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A CRITICAL ANALYSIS OF BIXBY CREEK BRIDGE

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Abstract: This paper provides a critical analysis of certain aspects of the Bixby Creek Bridge in California. Included is an analysis of the bridge's aesthetics, loading, structure, serviceability as well as a summary of the likely construction method used. The paper includes example calculations where deemed appropriate including analysis of temperature effects, wind loading and creep.

Keywords: *Bixby, Reinforced Concrete, Arch, Monterey, Seismic Retrofit*

1 Introduction

The Bixby Creek Bridge is an open spandrel reinforced concrete arch bridge located in the Big Sur area of California, about 20 miles south of the City of Monterey. Construction began on 27th November 1931 and was completed on 15th October 1932.

It is one of five bridges spanning canyons in the area that were necessary to connect what was then known as Route 56 and is known today as California's Highway 1. Of this group of bridges, Bixby Bridge has the biggest span and is the most well known. Before the construction of the bridge the journey between Monterey and the Big Sur River Valley consisted of a 30 mile detour that took 3 days by wagon on very rough tracks. The bridge is therefore important as it made the area more accessible by car as well as providing an important link for the Highway along the coast. It was also significant at the time as it brought jobs during the time of the great depression, providing some relief to the area. The fact that it is a concrete bridge meant that material costs were low compared to steel and so more of the total cost went straight to the workers, which was seen as very positive at the time by highway officials.

An alternative idea to the bridge was to build a smaller bridge downstream and a 270m tunnel through the valley side, however due to safety concerns and its superior aesthetic qualities, the arch bridge was chosen. A seismic retrofit was carried out on the bridge in 1996 in order to allow the bridge to resist even the highest magnitude earthquakes.

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2 Aesthetics

To analyse the aesthetics of the bridge, Fritz Leonhardt's ten areas of aesthetics from his book 'Brücken/Bridges' will be used as a guideline to assess how successful the designer was in achieving an aesthetically pleasing bridge. However all these criteria do not have to be fulfilled to create a beautiful bridge and it of course, in the end comes down to a matter of an individual's opinion.

Figure 1: Bixby Bridge looking out to sea

The first and probably most important of Leonhardt's areas of bridge aesthetics is that the function of the bridge should be fulfilled, showing clearly and simply how the structure works and therefore giving the bridge a sense of stability. In the central span you can see that the piers transfer the vertical load from the deck into the arches below. In Ref. [1] it is stated that for an arch shape, form follows function. In other words, when the public view the bridge they can see that the purpose of the arches is to transfer the loads through thrust to the sides of the valley. It is clear also that the side spans are supported by the piers which go directly into the side of the valley.

When considering the proportions of the bridge, the very large towers over the arch abutments immediately stand out. They are of considerable size compared to the other piers yet there doesn't seem to be a structural reason for this. This may be down to the designers following the old idea, stated in Ref. [1] that 'an arch has to land on something big and solid' and they wanted this impression emphasised over the full height of the structure. Most modern arch bridges tend to have smaller abutments which are considered superior in terms of both efficiency and aesthetics, such as the Bloukrans Bridge in South Africa. These towers don't, however, take away from the bridge's overall aesthetic appeal. They provide a transition between the main span and the approach spans making it clear to the viewer which part of the deck is taken by the arch and they focus the eye onto the central span which is the most important and interesting part of the bridge. With a fairly low span to rise ratio of 2.7 the arch gives an enhanced sense of height over the 79m clearance below, than a more shallow arch would. The other parts of the bridge appear to have very good proportions. The arches are sizeable enough to look strong without being too dominant. The deck is fairly slender and elegant as well and the piers appear appropriately slender considering the number of them being supported by the arch.

The bridge also has good order. Repetition of piers has been used to good effect to enhance the arch's symmetrical look, without looking too 'busy'. This also distracts from the fact that the approach spans are not symmetrical on either side of the bridge.

Several refinements help enhance the bridge's overall aesthetic appeal. The concrete parapets have a repeating arch pattern which fits well with the rest of the bridge's look. Also where the piers meet the bridge deck, the concrete is joined in a smooth curve which is more appealing than creating a sharp looking edge. The towers use slight tapers and steps to create an interesting shape. This is used to highlight the centre of the towers in elevation as well as where the arches meet the abutments by continuing these lines from the bottom to the top of the towers. The columns are tapered in order to prevent them from looking top heavy and illogical.

