite) tower. Fig 10 shows this connection, with the only connection provided being two small clamps on either side of the mesh.

This shows that the main load carrying capacity of the bridge is gained from the two main cables at handrail level. Another effect of this problem, is that the wooden hangers, the major construction of which being deck level, are carrying quite high loads between the deck and handrail.

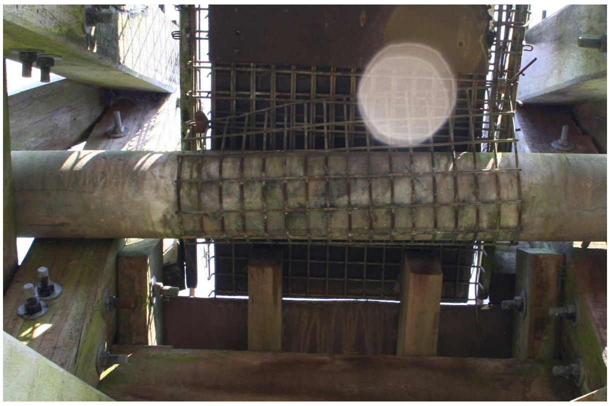

Figure 10: Connection between deck mesh and CHS beam, river towards top of picture.

In terms of erection of the central span the information was very sparse. The connection detail (figure 8) between the main cables and handrail attachments suggest that the cables were erected first, then the deck attached. The fixings are bolted plates fixing the wooden hanger to the cable. The main issue with this theory is that a large quantity of temporary works would have been required in the river while construction was occurring. Another method could possibly have been that the deck was and cables were constructed as one and then lifted in place. However this would have required larger machinery to carry the extra weight.

4.1.3 Abutments

All footings and tension anchorages are concrete, with three major anchorages carrying the load of the main cables shown in Fig 11 below.

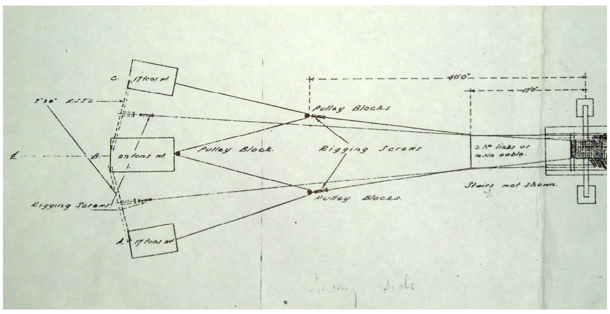

Figure 11: Showing the anchorages (left) and the main cables. [1]

The main cables travel to a barrel strainer, attached to a pulley round which runs the cable from the anchorages, making a W shape. The tensioning cables for the deck attach, via a barrel strainer, to I beams, running underground between the anchorage blocks.

This method of construction allows for inaccuracies in both the position of the anchorage blocks, and the length of the main cable to be accounted for. Any slack, or over tensioning in the cables can be adjusted by the barrel strainers. The addition of the barrel strainers means that the stringing of the main cable could occur out in the open, away from ground level, increasing access to the cable connections. The attachment of the cables to the abutments is show in Fig. 12.

Figure 12: Attachment of main cable to abutment (key for scale).

4.2 Strength

4.2.1 Assumptions

The assumptions made for the calculations are: That the bridge can be modelled as a single cable carrying a uniformly distributed loading (U.D.L.) acting vertically over a horizontal plane.

Further to the above statement, that the deck is carrying no longitudinal loads to the abutments.

That the tower tops, in the first case, are fixed, with a perfect hinge joint, allowing free movement of the cable over the saddle.

That the foundations perfectly fixed and that they will undergo no longitudinal movement or rotation, due to the failure of the soil.

These assumptions allow the bridge to be analysed statically, using standard methods as was the likely method of analysis that will have been used in 1957 when the bridge was constructed.

