A CRITICAL ANALYSIS OF THE ROYAL GORGE BRIDGE, COLERADO

S P I Cope

Department of Architecture and Civil Engineering, University of Bath

Abstract: This article provides a critical analysis of The Royal Gorge Bridge in Colorado. This is a suspension bridge spanning 366m with a main span of 268m. The Royal Gorge Bridge is also the world’s highest bridge above the surface that it crosses with a distance from deck to the Arkansas River below of 321m. The analysis in this report includes sections on aesthetics, loading, structural analysis and construction.

Keywords: Suspension bridge, Wind Cable

1 General Introduction [5,6,7]

1.1 Background of the Bridge

The Royal Gorge Bridge is located in the Rocky Mountains near Canon City in Colorado, in the USA. It was built to cross the Royal Gorge, and the Arkansas River and Royal Gorge Railroad which run along the bottom of the gorge. Due to the depth of the gorge, the Royal Gorge Bridge is the highest bridge in the world when measured from the deck to the river that it crosses, at 321m.

It was constructed in 1929 and took six months to build. It is unusual because it is at the heart of a theme park and was designed as much as a tourist attraction as a means of transport. The bridge underwent major refurbishment in 1984.

The bridge carries pedestrian and vehicular loading. The structure consists of steel towers and steel cables carrying a thin steel structure with a timber deck. Stiffness and stability is provided by an unusual wind cable system below the bridge deck which reduces sway and deflections under dynamic loading.

1.2 Dimensions of the Bridge

The bridge is 366m long with a central span of 268m and side spans of 40m and 58m. The deck is 5.5m wide. The north and south towers are 34m and 46m high respectively.

2 Aesthetics

2.1 Discussion of Aesthetics

The aesthetics of bridges is an area where an assessment is always subjective due to the personal feelings of the viewer, and the application of aesthetic criteria must take into account the style and location of a bridge. In order to assess the bridge I will consider the ten areas of aesthetics suggested by Fritz Leonhardt as important when considering a bridge.

The structural form of the bridge should be apparent to an observer, and in this the Royal Gorge Bridge is fairly successful as the large catenary cable between steel towers clearly shows the load path, and the deck is very shallow leaving little doubt as to which elements carry substantial load. The wind cable system also seems to aesthetically fulfil its function as it splays to the sides of the bridge at either side, making its stabilising effects apparent to the observer.

The proportions of a bridge are key to the perception of the bridge. The deck appears extremely slender when in proportion to the size of the gorge, together with the steel truss structure this gives the appearance of the bridge being very much floated across the gorge, due to its light weight and little impact on the gorge. Whilst the deck itself could appear too slender, the additional presence of the wind cables provides a lightweight depth to the bridge.

Due to the light mass of the bridge structure there is a very definite balance between this and the void below. Whilst this may not immediately appear to be the most ideal situation, when it is considered that the bridge is designed primarily as an attraction in itself then the added excitement and impact fits the purpose of the bridge. Unlike many suspension bridges the towers of the Royal Gorge Bridge appear fairly short in relation to the span of the deck. This appears slightly

Mr S Cope, spic20@bath.ac.uk
counter intuitive as it will result in higher stresses in the main cables. It does however increase the appearance of the bridge as being a slender structure atop the gorge.

The lines of a bridge should have order and this can help to provide a design which flows to the eye. The gorge bridge has many protrusions and breaks in the line of the deck. The look could possibly be enhanced by placing these flush to the sides of the deck. The hangers are also at a slight angle and this can result in a crossing of cables when viewed from certain angles, although the visual affect of this is small and unobtrusive.

The columns are tapered along their length which helps to reduce the illusion of the top of a parallel edged column appearing wider, and therefore illogical. This tapering reflects the lateral load carried by the column and helps to spread the load to the foundations whilst picking up the load at a single point at the top of the column.

The steel structure of the bridge is a very definite contrast to the rural surroundings. The majority of surrounding buildings are timber. The timber bridge deck however helps to integrate the bridge with its environment. This difference in styles does help the bridge to stand out which aids its purpose as an attraction.

The texture of the bridge could be regarded as fairly poor due to the simple white painted steel structure. However when viewed from close up it is possible for the pedestrians to touch various components of the bridge such as the main cable and wooden deck. Because the main cable is unsheathed this produces a raw, exposed texture that is not found on many suspension bridges.

