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A CRITICAL ANALYSIS OF THE BLOUKRANS BRIDGE

P M Isaac[1]

[1]*University of Bath*

Abstract: This paper critically examines and reviews aspects of the Bloukrans Bridge in South Africa. Among other things the paper is focused on issues including the aesthetics, loading, construction, geotechnics strength and serviceability. Example calculations are shown where appropriate, including analysis for the strength of the deck and the arch, the creep in the deck, the effect temperature has on the structure and the fundamental frequency of the bridge.

Keywords: *Bloukrans Bridge, Arch, Reinforced Concrete, Prestressed Concrete, Suspended Cantilever.*

1. Introduction

The Bloukrans Bridge is a stunning single span arched bridge, located in the Southern Cape region of South Africa. The bridge is one of three commissioned on the so called 'Garden Route' to improve the transport link between Port Elizabeth and Cape Town. Of the three the Bloukrans is the largest. The other two are the Bobbejaansrivier and the Groot River bridges, both are also concrete arch bridges. Fig. 1 shows a photo of the completed bridge.

Figure 1: Photo of the Bloukrans Bridge taken from the east side looking south; Ref. [1]

The design of the bridge was by Lienberg & Stander Western Cape Ltd and the principle contractor was Murray and Roberts. The bridge was initially intended to be twice the width but a lack of finance meant that the design had to be scaled back.

Construction commenced on the Bloukrans in February 1980, using a suspended cantilever method of construction for the arch and the deck was incrementally launched on completion of the arch. Construction was completed in June 1983. On completion the bridge was the highest single span arch bridge in Africa and the fourth highest in the world. The key statistics of the Bloukrans are shown in Table 1; Ref. [2]:

Table 1: Key statistics of the Bloukrans Bridge

Deck length	450m
Deck width	16m
Number of traffic lanes	2
Rise of arch	62m
Clear span of arch	272m
Width of Arch	12m
Number of columns	23x2
Max height of column	65.5m

2. Aesthetics

2.1. Background

Aesthetical analysis of bridges is often carried out in accordance with the 10 rules outlined by Fritz Leonhardt in his book "Bridges"; Ref. [3]. Leonhardt believed that simple, elegant and harmonious bridges were aesthetically the strongest. The rules he developed act as guiding principles to designers all over the world. It is however, important to bear in mind that these are only guidelines. The 10 rules are as follows:

1. Fulfilment of function.
2. Proportions of the bridge.
3. Order within the structure.
4. Refinement of design.
5. Integration into the environment.
6. Surface texture.
7. Colour of components.
8. Character.
9. Complexity in variety.
10. Incorporation of nature.

During the design strong consideration was given to the aesthetics of the Bridge as it passes through indigenous forests in an area of outstanding natural beauty. The following section analyses the Bloukrans Bridge with reference to Leonhardt's 10 rules.

P. M. Isaac
pi202@bath.ac.uk

2.2. Aesthetics of the Bloukrans Bridge

Aesthetically, the first and most important feature of the bridge is that it looks and feels right, the dimensions, proportions and shape all look correct. The arch is a simple shape and makes good design sense in this location for a number of technical and aesthetical reasons.

In terms of fulfilment of function the Bloukrans cannot be faulted. The arch is a familiar shape and one used on bridges for hundreds of years. The regular spacing of columns and the size of both the arch and the deck give comfort to users of the bridge. The clear load path is a key feature and one which can be appreciated by the general public. Leonhardt laboured the point about harmony and simplicity in a bridge and it is easy to see with the Bloukrans that both have been achieved with some skill.

The proportions of the bridge have obviously been given careful consideration, from the thickness of the arch to the depth of the road and the width of the columns. The constantly spaced columns look and feel right. The thickness of the arch is greater than the other components showing clearly how the structure works, furthermore, the slender deck has the effect of enhancing the “aesthetical” strength of the arch.

According to Leonhardt the edges and lines of the structure should be smooth and continuous to give good order to the structure. The Bloukrans does this expertly with a smooth continuous fascia beam and by recessing the piers from the decks edge, although necessary as the arch is narrower than the deck. By having only two piers side by side the problem of opacity when viewed obliquely has been avoided.

Many of the features mentioned already show good refinement in the design, particularly the order and the proportions, both of which appear to have been given careful design consideration. The orientation of the columns shows good design refinement, having the slender dimension visible on elevation is superior aesthetically, due to the dissociation between the arch and the deck. This is shown in Fig. 2.

Figure 2: Photo showing the column, arch and deck arrangement; Ref. [1]

Some elements appear not to have been refined to such an extent. One example is that the columns have perfectly straight edges. Leonhardt pointed out that by having tall straight columns it gives the impression that they are actually wider at the top, which makes no sense. Also the deck appears to dip down in the centre which is relatively uncommon and slightly uncomfortable in bridges, however, this was dictated by the topography. The height of the parapets is also significant. By allowing relatively low parapets with gaps between, road users can take full advantage of the stunning view afforded by the elevated vantage point.

The bridge has been left in its natural concrete colour. This appears to work well with the surroundings as the quartzitic sandstone; Ref. [2], outcrops are of the same colour. Also as the bridge can be seen from below the use of the grey colour allows the bridge to stand out against a clear sky.