4.2.2 Calculations for central span

The initial calculations use the current standard live loading and assume the cable is pinned to a rigid body at the point of the tower bearings.

$$w_1 = 5.59kN/m. \quad (2)$$

Equation (2) is the factored loading, using the factors below in Eq. (3). The load factor for the dead load is taken as that for steel, as the primary load carrying material on the bridge, is the steel mesh and cables.

$$\begin{aligned}\gamma_{f1} &= 1.05\\ \gamma_{f1} &= 1.50\\ \gamma_{f3} &= 1.10\end{aligned} \quad (3)$$

Please see Fig. 14 over the page for diagram of the calculation.

Firstly, taking moments around point A for the whole structure:

$$\begin{aligned}V_B l &= 5.59\frac{l^2}{2}\\ V_B &= 5.59\times\frac{57.9}{2}.\\ V_B &= 162kN\end{aligned} \quad (4)$$

From vertical equilibrium of the whole structure then:

$$V_A = 162kN. \quad (5)$$

Taking a free body AC (Fig. 13)

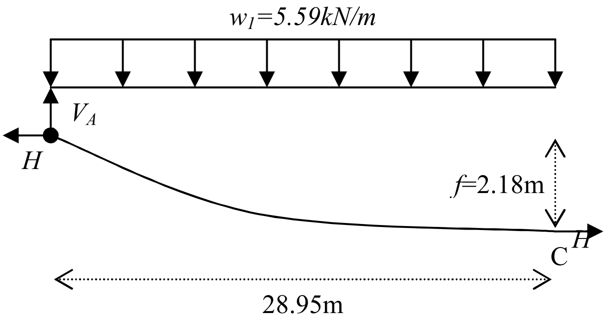

Figure 13: Free body AC.

Moments around C using V_A from Eq. (5) above:

$$\begin{aligned}2.18H &= 162\times 28.95 - 5.59\times\frac{28.95^2}{2}\\ 2.18H &= 4690-2340\\ H &= 1080kN\end{aligned} \quad (6)$$

This gives a maximum tension in the cable of:

$$\begin{aligned}T_{\max} &= \sqrt{1080^2+162^2}\\ &= 1092kN\end{aligned} \quad (7)$$

This is carried by the two main cables so the actual tension in a single cable is:

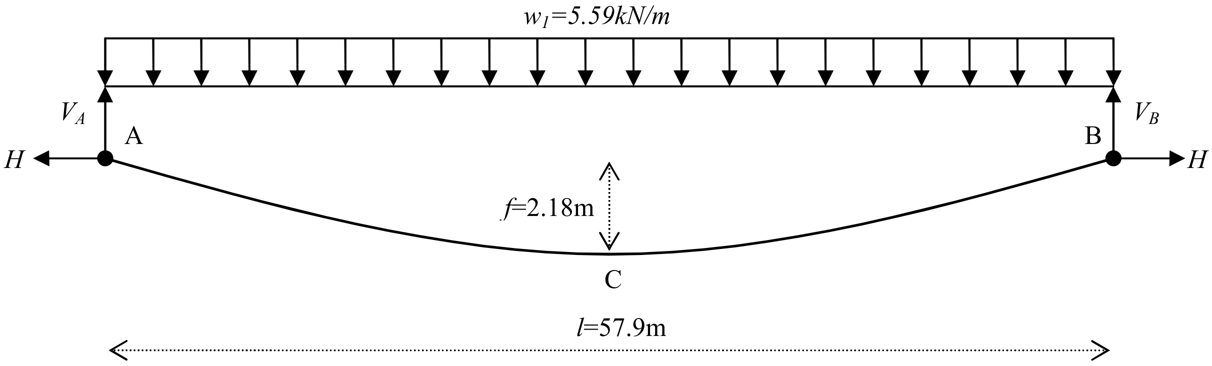

Figure 14: Diagram of assumed structure for calculations.

$$T = \frac{T_{\max}}{2} = 546kN . \tag{8}$$

From this the stress induced in the cable can be calculated, using an area of 615mm^2 for the cable the stress comes out as:

$$\sigma_1 = \frac{546 \times 10^3}{615} = 888N/mm^2 . \tag{9}$$

The stress in the cable is very high, suggesting that the bridge was designed for a much lower design load, this is likely as the bridge was designed in 1957, and today's design codes will not necessarily apply. For this reason the following calculations consider a lower live loading. The calculations make the same assumptions as the above calculation and use the same diagram as Fig. 14, with w_1 replaced by w_2, which is calculated below.