The main cables and wind cables are dark coloured and so difficult to see from many angles. This results in the white painted deck appearing to be incredibly slender, stretching across the gorge between the white painted towers. This accentuates the idea of the bridge being a light object between the solid rock abutments.

The Royal Gorge Bridge makes a point of displaying exactly how the bridge works, to the extent of showing the cable anchor adjacent to the roadway. This gives the bridge a very open character. Much of the character of the bridge is gained from some of its features which would be considered undesirable in most bridge designs. The wooden planked deck allows pedestrians to see down gaps between the boards to the gorge floor hundreds of meters below. Also the fact that the deck sways in windy conditions increases the slightly wild and exciting character of the bridge which fortunately can be considered a positive feature for a park attraction.

Despite the fairly simple basis for the bridge the design does still have a fair amount of complexity. The wind cables are a feature found on very few suspension bridges and as such add an extra level of complexity to that expected from a suspension bridge.

The natural environment around the gorge is predominantly of bare rock with sparse vegetation. The painted steel structure contrasts with this natural surrounding, and so does not integrate nature in the way that Leonhardt describes. However this contrast does create a strong aesthetic impact which is considered by some to be pleasing to the eye.

2.2 Conclusion of Aesthetics

The Royal Gorge Bridge does not meet several of Leonhardt’s principles and it is unlikely that the braced open steel frame used for the towers is the most aesthetic choice. However despite this, whilst perhaps not beautiful, the resulting bridge is still a striking and impressive design which aesthetically suits the park surroundings.

3 Design

Normally it is unlikely that a cable supported bridge will be used to cross a valley as the towers often would be required to be very tall in order to gain sufficient founding strength. The Royal Gorge Bridge has the towers situated at the edge of the gorge, which is only possible due to the evident high strength of the granite rock which makes up the gorge walls. If the walls were less strong then there would be a risk of slip occurring due to the vertical load which is applied on the edge of the gorge.

The location above a gorge ensures that the bridge will be subjected to gusting winds, making the design of a suspension bridge considerably more complex. For this reason the design uses wind cables to stabilise the deck. Especially before the refurbishment and resulting repositioning of the wind cable system, the deck has been found to sway considerably during windy conditions. However the lack of plated sides gives a very thin area with which to catch the wind. The use of wind cables allows the design to be free from the more usual trusses for lateral bracing, therefore improving the aesthetics and providing resilient and dependable stability to the bridge deck.

3.1 Alternative Bridge Types

There are several possibilities for alternative choices for a bridge in this location. An arch bridge is often used for steep sided valleys due to the suitability for the abutments. However in this case it would be
extremely difficult to construct even the formwork for
the bridge due to the inability to reach down to the base
of the gorge. Aside from the construction difficulties
the bridge would also be less striking, which is an
important factor for an attraction. On the other hand an
arch bridge could be made to fit in with the natural
surroundings if this were desired.

In order to construct a truss or cantilever deck
across the gorge a deep section would be required due
to the lack of possibility for intermediate supports.
Although this would certainly be a striking image, it
would not have the aesthetic appeal of the suspension
bridge as it would appear too heavy and large for such
a location.

The use of a cable stayed bridge would certainly
be worth considering, although the piers would require
placing unusually near to the ends of the span in order
to be footed. In this situation it would be necessary to
take the back stays to anchor blocks and the cables
would be under very high tension. Whilst this would be
an option, the first use of cable stayed bridges was in
1952 in Germany, 23 years after the Royal Gorge
Bridge construction.

3.2 Design Conditions

The location of the bridge was determined by the
existing park on one side of the gorge, and the desire to
expand this attraction by providing transport from one
side to the other. Due to the location within an
attraction park the bridge is privately owned, and has
very little vehicular traffic. The bridge is designed
though to take both vehicular and pedestrian traffic.

3.3 Foundations

Generally the foundations for a suspension bridge
tower will be very large, reaching deep to provide a
sufficiently strong base. However at the Royal Gorge
the high strength granite from the surface allows
relatively shallow foundations. Concrete abutments set
into the surrounding rock are used as foundations for
the towers. These are very much placed to fit with the
existing rock layout, resulting in each pier having
unsymmetrical towers.

As already mentioned, the piers being situated so
close to the edge of a nearly shear cliff brings a risk of
slip of the foundations and potential catastrophic
failure of the bridge. This method of footing the piers is
only possible in this case due to the high strength of the
granite rock which makes up the gorge walls.