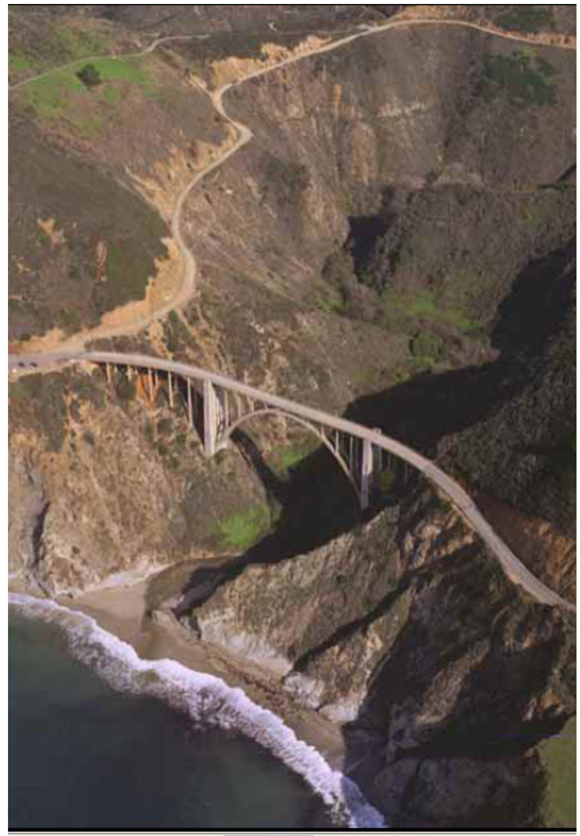

Figure 2: The Bridge from above

Of course it is well known that an arch shape compliments the steep sided valley and enhances the visual effect of rising over the valley, which is why it is used for this application so often around the world with much success. This bridge is no exception. The choice of concrete as a material compliments the surrounding rock of the valley walls and rugged coastline in terms of both colour and texture. A glossy finish with no subtle imperfections would look less integrated. A steel truss bridge would simply look out of place in this natural valley environment. The lighter colour of the bridge provides a subtle contrast to the environment around it which compliments the natural colours of the surroundings, whilst not shying away from the fact that it is a manmade structure. This colour is used for the whole bridge, down to the parapets, and so when viewed no one element stands out. This seems appropriate for this bridge, for instance the arch does not need a difference in colour in order for it to be emphasised.

This bridge certainly has character. With its oversized towers, high rise arch and stunning natural setting it has become very popular with tourists and photographers. Although the function is fairly clear the nature of the two arches with the two sets of piers and supports in between creates enough visual interest to encourage people think about how it works. The importance of the bridge's aesthetic and historical qualities was highlighted when the seismic retrofit programme was carried out. Due to the requirement that the appearance of the bridge should not be affected, a considerably larger amount of money and time were spent than was potentially needed in order to fulfil this. Although the colour and texture of the concrete have a very natural feel to them, there is little sign of nature being used in terms of influencing the design of the structure.

Figure 3: Repeating arch pattern/ tapering columns

2.1 Summary

By fulfilling most of Leonhardt's aesthetic requirements, the designers of the Bixby Bridge have created a truly great bridge. By using texture, colour and most importantly the arch structure, the bridge fits in with the surrounding environment perfectly whilst maintaining the appearance of a bold and iconic structure.

3 Loading

Bridges need to be designed to withstand several different types of loads. Dead loads, superimposed dead loads, live loads, wind loads and seismic loads all need to be considered as well as effects from temperature and creep. The nominal loads are calculated and then multiplied by the relevant partial load factors, γ_{fl} and γ_{f3}, as outlined in BS 5400. These factors vary depending on the type of load being analysed, which load combination is being considered and whether ultimate limit state (ULS) or serviceability limit state (SLS) are being considered. Five combinations of load need to be checked at SLS and ULS. These combinations are listed below.

1. All permanent loads plus primary live loads
2. Combination 1, plus wind, and if erection considered, temporary erection loads.
3. Combination 1, plus temperature, and if erection considered, temporary loads.
4. All permanent loads plus secondary live loads and associated primary live loads.
5. All permanent loads plus loads due to friction at supports.

The carriageway is 7m wide which equates to 2 notional lanes. The density of reinforced concrete will be taken as 2400 kg/m^3.

3.1 Dead Load

Deck = 79.2 KN/m length
Arches: - Each arch weighs 907,200 kg, Ref. [7] = 9 MN.
9,000/98 x 2 = 183.6 KN/m horizontal length
Columns: - 69.4 KN/m height

3.2 Superimposed Dead Load

The superimposed dead load includes the road material and any other permanent loads that aren't part of the original bridge structure. Because these loads are difficult to predict, a corresponding high safety factor is applied.

Table 1: Superimposed loads

Material	Load
200mm saturated sand fill	5 KN/m2
100mm tar	2.4 KN/m2
Services and other superimposed loads	1KN/m2
Total nominal superimposed dead load	8.4 KN/m2

3.3 HA Loading

The carriageway is 7m wide, giving 2 notional lanes.