Assuming a spacing of 1 person for every 0.7m of the span, a reasonable spacing for the likely maximum live loading on the bridge, for psychological reasons, the total number of people on the bridge will be 83. Assuming a maximum loading of 1kN per person the total load is, therefore, 83kN this equates over the length of the bridge to a live u.d.l. of:

$$w = 1.43kN/m . \tag{10}$$

This gives a factored loading of:

$$w_2 = 2.89kN/m . \tag{11}$$

Repeating the above calculations gives:

$$H = \frac{w_2 l^2}{8f} .$$

$$\begin{aligned} H &= \frac{2.89 \times 57.9^2}{8 \times 2.18} \\ &= 556kN \end{aligned} \tag{12}$$

$$V = \frac{w_2 l}{2} = 75.0kN .$$

$$\begin{aligned} T_{\max} &= \sqrt{556^2 + 75^2} \\ &= 561kN \end{aligned}$$

T_{max} from Eq. (12) above equates to a stress, in the steel cables, of:

$$\sigma_2 = 456N/mm^2 . \tag{13}$$

This stress, while very close to the yield stress of steel is not likely to be too unsafe. The steel rope of the main cables will have a higher yield stress, due to their construction, than a plain steel wire of a similar dimension.

4.2.3 Calculations for anchorages on southern bank.

Using the loading calculated in Eq. (12) the forces applied to the top of the tower and footings can be calculated. It can be assumed that the footing is fixed, as in reality lateral resistance to the cable tension would be provided by the soil, which will not be investigated in this report. Vertical resistance of the footing to uplift will be calculated. It is also assumed that, for loading in the tie back cables, the cable is free to move over the top of the tower and no horizontal load is imparted to it, hence the tension in the tieback cables is governed by the horizontal load H calculated in Eq. (12) above.

$$\begin{aligned} \alpha &= \tan^{-1}\left(\frac{5.6}{14.7}\right). \\ &= 21^\circ \end{aligned} \tag{14}$$

The term α in the Eq. (14) is the angle between the tie back cable and the horizontal, hence giving a tension in the tie back as:

$$T_{tie} = \frac{H}{\cos\alpha} = \frac{556}{\cos 21} = 596 \quad (15)$$

This does give a tress in the tie back cables, higher than that of the yield stress of plain steel, therefore the loading would have to be reduced further. That said, using a higher loading will give an extra degree of safety to the anchorages.

From Eq. (15) the vertical lift on the anchorage blocks can be calculated:

$$V_{anc} = T_{tie}\sin\alpha = 596\sin 21 = 213kN \quad (16)$$

From the original drawings the total mass of concrete in the cable anchorages is 110tons (imperial) which equates to 99.8tonnes. The capacity of the concrete anchorages to withstand vertical loads is therefore997kN, nearly 5 times the applied design loading.

For assessment of the construction of the towers, the loading can be taken as being the vertical load imparted by the tie back cables, and the main span. A horizontal load will also be applied to the top of the tower, to allow for the likely chance that the bearings will be restricting free movement of the cable, this load is calculated below, assuming a coefficient of friction of 0.2.

$$H_t = \frac{\mu H}{2} = \frac{0.2\times 556}{2} = 56kN \quad (17)$$

The vertical load is as follows:

$$V_t = \frac{V + V_{anc}}{2} = \frac{213+75}{2} = 144kN \quad (18)$$

Both values are divided by two to take in to account the two cables.