3.4 Piers

The piers, or towers, are of a lightweight steel
truss type consisting primarily of bolted angle sections.
Gusset plates are used to connect the cross bracing
members, and these too are bolted to each receiving
surface. Where the cross bracing members meet a
gusset plate has been used to provide continuity for the
crossing members. This bracing provides stability in
the two vertical directions. Horizontally stability is
provided in each individual tower by steel rods bolted
to angle sections in the corners of the tower. Also,
between towers, above the roadway, there are
horizontal angle sections providing cross bracing.

Figure 3: Braced steel towers

The choice of steel piers appears to be an early use
as steel truss piers did not come into mainstream use
until the George Washington Bridge in New York in
1932.

3.5 Main Cables

The main cables are approximately 230mm in
diameter and are each made up of 2100 galvanised
steel wires with a specified ultimate strength of
827N/mm$^2$. This is a strength of 0.83GPa, less than the
normal strength specified today for structural cables of
between 1.3 and 2.2GPa. The cables are unusual as
they are not protected by circumferential wires
wrapping the cable. This can be attributed to the mild
conditions offered by a favourable climate and lack of
pollutants in rural Colorado.

3.6 Cable Anchors

The main cables must be anchored to foundations
on either side of the bridge. This allows the tension in
the cable to be transferred into the ground. Due to the
large forces present in the main cable these anchors
must be securely fixed into the bedrock. The presence
of solid granite at the surface aids the positioning of the
anchors as they can be placed very shallow whilst still
obtaining the required strength. The original main
cable anchors were extremely unusual but very cost
effective.

In order to provide anchorages trenches were cut
for each cable at the same angle as the backstay for
approximately 23m and to a depth of 7.6m. The
trenches were around 1.2m wide and had two lines of
holes drilled 0.5m apart. 100 of these holes in each
trench then had 1m sections of 50mm steel pipe
grouted into them. 21 of the 2100 wires in each cable
were then wound around each length of pipe. With all
the wires connected in this way the trench was filled
with concrete, securing the wires and anchoring the
cables.

However as discussed later this was found to have
degraded upon inspection and was replaced with a
more conventional system to each side of the original cable anchor.

3.7 Hangers

Originally the hangers consisted of forged steel rods which were pin connected at each end to the deck and bands on the main cables. The hangers now are made up of rope suspenders which are pin connected to the main cable bands and deck with closed end fittings. The rope is approximately 40mm steel rope, and the hangers are spaced at approximately 3.0m centres, connecting to every primary beam. Because the main cables are further out than the deck at the piers the hangers are at a slight angle to reach the deck, although this angle is less prominent towards the centre of the bridge as the main cable is pulled in slightly.

3.8 Deck

The deck is of a simple makeup, with the timber deck supported by steel joists which run lengthwise to the bridge. These joists are supported by perpendicular beams which are held by the hangers. The only substantial longitudinal members are a single steel beam either side of the deck supported by the perpendicular beams. There are 1292 timber planks along the length of the deck, of these around 250 require replacing each year. Although this will require the bridge to be temporarily closed, the system of timbers bolted to the deck allows simple and quick replacement of only those timbers which are no longer suitable.

The deck provides some stiffness such that a concentrated load, such as a single vehicle or large group of people, is spread between several hangers.

3.9 Bearings

The deck appears to be vertically supported at the piers, with steel C sections running between the towers, below the deck. This vertical load carrying capacity of the pier members is clear from their size relative to the non-load carrying members. However the small flanges on the C section indicates that there is no horizontal fixing of the deck and so it is assumed that a vertical simple support only is provided.

3.10 Wind Cables

A wind cable system was added to the bridge shortly after opening, although it must be assumed that it was always part of the intended design to include a wind cable system connected to the otherwise very unstable deck. Wire rope wind cables are anchored to the canyon walls on either side of the bridge and arc in towards the deck at the centre of the span. The cables were originally 38mm diameter galvanised wire rope with 13mm diameter wire rope being used to tie the cable to the deck. These wind cables are connected to every second primary beam along the length of the deck, and are pre-stressed to ensure that neither they nor the main cables will become slack.

4 Construction [8]

The bridge was constructed using mostly inexperienced local labour in 1929 at a cost of around $350,000. Impressively, construction took just seven months without a single major injury.

The abutments are assumed to be relatively shallow due to the rock footing and small physical size of the abutment. They were constructed by filling trenches in the rock with concrete produced using crushed granite from the cable anchor trenches. Steel for the piers and cables was produced and fabricated locally before being transported to site.