From **BS 5400-2 6.2.1:** Eq. (1), gives the loading per notional lane due to HA loading.

$$W = 36\left(\frac{1}{L}\right)^{0.1} KN/m \qquad (1)$$

$$W = 36\left(\frac{1}{218}\right)^{0.1} = 21KN/m$$

A knife edge load of 120 KN per notional lane is also applied to give the most adverse effect depending on which element of the bridge is being analysed.

3.4 HB Loading

HB loading is applied to simulate an abnormal truck load as shown in the diagram below. Each axle represents 10 KN per unit of HB loading. Full HB loading is 45 units; therefore each wheel represents 112.5 KN and the truck is 1800 KN in total. The length between the two sets of axles can be varied between 6m and 26m in order to produce the most adverse effect.

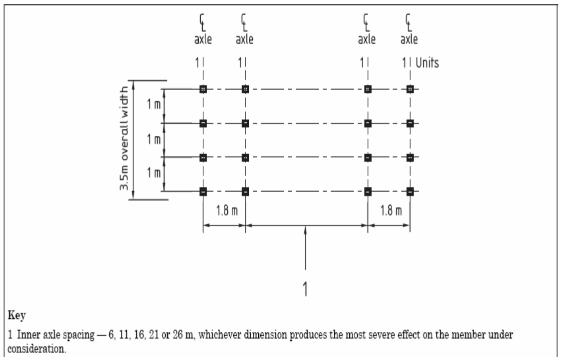

Figure 4: Diagram of HB loading

This HB loading must then be applied with associated HA loading. This can be applied in several different ways. One possible combination is shown below with the HB loading straddling two lanes.

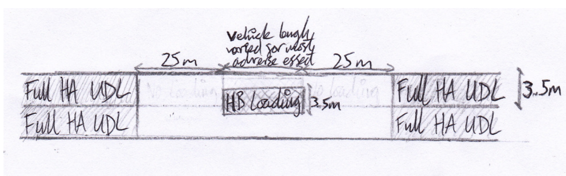

Figure 5: Possible application of HA & HB Loading

3.5 Braking and Acceleration Loads

This can be taken as a horizontal force of 8 KN/m along one notional lane plus a single 200 KN force. Also for HB loading, 25% of the total nominal HB load should be applied over just 2 of the axles. These should be considered along with the associated vertical HA and HB loading.

3.6 Wind Loading

To calculate wind loads a 120-year return value for heights 10m above the ground and up to 300m above sea level are used. To calculate the maximum wind gust, Eq. (2) is used. v is the basic wind speed, taken as 36 m/s [2]. K_1 is the wind coefficient, for 218m length at 80 m above ground $K_1 = 1.74$. There will be some funnelling effects due to the valley so S_1 is taken as 1.1. The gust factor S_2 is taken as 1.42.

$$v_c = vK_1S_1S_2 \qquad (2)$$
$$v_c = 36(1.74)(1.1)(1.42) = 97.8m/s$$

The dynamic pressure head can then be calculated using Eq. (3).

$$q = 0.613{v_c}^2 \qquad (3)$$
$$q = 0.613(97.8)^2 = 5.9KN/m^2$$

The wind loading on different elements of the bridge can then be calculated using Eq. (4). For the deck A_1 considers the bridge as live loaded and therefore takes into account the height of a lorry which comes to 3.9m depth. To find C_D the b/d ratio of 2.8 is used to obtain a value of 1.4.

$$P_t = qA_1C_D \qquad (4)$$
$$P_t = 5.9(3.9)(1.4) = 32.2KN/m$$

The total wind load on the arches, assuming the same value of q as the deck comes to 5.7 MN, using values of $C_D = 2.6$ and $A_1 = 370$ m^2. The wind load on each tower will be 2.6 MN, using $C_D = 1.5$ and $A_1 = 290$ m^2. The piers will also experience wind loading which will vary depending on their height which affects the value for C_D.

Wind uplift or downward force also needs to be considered for the bridge deck. The deck's lift coefficient value is 0.4 and using the width 7m for A_3, Eq. (5) can be used to calculate the maximum uplift or downward force.

$$P_v = qA_3C_L \qquad (5)$$
$$P_v = 5.9(7)(0.4) = 16.5KN/m$$

Eq. (5) can also be used to calculate the uplift on the arches using the same value of q, 1.8m for A_3 and 0.4 again for the lift coefficient.

$$P_v = 5.9(1.8)(0.4) = 4.3KN/m$$

This value is for the crown of the arch and is likely to be different further down due to a different section and different wind speed.

4 Construction

The approach spans were constructed first, most likely using in-situ construction on centring which was a very popular method at this time. Centring is constructed for one span plus for a further fifth of a span at which point the bending moment due to dead weight is roughly zero, as shown in figure 6. The formwork for the second span is then tied to the first casting to minimise differential movement. The concrete is then poured from the right to avoid the joint becoming strained during pouring which can lead to cold-joint cracking.