Hence the loading is as shown below:

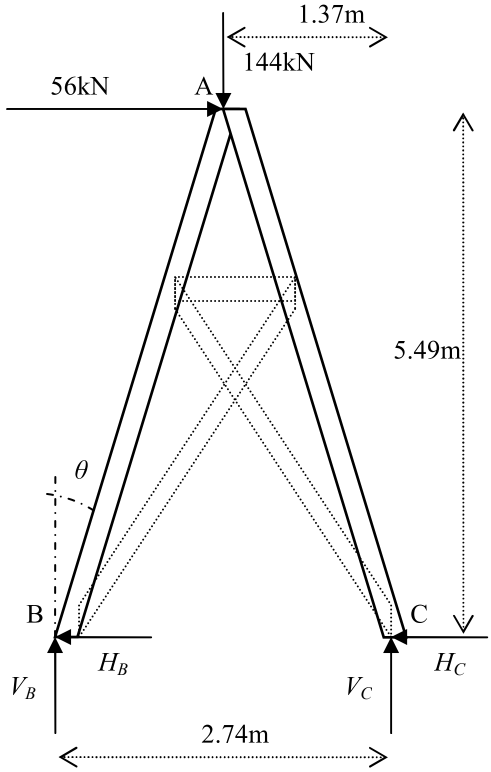

Figure 15: Diagram of loading on tower.

For these calculations it can be assumed that the tower legs are pin-jointed, with each other and the footings. Secondly assume the cross bracing serves as a means to prevent buckling rather than carrying direct load from the cable saddle to ground. Hence the major load carrying members are AB and AC, and the analysis will treat these as such. Theta (*θ)* is the angle between the legs and the vertical.

$$\theta = \tan^{-1}\left(\frac{1.37}{5.49}\right). \quad (19)$$

For equilibrium of the towers, with the structure being symmetrical (compression negative):

$$T_{AB} = \frac{56}{2\sin 14} - \frac{144}{2\cos 14} = 41.5kN$$
$$T_{AC} = -\frac{56}{2\sin 14} - \frac{144}{2\cos 14} = -190kN \quad (20)$$

This means that the reactions are as follows:

$$V_B = -41.5\cos 14 = -40.2kN$$
$$V_C = 190\cos 14 = 184kN$$
$$H_B = 41.5\sin 14 = 10.7kN\ .$$
$$H_C = 190\sin 14 = 46kN \qquad (21)$$

The V_B reaction is negative due to the member being held under tension.

The main legs of the structure are 225x225mm oak columns, therefore, taking the compression member as the critical case and using the codes of practice from BS:5268-2:2002 the sizing is as follows:

$$\sigma_{c,a,||} = \frac{190\times 10^3}{225^2}$$
$$= 3.75N/mm^2\ .$$
$$l_e = 4.81m \qquad (22)$$

Taking oak class TH2, i is the radius of gyration, λ the slenderness ratio:

$$\sigma_{c,g,||} = 8.4N/mm^2$$
$$i = \sqrt{\frac{I}{A}}$$
$$= \sqrt{\frac{241\times 10^6}{51\times 10^3}}\ .$$
$$= 64.9mm$$
$$\lambda = \frac{l_e}{i}$$
$$= 74.1 \qquad (23)$$

Hence the reduction factors for the grade stress are as follows:

$$K_2 = 0.6$$
$$K_3 = 1.25\ .$$
$$K_{12} = 0.66 \qquad (24)$$

This gives an allowable stress of:

$$\sigma_{c,adm,||} = 8.4\times 0.6\times 1.25\times 0.66$$
$$\sigma_{c,adm,||} = 4.16N/mm^2 \qquad (25)$$

The towers are, therefore constructed to a size sufficient to withstand a high loading, higher than that which the cable can carry, as shown above.

5 Serviceability

Due to its nature the central span of the bridge is the part that governs serviceability. The side spans, somewhat over engineered, are governed by deflection, while the serviceability of the central span is governed by dynamic response.

Due to the very low bending stiffness of the deck the bridge is free to react dynamically to applied loading. From first hand observations of the bridge it is apparent that the natural frequency is very close to that of most people's walking pace. The first fundamental mode (top diagram in Fig. 16) is not easily excitable due to the cable needing to expand and contract, requiring a lot of energy.