In order to bring the cables into place, firstly half inch steel cables were lowered into the gorge from each
At the bottom the ends of these cables were spliced, and then these cables pulled back to the level of the bridge. The resulting cable across the gorge allowed three quarter inch cables to be pulled across the gorge. Each of the 2100 wires in each cable were then pulled along this cable individually. The resulting main cable had its wire rope core and each of the wires in it anchored to one of more than 100 steel pins set into the anchor at the end of each cable.

With the cables set into position steel collars were clamped around the cable to allow the connection of hangers to the steel I beams which support the deck. Onto these the secondary beams and joints were connected and the timber sections fixed at their ends to the steel. This was done each pair of hangers at a time, working out from the end of the bridge as shown in Fig 8.

The original wind cable system was designed primarily to suit the topography of the available anchor points. As a result of the varying outcrops of rock the cable system on one side of the bridge was not symmetrical to the other. When inspected it was found that the tension in the cables had been lost, and some sections were slack. This resulted in large movements of the otherwise unstiffened deck during high wind conditions.

The failure of the wind cable system also resulted in compressive forces being applied to the hangers during high winds. The pin connected steel rods were not designed to take these forces and so this had resulted in buckling failures and fatigue cracks in many of the rods. These failures required the replacement of all hangers.

As a result of the inspection the bridge was found to be mostly in good condition, but in need of replacement of the wind cable system, hanger rods and main cable anchors.

5.2 Rehabilitation

Due to the good condition of the main length of cable, it was decided to keep this cable, just replacing the cable ends and anchors. In order to allow this temporary stays were connected to the cables at around 36m from the anchorages and secured by rock anchors. These were to provide a temporary safety measure against failure whilst the refurbishment took place. The previous anchors were left in place, with new anchorages placed around them. These consist of rock anchors in the granite connected to structural steel plates which are embedded in concrete.

The cable was replaced by a series of parallel wire strands, each one made up of 61 galvanised bridge wires, each one 3.8mm diameter and of strength the same as the original wires. These strands are in an open strand socket at one end and pin connected to the new anchor. The other end was brought 30m from the anchorage where groups of 61 wires were peeled off the existing cable, and each wire spliced to a new wire using splice ferrules, protected using a heat shrink sleeve. This technique required almost 8200 wire splices to be made, however the connections both to the cables and the anchors are completely open for inspection and maintenance whilst providing a free run off for water.

The hanger rods were replaced with wire rope suspenders which are pin connected between the cable and deck. These will be much less susceptible to damage through loss of tension, although this is now much less likely to occur.

The wind cable system was completely redesigned for the refurbishment, and as such is now symmetrical. This was achieved by providing a steel tower adjacent to the bridge which provides an anchorage for the new system. The new wire rope cables are now tensioned correctly, this ensures that all the cables required are
sufficiently pretensioned to avoid any sections becoming slack.

The tower is constructed on a new concrete foundation and has both stay cables and a steel truss to hold it in position. Due to the position of this tower it had to be constructed entirely by hand.

![Wind cable tower](image)

**Figure 9:** Wind cable tower

6 Structural Scheme Assessment

The Royal Gorge Bridge uses a typical suspension bridge structural scheme. In this case loads on the deck are transferred via the hangers to the main cable. This puts the main cable into increased tension. This tensile force is carried to the cable anchors, as well as subjecting the piers to a compressive force. This is an extremely efficient load carrying technique as the steel cable elements are in direct tension only, and as the cable is allowed to move at the top of the pier then the piers will be subjected to a vertical compression only.

The wind cable system provides stability and stiffness to the deck, allowing it to be extremely light and simple, its only requirement to carry the applied loads to the hangers. Often a major issue would be the strength of the cable anchors and resistance to sliding when subjected to prolonged horizontal forces. In this case the granite rock provides the opportunity to use a simple and low cost anchor which carries the applied loads. This same strong granite foundation allows simple pier abutments to be constructed extremely near to the gorge edge without issue.

7 Loading [2]

All bridges are checked at the Ultimate Limit State (ULS) and Serviceability Limit State (SLS). The bridge design will be checked using BS 5400.

The worst case load for the ULS must be checked for each element of the bridge. The ULS will be considered for the assessment of the bridge for the purpose of this paper. The SLS will not be investigated further as the resulting loads are lower than in the ULS, and it is outside the scope of this paper to calculate the actual and permissible deflections necessary for a useful SLS check.