The arches were then cast in-situ using form work which consisted of 91,000m of timber that was 26 storeys tall as stated in Ref. [3] and went right down to the bottom of the valley. This form work, shown in figure 7, took 2 months to build alone and was clearly one of the biggest challenges that the contractors faced.

Due to it's location over a creek there was obviously no disruption to roads, which is often seen as the major problem when using this method of construction. One of the major difficulties was raising and holding this huge arch frame, made more difficult due to the high winds it was exposed to. On several occasions work had to be halted due to the severity of winter storms as well as swells causing flooding around the base of the formwork.

The removal of the formwork and allowing the arches to take their own weight was a delicate operation. The arch must shorten as it takes up its own self weight, leading to a drop in the level of the crown. Ideally false work should be lowered incrementally over the full span simultaneously so loads due to self weight that could cause damage, aren't applied to only part of the span but are distributed as evenly as possible [4]. This was a difficult task for the Bixby Bridge as the form work consisted of such a large amount of members. In all the bridge took just under a year to construct which is fairly quick, considering the methods used.

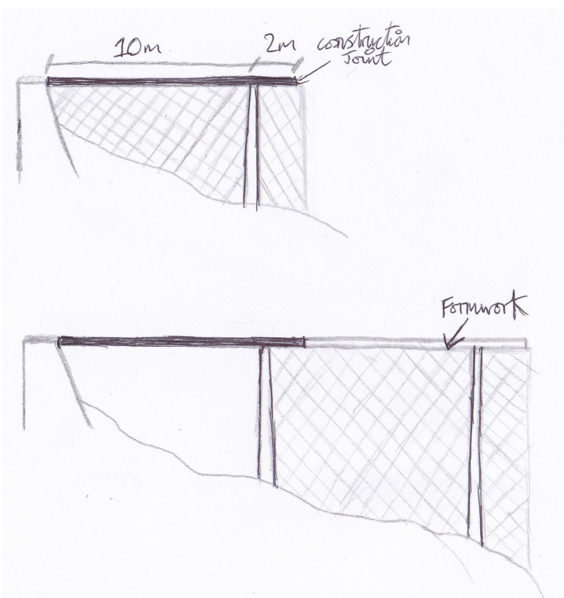

Figure 6: Diagram of in-situ on centring construction

4.1 Materials

Around 3600 m^3 of earth and rock needed to be excavated. 45,000 sacks of cement were used, sourced from Davenport around 70 miles away. The method used to transport the materials across the canyon came from platforms along with slings suspended from a cable 100m above the creek. The creek provided a cheap and easy supply of water for the concrete mix [5].

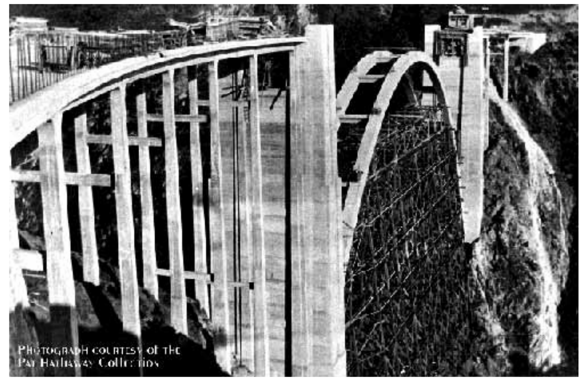

Figure 7: Photograph of arch formwork

5 Seismic

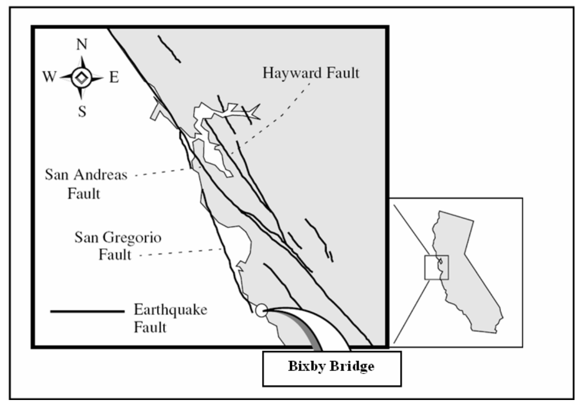

Figure 8: Map showing location of faults nearby [6]