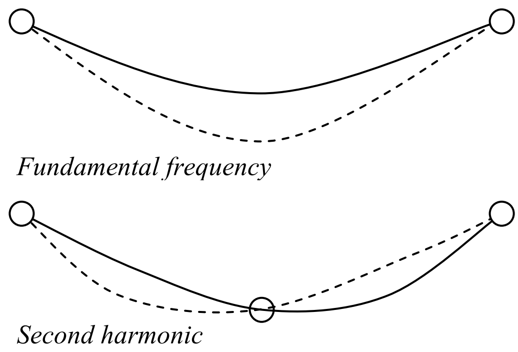

Figure 16: fundamental and harmonic mode of cable oscillation.

The second harmonic is the mode which is most easily excitable as the cable only needs to change geometry for this mode, with no extension of the material. This is supported by the observation that it is easiest to impart motion to the bridge, while walking through the quarter span points. These factors put the frequency of the second harmonic to about 1.2-1.6Hz, a value close to that of walking pace.

Figure 17: Sign on approach ramp to bridge.

Figure 17 shows the sign positioned at the end of the ramp on either side of the bridge requesting the number of people present on the bridge at any one time to be kept to a minimum. This can be attributed to the fact that a small number of people impart very little energy to the bridge and although the bridge still oscillates, the amplitude is kept reasonably small.

This oscillation, while expected, does cause problems for people walking across the bridge. Although it is designed for many people find the sensation uncomfortable and dislike using the bridge. This physiological effect was amplified in the original bridge (before 1997) when the flooring was left as the open reinforcement mesh. The bridge as it currently stands has wooden planking overlaid on the mesh to give a solid floor which greatly improves the feeling of security while using the bridge.

The bridge has little issues with either differential settlement, as small changes in the geometry of the bridge have the effect of slightly changing the profile of the cables. This small change does not seriously affect the loading as it would in a concrete or steel road bridge. Similarly creep is not an issue for serviceability as the only concrete used is in the foundations.

6 Future requirements

The bridge, due to it being a pedestrian bridge, is unlikely to require any strengthening or enlargement due to increased loading. The towers are unlikely to ever be enlarged in order to withstand impact loadings as the area is protected and managed woodland owned by the Forestry Commission, roads are unlikely to ever be built as there is little need for vehicular access to the area of the river bank around the bridge.

The timber in the towers will need to be replaced at some time in the future, however this has already been proved to be possible. As previously stated the towers were refurbished in 1997, with rotten members being replaced. Scaffolding towers were built around and above the towers and hydraulic jacks attached to the cables. The cables were then jacked off of the towers which were then dismantled and replacements built. This method could easily be used again if renovation work was required.

7 Conclusion

The bridge is a aesthetically pleasing bridge, suitable for its purpose and location. While the bridge is not strong enough by today's standards it is still strong enough to take a considerable pedestrian loading that is extremely unlikely to ever be exceeded. The bridge oscillates at a frequency close to that of walking pace, which, while not ideal, is not overpowering for most pedestrians who use it.

Overall the bridge is very successful and was well designed considering the technology available at the time. It is in need of some maintenance to bring it back to the mode of operation for which it was designed, however the redundancies in the structure allow for this not to be a serious issue.

Figure 18: Biblins bridge from northern approach.

8 References

[1] Original Drawings. 1957. Forestry Commission.

[2] © Crown Copyright/database right 2006. An Ordnance Survey/EDINA supplied service.

9 Bibliography

Ibell, T.J. 2007. *Bridge Engineering Course Notes* Department of Architecture and Civil Engineering, university of Bath

BS:5268-2:2002. 2002. *Structural Use of Timber – Part 2*. British Standards Institute

http://www.brantacan.co.uk/. 2007

Freedman, G. 1997. *Biblins Bridge Tower Temporary Supports Project Number 9716*. Forestry Commission.