Reference [3] states that the load the bridge has been designed to is approximately equal to two lanes of H-15 loading defined by the American Association of State Highway and Transportation Officials (AASHTO). This is equivalent to a medium sized truck with a load of 107kN per axle, where the truck has three axles. A truck this size can be considered an HB load for the purpose of this bridge. As this is the intended strength of the bridge checks will be carried out for two lanes of an equivalent 22 units of HB loading, with only three axles considered.

Creep does not occur in steel structures and so will not be considered.

7.1 Load combinations

In order to find the worst case of loading it is necessary to consider the effects of dead, superimposed dead, live traffic, wind, temperature and other loads.

It is important to find the worst case of loading in order to ensure that the bridge is being analysed against the most testing possible load combination. The combinations to be considered are:

1. All permanent loads and primary live loads.
2. Loads from 1, and wind.
3. Loads from 1, and temperature.
4. All permanent loads plus secondary live loads and associated primary live loads.
5. All permanent loads and loads due to friction at support.

7.2 Dead Loading [4]

The dead loading of the deck is made up of the primary beams which are connected to the hangers, secondary beams running the length of the bridge and joists which also run the length of the bridge.

A partial load factor ($\gamma_f$) must be applied to the dead load and for steel this is always 1.05 for ULS. A further factor ($\gamma_0$) allows for inaccuracy in the analysis and for steel is always 1.10 for ULS.

There is little information available on the sizes of steel which are used in the structure, and so therefore it is necessary to make some assumptions of the dead load present. The steel present is assumed to be as follows:

- **Primary beams** – 406x78x54 I beam section at 3400mm c/c, 5500mm each. Therefore 2.95kN each, 0.90kN/m.
- **Secondary beams** – Because there are no flanges visible it is assumed that a C section has been used. Assume 380x100x54 C section. There is one either side, therefore total loading is 1.10kN/m.
- **Joists** – There are seven joists across the width of the deck. Assume 254x102x22 I beam sections. Therefore total loading is 1.55kN/m.

Therefore the total unfactored dead load is 3.55kN/m of the bridge. Applying the factors gives:

$$\text{Dead Load}_{\text{ULS}} = \gamma_f \times \gamma_0 \times w_{se}$$

$$= 1.05 \times 1.10 \times 3.55$$

$$= 4.10kN/m$$

7.3 Super Imposed Dead Loads [4]
7.4 Live Traffic Loading

The live loading depends predominantly upon the number of carriageways, in this case the carriageway width is 5.5m, between 4.6m and 7.6m, and so is to be considered to have two notional lanes.

Because the loading from vehicles is generally a much worse case than pedestrian loading it becomes the most important factor to consider, especially on a light weight suspension bridge.

The existing HA and HB load cases consider the loads possible from uniformly distributed, heavy, fast moving traffic with impact factors. Due to the 10mph speed limit and lack of traffic on the bridge the loads obtained from the codes will be likely to be highly conservative.

HA loading is considered as a UDL in kN per meter of the lane, and due to the span of 270m should be taken as 10.6kN/m per lane. The total loading per meter of the bridge can then be taken as 21.2kN/m, as the worst case loading is obtained by considering two lanes of full loading. A knife edge load of 120kN should also be applied per lane of the bridge, in the worst case position.

Due to the size of the bridge and its use it is to be considered that while it is possible that a load case to be considered HB loading could be brought upon the bridge, no other live vehicular loads would be present on the bridge at the same time. As indicated it is to be considered that two lanes of HB loading will be considered, as this is equivalent to the original design loading, however this should be conservative as it is highly unlikely that two trucks will both use the bridge at the same time, and indeed unlikely that they would be able to pass each other on the narrow deck. To consider two HB loads in the assessment may be unconventional but is considered useful as this is the closest approximation to the original design value possible and in any case will be more conservative than an approach following BS 5400 more precisely. As defined in the standards there should be a clear distance of 25m in front of and behind the HB loading and so this could be extended to assume that whilst crossing the private bridge the only loading would be from the two HB loads. For HB loading normally 45 units would be considered, which corresponds to 112.5kN per wheel. However as this is an extremely large load it is definitely more appropriate to consider a reduced HB load case. A value of 22 units of HB loading corresponds to the AASHTO H-15 loading and approximates to the original calculations. This is more realistic, and is in fact probably still very conservative. Using this value a loading of 220kN acts on each axle, where there are three axles. The length of the HB truck is variable and must therefore be chosen to give the worst case loading on the bridge.