Figure 8 shows the location of the bridge relative to nearby major active earthquake faults. It is clear then that the bridge is located in a high risk seismic zone, along with many other places along the California coast, due to the San Andreas Fault and other minor faults branching off of it. After the Loma Prieta earthquake in 1989 the California Department of Transportation (Caltrans) embarked on a project to retrofit the state's bridges to withstand the highest earthquake loads preventing collapse and any loss of life. For Bixby Bridge this meant a seismic retrofit scheme that fulfilled these criteria whilst also preserving the bridges appearance due to its historical importance. Also, any works were to be staged from the bridge deck due to environmental concerns about the canyons ecosystem. This was made tricky by the requirement that the bridge also had to remain open and so for many months the bridge only had one lane operating. The retrofit was carried out by Buckland & Taylor Ltd and completed in 1996. Using complex linear and non-linear computer analysis it was found that for an earthquake of magnitude 7.0, the struts linking the arch ribs would be destroyed and the ribs would buckle under their own weight. The expansion gaps in the bridge deck and columns would also allow the spans to move independently during an earthquake, overstressing the lightly reinforced columns. Due to the aesthetic requirements of the project the engineers devised a retrofit scheme that provided an entirely new lateral load path for the deck and arch ribs whilst remaining hidden from view. Prior to the works the bridge would have attempted to resist seismic loads through bending of the struts, columns and arch ribs which would have been stressed way beyond their elastic limits. The engineers decided to strengthen and post tension the deck, as shown in figure 9 using heavily confined edge beams dowelled to the inside of the exterior concrete girders which contain high strength steel rods, shown in blue. These were made continuous across the entire bridge length, providing continuity across the expansion joints. In addition four pre-stressing tendons, shown in red, compress the deck giving it the strength to act as a structural diaphragm and therefore in the event of an earthquake it will transfer lateral loads to the newly retrofitted abutments and towers which were also strengthened. A pair of transverse in-fill shear walls were also designed that connect the crown of the arch to the deck, providing the arch ribs with lateral support.

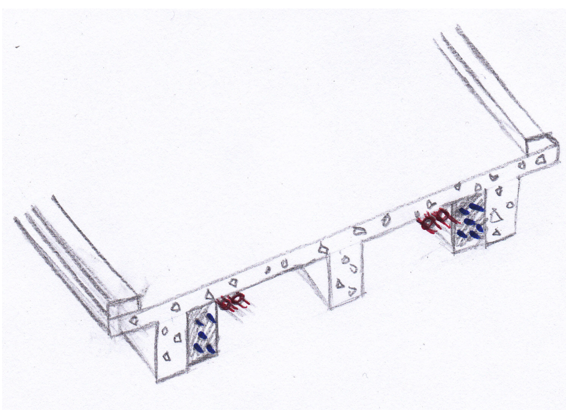

Figure 9: Diagram showing retrofit edge beams and tendons on underside of deck

6 Foundations and Geotechnics

In Ref. [5] it is stated that the slopes of the canyon consist of altered and disintegrated granite. The material is considered competent rock in Ref. [6] and is therefore ideal foundation material for resisting the vertical and horizontal components provided by the thrust from the arches. The bridge has two abutments at the beginning of the approach spans as well as foundations at the base of the towers and columns. The tower foundations resist loads from the bridge deck and the thrust from the arches. During the seismic retrofit scheme in 1996 major changes were made to these foundations as shown in figure 10.