The deck will carry a total load of 660kN over the length of the truck, which can vary between 7.8 and 27.8m.

7.5 Additional Traffic Loading

The bridge deck is straight and so no centrifugal loading is considered.

Due to the especially slow speed of travel over the bridge it is not appropriate to consider skidding loads, however longitudinal braking forces must still be considered. Braking forces should be considered separately for both HA and HB loading. Under HA loading a value of 8kN/m is considered along one lane in addition to a single 200kN force. Due to the total length of the deck of 366m this results in a total horizontal force of 3130kN. This value seems to be extremely high and it seems reasonable to reduce this value due to the exceptionally low volume of traffic which uses the bridge, as there are rarely more than two cars on the bridge at any one time. Also the extremely low speed limit of 10mph will limit the braking force applied.

A more reasonable force may be found by considering the length of deck which is likely to have vehicles braking along it. If it is assumed that a length of deck of 50m is the most that would be loaded to this extent at any one time then a value of horizontal force of 600kN is obtained. Whilst still high this gives a more accurate representation of a load which the bridge is likely to encounter.

The horizontal force due to HB loading is 25% of the total HB load, which is equal to a total horizontal force of 250kN. This is substantially less than the braking force due to HA loading and so does not need to be considered further.

By inspection it is apparent that the parapet has been designed to withstand small loads only, and not the 25 units of HB loading which is suggested in the standards. The parapets do have a bracing member though which gives them sufficient lateral strength to withstand any likely slow speed impacts from vehicles.

7.6 Pedestrian Loading

Because the magnitude of pedestrian loading will be substantially less than that from vehicular traffic it is not necessary to consider the effects of pedestrian
loading in addition to vehicle loading. One situation where the magnitude of pedestrian loading may approach that subjected by HB loading is when subjected to crowd loading. The Royal Gorge Bridge hosts the annual Royal Gorge Go Fast Games where bungee and base jumpers launch themselves from the centre of the span. This results in a concentration of pedestrians spectating from the centre of the bridge. Although this results in clearly visible deflections of the deck, the dynamic loading should be small due to the unsynchronised movements of the crowd.

Figures 10 and 11: Deflection under crowd loading

One aspect of pedestrian loading which may need to be considered is forced vibrations due to intentional damage of the bridge. It is however unlikely that sufficient oscillations or vibrations could be set up to cause damage in a bridge which has been designed to carry vehicular traffic. A sensible assumption is that all bearings and bolts are sufficiently robust and strong to carry any such loads which could be applied.

7.7 Temperature [4]

In order to assess the effect of temperature on the stresses in the bridge it will be assumed that the entire bridge cross section increases in temperature by 25°C. The coefficient of expansion for steel is $12 \times 10^{-6} \, / ^\circ C$, and for a bridge of the length being considered this can result in relatively large resulting movement or stresses. As stated it is assumed that the piers have vertical supports only, and that the deck is fixed at one end and free to move longitudinally at the other end. This layout of connections should allow any forces due to thermal expansion to be shed by moving the free end of the deck. This situation will be considered first.

If the deck is fixed at one end and then the far end is free to move, the total expansion of the bridge deck must be allowed by a movement joint at the free end.

The expansion along the 366m length of the bridge is given by

$$\Delta l = \alpha \times \Delta t \times l$$  \hspace{1cm} (3)

where

$\Delta t =$ Change in temperature (assume 25°C)
$\alpha =$ Coefficient of thermal expansion for steel
$l =$ Original length of deck

$\Delta l = 12 \times 10^{-6} \times 25 \times 366000 = 109.8 \text{ mm}$

This is a large movement to be carried by the expansion joint. Due to the type of deck it is not appropriate or fitting to use a toothed joint, as would commonly be used for a more common deck surface such as Tarmac. Instead a thin steel plate has been fixed to the road surface on the land side, and appears to simply rest on the timber deck. This is assumed to allow the deck to slide underneath it, and so allow changes in deck length without compromising the interface with the continuing road.

Figure 12: Simple movement joint

This system is suitable for the conditions found, with little vehicular traffic, however it is likely that this would not be an applicable system to an interface with heavier traffic as the wear on the surface may be too great.

Due to the apparent ability of the flexible deck to deform vertically under load, it may be that some of the expansion is not shed at the expansion joint, but is taken up by a slight increase in curvature along the deck length. This will however increase stresses in the cables as well as small bending stresses in the deck.

Although expansion along the surface of the deck is catered for by the steel plate, it is also essential to provide a bearing for the deck structure which allows longitudinal movement, whilst still preferentially providing vertical and lateral support. It is assumed that a bearing has been provided to fulfil these functions in the same location as the steel plate. However it must be considered that this bearing can become jammed or blocked and this can result in the longitudinal force being taken by the deck.

The maximum stress in the deck can be found by multiplying the strain by the Young’s Modulus, $E$.

$$\sigma = \Delta t \times \alpha \times E$$  \hspace{1cm} (4)

$$\sigma = 25 \times 12 \times 10^{-6} \times 200000 = 60 \, \text{N/mm}^2$$

7.8 Wind

The wind loads are considered to BS 5400, the maximum speed of wind gust ($v_c$) to strike the bridge is given by

$$v_c = v \times K_1 \times S_1 \times S_2$$  \hspace{1cm} (5)

where

$v =$ Mean hourly wind speed
$K_1 =$ Wind coefficient
$S_1 =$ Funnelling factor
$S_2 =$ Gust factor

The mean hourly wind speed is assumed to be 36m/s, with an interpolated wind coefficient of 1.70. The funnelling factor will be taken as 1.10 due to the location in a gorge. The gust factor is 1.96 as
The maximum gust wind speed is:

\[ v_c = 36 \times 1.7 \times 1.1 \times 1.96 \]  
\[ v_c = 132 \text{ m/s} \]  

This value of gust speed can be used to calculate the horizontal load \( P_t \) on the bridge

\[ P_t = q \times A_1 \times C_D \]  

where

\[ q = 0.613 \times v_c^2 \]  
\[ A_1 = \text{Solid horizontal projected area} \]  
\[ C_D = \text{Drag coefficient} \]

When calculating the value of \( A_1 \) the depth of the section to be considered should be the depth of the deck, plus 2.5m above the deck to allow for the live loading. Therefore the total depth of the section to be considered is 2.8m. If the total length of the bridge is considered between piers then this should produce the most adverse affect. This gives a value for \( A_1 \) of 2.8 x 268 = 750m\(^2\).

The drag coefficient is dependant upon the ratio of the bridge depth to the width. This ratio is found to be \( b/d_L = 5.5/2.5 = 2.2 \). Using Figure 5 from BS 5400-2:2006 the value of \( C_D \) is then found to be 1.47.

Therefore the total horizontal load on the bridge is as below

\[ P_t = 10681 \times 750 \times 1.47 \]  
\[ P_t = 11775 \text{ kN or 44kN/m} \]

It is also important to consider the possibility of a vertical wind load, \( P_v \), acting on the bridge. This is found from the equation

\[ P_v = q \times A_3 \times C_L \]  

where

\[ A_3 = \text{Plan area} \]  
\[ C_L = \text{Lift coefficient} \]

The plan area of the bridge between the piers is 268 x 5.5m giving a value for \( A_3 \) of 1474m\(^2\). The lift coefficient can be taken as 0.4.

\[ P_v = 10681 \times 1474 \times 0.4 \]  
\[ P_v = 6298 \text{ kN or 23.5kN/m} \]

Longitudinal loads due to wind will also be applied to the bridge, however these will be neglected for the purposes of this analysis. There will be a small longitudinal load applied to the towers, however their open truss design will ensure that this load is small. The other main contributor to longitudinal loading is that from vehicles on the bridge. However as it is extremely unlikely that there would be enough vehicles on the bridge at one time to generate any substantial load it will be assumed that this load is negligible compared to the loads applied due to braking of vehicles.

8 Ultimate Limit State [2]

8.1 Hanger Cable check

Firstly the maximum load on a hanger cable will be calculated. The worst case load combination is found when using the second combination of loads where dead, superimposed dead, traffic and vertical wind loads are considered. It is assumed that due to the flexible deck each pair of hangers carries the load applied from the midpoint of the adjacent spans.

In this case the factored loads to be applied for this ULS combination are 4.10kN/m for dead loads. The superimposed dead load is 11.00kN/m. As the span between hangers is taken as 3m it is only possible for two of the axles of an HB load to be applied and so a total load of 440kN will be applied in each lane. \( \gamma_b \) and \( \gamma_f \) factors each of value 1.10 must be applied to this value so that the total factored HB load is 535kN.

The wind load contribution is 23.5kN/m but this must also be factored. The wind load also has \( \gamma_b \) and \( \gamma_f \) factors each of value 1.10 and so the final wind load is 28.5kN/m.