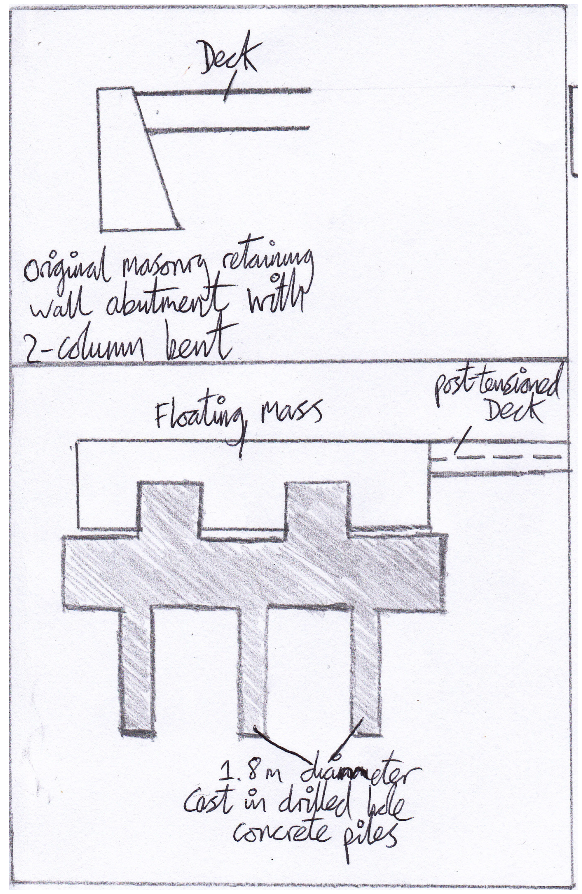

Figure 10: Sketch of old abutments at the top and new abutments below

In Ref. [7] it states that the bridge's original abutments were modified two-column bents with rock masonry retaining walls holding back the roadway fill. These had to be significantly strengthened during the retrofit. As part of the seismic retrofit, the abutments at the end of the approach spans were replaced by pile groups consisting of several 1.8m diameter cast in drilled hole piles in order to accommodate the new floating slab system and re-routing of the lateral load path through the deck which is now designed to act as a diaphragm. The original tower foundations were un-reinforced and well socketed into the canyon walls. 16 tie-down anchors were introduced during the retrofit in the towers foundations, lengthening the moment arm at the towers' base. These anchors have increased their resistance against uplift and seismic overturning moments. For them to provide adequate restraint, the anchors were fitted within holes drilled up to 15m through the foundation block and then 11m into the rock, they were then grouted to the rock, pre-stressed with a force of 4225 KN each and locked off against new 3m deep reaction blocks that fill the base of the towers [6].

7 Temperature

The 9 to 12 m deck spans between the columns are separated by 9.5 mm wide expansion joints that continue down through the top 4 m of each column. These joints allow free thermal expansion of the structure. Banded steel collars were fitted during the retrofit to restrain excessive displacements across the expansion joints during an earthquake whilst still allowing the thermal expansion joints to work as usual [6].

Overall temperature changes in the deck will induce effective strains on the reinforced concrete. The thermal coefficient of expansion for concrete and steel will be taken as $12x10^{-6}$/°C.

For a linear increase in temperature Eq. (6) gives the strain in the deck due to this increase in temperature, taken here as 25˚C. The increase in length of the deck that this causes for each span between the expansion joints is then given by Eq. (7). If the joints were to become blocked and the deck became effectively restrained then a compressive stress will be induced in the deck, given by Eq. (8).

$$\varepsilon_T = \alpha \Delta T \qquad (6)$$

$$\varepsilon_T = 12x10^{-6}(25) = 300\mu\varepsilon$$

$$\Delta L = \varepsilon_T L \qquad (7)$$

$$\Delta L = 300x10^{-6}(12{,}000) = 3.6mm$$

$$\sigma_c = E\varepsilon_T \qquad (8)$$

$$\sigma_c = 30x10^3(300x10^{-6}) = 9N/mm^2$$

If this stress occurred in the deck due to temperature increase it could cause major problems, and therefore highlights the importance of the need for designing and maintaining expansion joints to avoid inducing these stresses. The increase in length due to the temperature increase also justifies the need for the 9.5mm wide expansion joints that exist in the split columns between the decks spans, although it can be seen that they are more than adequate.

Differential temperature variations between the top and the bottom of the deck will induce bending stresses. It is assumed that the temperature varies linearly through the section between +20˚C on the top and -5˚C on the bottom. Figure 11 shows the temperature variation through the deck, the strain profile through the deck due to this temperature gradient and the associated stress that this causes in the concrete. Using Eq. (6) again gives the strains due to temperature change at the top and soffit of the section respectively.

$$\varepsilon_T = 12x10^{-6}(20) = 240\mu\varepsilon$$

$$\varepsilon_S = 12x10^{-6}(-5) = -60\mu\varepsilon$$

Eq. (8) can then be used again to give the stresses in the top and soffit of the section respectively.

$$\sigma_T = 240x10^{-6}(30x10^3) = 7.2N/mm^2$$

$$\sigma_S = -60x10^{-6}(30x10^3) = -1.8N/mm^2$$

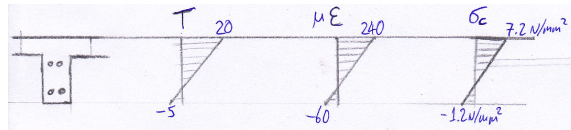

Figure 11: Temperature gradient, strain profile and stress profile of concrete

These stresses can then be used to find the associated forces in the deck. Reinforcement will also be present in the deck from which the stresses need to be calculated separately from the strain profile.

Linear and differential temperature changes will also induce stresses in the arch which also needs be checked during the design process.

8 Strength

8.1 Deck

The bridge deck is considered as a continuous beam between the abutments and towers with internal supports provided by the columns as shown in figure 12. In order to calculate the maximum bending moment, the largest spans of 12m on the approach spans, will be used which will create the largest bending moment in hogging over the column supports, found using Eq. (9). The factored dead and live loads are 149.8 + 50.4 = 200.2 KN/m at serviceability limit state.

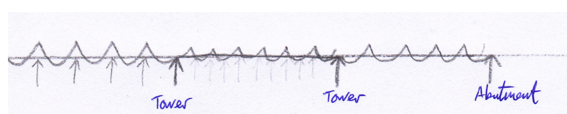

Figure 12: Moments in deck

$$M = \frac{Wl^2}{12} \qquad (9)$$

$$M = \frac{200.2(12)^2}{12} = 2.4MNm$$

This moment causes tension on the top side of the deck section which if excessive, can cause cracking in the concrete and will cause damage to the road surface above. Nowadays concrete is prestressed to limit this effect but this bridge was designed before prestressed concrete was widely available. This check relates to the deck before the seismic retrofit was carried out. The deck section is shown in figure 13. The neutral axis is taken at a depth of 400mm. Eq. (10) gives the stress at the top of the section due to the hogging moment.

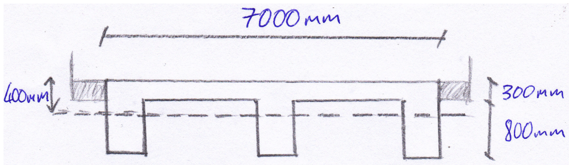

Figure 13: Deck section

$$\sigma = \frac{My}{I} \qquad (10)$$

$$\sigma = \frac{2.4x10^9(400)}{5.41x10^{11}} = 1.77N/mm^2$$

This value is quite high and you would normally expect the concrete's tensile stress to not exceed $1N/mm^2$. Now that the seismic retrofit has been carried out this value is likely to have reduced significantly due to the stressing of the deck.

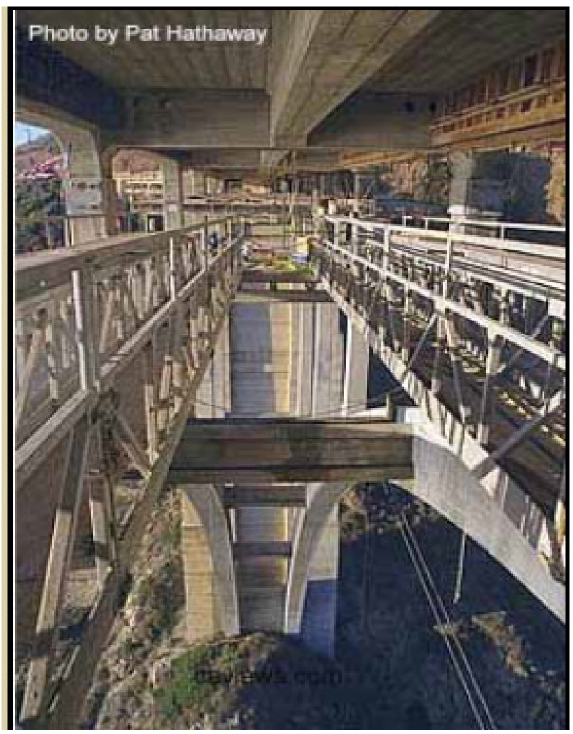

Figure 14: View from under the deck on a temporary platform during the retrofit

8.2 Arch

The reinforced concrete arches are fixed and have a parabolic shape. The maximum positive bending moment at the crown is induced when the middle third of the span is loaded. The maximum negative bending moment at the springing is induced when four tenths of the span adjacent to the springing is loaded. The maximum positive bending moment is induced at the springing when the span except the adjacent four tenths is loaded [8].

Fixed arches are statically indeterminate therefore existing formulas will be used to analyse the arch. The equations in Ref. [9] are used to calculate reactions and moments for reinforced concrete arches. Eq. (11) and Eq. (12) give the horizontal and vertical reactions at the springings due to the dead loads. Each arch is assumed to take half the dead load from the deck. The factored live load acting on one arch is a UDL of 27.7 KN/m. The factored dead load, g is equal to 385.2 KN/m. The K factors in the equations take into account that the bridge is an open spandrel bridge and therefore the load is not uniformly distributed on the arch.

$$H_D = K_1 g l^2 / y_c \qquad (11)$$

$$H_D = 0.156(385.2)(98)^2/36 = 16MN$$

$$V_D = gl/2 \qquad (12)$$

$$V_D = 385.2(98)/2 = 18.9MN$$

An elastic shortening of the arch occurs due to the thrust which in turn provides a counter thrust H_1, calculated in Eq. (13).

$$H_1 = -K_2\left(\frac{h}{y_c}\right)^2 H_D \qquad (13)$$

$$H_1 = -1.05\left(\frac{2}{36}\right)^2 16 \quad = -0.05MN$$

This then causes an eccentricity in the thrust which induces bending moments in the arch under dead loads. Eq. (14) gives the bending moment in the crown due to dead loads and Eq. (15) gives the bending moment in the springings due to the dead load.