Therefore for each pair of hangers the total load to be carried, \( Q \), is

\[ Q = 4.10 \times 3 + 11.00 \times 3 + 1070 + 28.5 \times 3 \]  
\[ Q = 1201 \text{ kN} \]

The force in each hanger is 601kN. Assume that the steel suspender has a yield stress of 760N/mm\(^2\).

The area of rope, \( A \), required to carry this load is given by

\[ A = Q / \sigma \]  
\[ A = 601000 / 760 \]  
\[ A = 791 \text{ mm}^2 \]

This corresponds to a cable diameter of 32mm. This is slightly less than the 40mm assumed as the cable diameter. Some of this discrepancy may be explained by the additional vertical load that the prestressed wind cable system will apply to the deck. Also dynamic loading of the cables during oscillations due to wind will have been taken into account in more detail than has been covered here.

8.2 Main Cable Check

In order to assess the main cable it is best to estimate the uniformly distributed load (udl) that is acting upon it. The second combination of loads should again be used when assessing the main cable, although it is necessary to check whether HA or HB loading gives the most adverse effect. HA loading has a total unfactored load on the total central span of 5922kN.

This is substantially more than the HB loading of 1070kN.
Therefore the loads to be taken into account are 4.10kN/m for dead, 11.00kN/m for superimposed, 26.7kN/m for traffic and 28.5kN/m for wind. These must be halved to divide the applied load between the two cables. This gives a total UDL of 35.2kN/m.

The horizontal component of force, \( H \), in the cable can be found as

\[
H = \frac{w \times l^2}{8 \times f}
\]

(14)

where

\( w = \text{UDL} \)
\( f = \text{Sag in cable} \)

\[
H = \frac{35.2 \times 268^2}{8 \times 22}
\]

(15)

\( H = 14365\text{kN} \)

The height of the pier above the deck is not known so the total sag is estimated by scaling the total height of the pier to be 22m.

The maximum force in the cables is found at the top of the pier. Here there is an additional component of force in the vertical direction. Because the total vertical force applied to the deck must be carried between the four ends of the main cables in the central span then the vertical load, \( V \), is found to be

\[
V = \frac{w \times l}{2}
\]

(16)

\( V = \frac{35.2 \times 268}{2} \)

\( V = 4717\text{kN} \)

The total maximum force, \( T \), in the cable is then given by using Pythagoras’ theorem

\[
T = \sqrt{H^2 + V^2}
\]

(17)

\( T = 15120\text{kN} \)

The area of cable required to carry this load, using the specified yield strength of 827N/mm² is

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

(18)

\( A = \frac{15120000}{827} \)

\( A = 18283\text{mm}^2 \)

The existing cables are approximately 230mm in diameter, however they can be more accurately assessed by calculating the area of the 2100 3.8mm wires which make up the cable. This area is 23816mm², 30% more than that calculated as required.

8.3 Calculating the compression in the tower

As the main cable leaves the top of the pier at approximately the same angle on each sides, and has the same tensile force, it is then reasonable to assume that the total vertical force carried by the tower, \( V_t \), is equal to twice the previously calculated vertical component of force in the cable. The total vertical force on each tower is then 9435kN, and therefore 18870kN on each abutment.

The use of a single highway with towers either side has the result that it would be extremely difficult and probably uneconomical to attempt to widen the bridge. A more likely choice would be to construct a second bridge beside the current bridge, however this is unlikely to occur due to the large cost for such a project and little demand. The demand for another bridge is extremely unlikely to increase as most people who use the existing bridge do so primarily in order to travel across that specific bridge, making another bridge crossing redundant.

9 Susceptibility to Intentional Damage

Structurally, due to the proximity of pedestrian traffic to main structural elements and lack of protection it would be theoretically possible for intentional damage to be caused. This problem is dealt with primarily by the regular patrols by park and security personnel of the bridge, as well as the time that would be required to inflict serious damage to the bridge. Of more concern is the proximity of vehicular traffic to the piers which have little protection besides the guard rail to the side of the deck. The risk of any problems arising due to impact has been minimised by the enforcement of a low speed limit of 10mph whilst crossing the bridge. It seems likely that this speed limit would be adhered to due to the necessity for the vehicles to share the roadway with pedestrians.

The substructure is protected from damage as it is below the level of the roadway and inaccessible by vehicle.

Acknowledgments

The paper guidelines and course notes provided by Professor Tim Ibell have been used as a basis for this paper.

References