$$M_{CD} = K_3 y_c H_1 \qquad (14)$$

$$M_{CD} = 0.271(36)(-0.05) = -0.5MNm$$

$$M_{SD} = (K_3 - 1) y_c H_1 \qquad (15)$$

$$M_{SD} = (0.271-1)(36)(-0.05) = 1.3MNm$$

Eq. (16) gives the maximum possible moment in the crown due to the live load, when the middle third of the span is loaded.

$$M_{CL} = K_5 q l^2 \qquad (16)$$

$$M_{CL} = 62x10^{-4}(27.7)(98)^2 = 1.6MNm$$

Eq. (17) and Eq. (18) give the maximum negative and maximum positive moment at the springings due to live load respectively.

$$M_{SL} = K_7 q l^2 \qquad (17)$$

$$M_{SL} = 0.017(27.7)(98)^2 = -4.5MNm$$

$$M_{SL} = K_{10} q l^2 \qquad (18)$$

$$M_{SL} = 0.022(27.7)(98)^2 = 5.9MNm$$

The maximum moment will therefore occur under the loading condition shown in figure 15 at the springing, giving a total moment of 5.9 + 1.3 = 7.2 MNm. Eq. (19) and Eq. (20) give the horizontal and vertical thrusts at the springing under the live load.

$$H_L = K_{11} q l^2 / y_c \qquad (19)$$

$$H_L = 0.094(27.7)(98)^2 / 36 = 0.7MN$$

$$V_L = K_{12} q l \qquad (20)$$

$$V_L = 0.159(27.7)(98) = 0.4MN$$

The total horizontal force at the springing under this load condition = 0.7 + 16 – 0.05 = 16.65 MN and the vertical force is 18.9 + 0.4 = 19.3 MN.

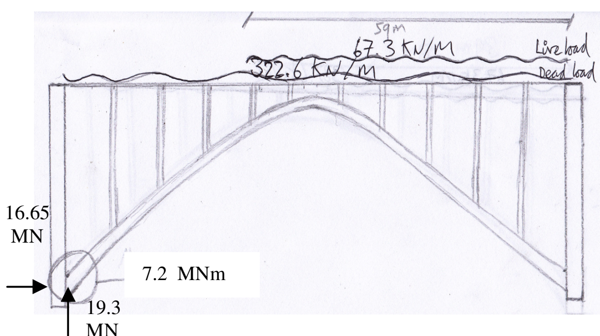

Figure 15: Arch worst case loading , moments and reactions

9 Creep

Creep occurs in concrete over time due to constant dead and superimposed dead load. This slows down over time, with most of creep occurring in the first 28 days after the concrete is formed. The total amount of creep can be estimated as roughly 3 times the elastic deformation due to dead weight. The elastic deformation of the deck due to dead weight is given in Eq. (21).

$$\delta = \frac{Wl^4}{384EI} \qquad (21)$$

$$\delta = \frac{149.8(12000)^4}{384(30x10^3)(5.41x10^{11})} = 0.5mm$$

Therefore the deflection due to creep over time, in the deck is likely to be 1.5mm which is very small over a span of 12m.

10 Durability and Vandalism

The bridge is an important asset both historically and practically to the Californian Transport Authorities. They clearly intend for the bridge to last for many years to come, highlighted by the $20 million spent on the seismic retrofit. It is therefore important for both the structure and aesthetic considerations to be regularly inspected and maintained if necessary. The last full inspection, carried out in July 2007 returned the results shown in table 2.

Table 2: 2007 inspection results [10]

Deck condition rating:	Satisfactory (6 out of 9)
Superstructure condition rating:	Fair (5 out of 9)
Substructure condition rating:	Good (7 out of 9)
Appraisal:	Functionally obsolete
Sufficiency rating:	50.0 (out of 100)

The appraisal rating of functionally obsolete refers to the suitability of the bridge to meet traffic demand, which since the width of the bridge is less than 9.8m it officially falls into this category [11].

Since the bridge is a fairly popular spot for tourists because of its aesthetic qualities, keeping it clean of graffiti is important. However, most of the bridge is inaccessible to people without them risking their life. The parapet's are about the only part of the bridge at risk and since there is no footpath on the bridge and most people view it from the valley side there is a small chance that this form of vandalism will occur compared to a pedestrian bridge in an urban area. In terms of weathering of the concrete, adequate cover should be provided to protect the steel reinforcement for many years. The weathering of the concrete has not had a negative effect on the aesthetics and has actually helped give the bridge its rustic look.

11 Future Changes

The fact that the bridge is officially functionally obsolete presents the possibility of widening the bridge in the future, however as any widening would require significant work and the bridge is still able to function as a two-way road it is unlikely that any major works of this nature will be carried out in the near future. Some work may need to be carried out on the deck and superstructure in the near future as the condition ratings suggest they are in a deteriorating, yet still acceptable condition.

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