The use of the natural colour plays an important role in helping with the bridges integration into the environment. The use of an arched bridge seems most appropriate in the location and a concrete arch seems better suited to the surroundings than a steel trussed alternative. The hidden substructure works particularly well as the arch appears to come directly from the valley walls.

The texture of the bridge has been left to a natural finish. As the bridge is an under-bridge the texture is not significant.

The Character of the bridge is undeniable with the structures simple lines and striking boldness.

Leonhardt made an interesting observation regarding the complexity of a structure in that variety was good if viewed with simple neighbouring elements but too much variety increased the tension and over taxed the viewer. The Bloukrans succeeds in balancing simplicity with subtle complexity, this comes largely from the refinement in the design and includes factors such as limiting the breaks in lines, hiding the abutments and making the smaller dimension of the columns visible in elevation.

Finally Leonhardt looked at how a structure incorporates nature in its design. The Bloukrans appears to have been less successful in this aspect. The use of the natural colour, which is visually similar to the surrounding rock is one possible example of the incorporation of nature. A good feature of the bridge, although not coming directly from nature is the use of the arch which is a very familiar and historic shape.

2.3. Summary of Aesthetics

Overall it can be said that visually the Bloukrans is a great success and compares favourably with Leonhardts 10 rules for aesthetics. The use of the arch is fitting for the environment and the size of the arch gives the feeling of great strength. Generally it would be difficult to improve on the aesthetics of the design.

3. Geology

Over thousands of years the River Bloukrans has eroded a deep valley through an elevated wave cut platform. The wave cut platform was folded into its current location by the work of plate tectonics over time. The river itself runs from the Tsitsikamma mountains in the North to the Indian Ocean in the South.

The choice of an arched bridge with the load transferring directly to the valley sides, almost perpendicularly, makes geotechnical sense and is the obvious design choice for this location. Building out of the valley as in the case of the Clifton Suspension Bridge makes less sense technically due to the extensive work which is required to stabilise the valley walls to prevent slope failure.

Over many years the weak rock has been weathered and eroded from the valley sides leaving the strong resistant rock remaining, however this rock is likely to be heavily jointed so its complete structural integrity cannot be relied on. The rock layers in the platform have been folded to near vertical. This has caused a rib like pattern to form along the valley walls due to the different layers and rates of erosion.

The main material present in the valley is quartzitic sandstone Ref. [2]. Also present are elements of argillaceous sandstone, arenaceous shale and shale. The mineral quartz ranks highly on Mohs scale of hardness Ref. [4]. This gives a good indication that the valley walls will have adequate strength to support the loading from the arch.

4. Foundations and Geotechnics

Choosing a suitable site required detailed exploratory ground work to be carried out. From the report "The planning, design and analysis of the Bloukrans Bridge"; Ref. [2] it is stated that the initial survey indicated highly fractured but tight, unweathered rock on the east side. On the west side were a number of weak shale areas but these were well confined by massive unweathered quartzitic sandstone to the north and south.

The zone of the abutments required careful geotechnical consideration in order to minimise the settlement of the arch. A two dimensional analysis showed that a displacement of just 13mm at each of the arches springing points could be expected; Ref [2].

Approximately 12m deep of loose material was removed from both embankments as part of the site preparation work.

Two methods were used to stabilise the valley sides and prepare the ground for loading Ref. [5]. Firstly a grout was pumped into holes drilled into the face of the valley to hold the rock together. Secondly rock anchors were driven into the face to prevent failure of the slope.

5. Earthquakes

South Africa lies in a relatively un-active seismic area. Until the 29th September 1969 the country had experienced no major earthquakes and seismic loading was given relatively little consideration by designers.

However, on the 29th September 1969 a magnitude 6.3 earthquake occurred in the Tulbagh region; Ref. [6]. After the event a number of relatively strong aftershocks have occurred with no predictable locality- a characteristic common of intraplate earthquakes.

This event had serious implications on the design of the Bloukrans. The economic and social impact of the bridge collapsing due to an earthquake was immeasurable.

Preceding the design, a report was commissioned to determine likely ground accelerations during an earthquake and the probability of these events occurring during the bridges working life. The report suggested that a seismic event of intensity VI on the modified Mercalli scale; Ref. [4], had a 90% probability of not being exceeded in 200 years. This corresponded to ground accelerations of between 0.04g and 0.08g. The final design of the bridge was checked with ground accelerations of 0.1g, simulating somewhere between VI and VII on the Mercalli scale.

6. Construction

All elements of the bridge were constructed from reinforced or prestressed concrete. This presents a number of technical problems for designers. The biggest of the decisions is whether to use precast or in-situ concrete. This decision comes down to the locality of reinforcing steel, aggregate and cement, or ready mixed concrete.

The construction process was carried out in three main stages over a period of 36 months. These are:

1. Arch construction
2. Erection of columns
3. Deck construction

The issues surrounding each of these areas is covered below.

6.1. Arch Construction

The arch was the first of the three elements to be constructed. Although, this part covers two aspects: the construction of the substructure and the construction of the arch itself.

The thickness of the arch varies parabolically from a thickness of 3.6m at the centre to 5.6m at the springing points so as to make efficient use of the materials.

The first phase of construction was to carry out the ground works, this is covered in section 4. One additional span was required on the East side for geometric alignment due to the asymmetric nature of the gorge. Columns were erected on either embankment up to the location of the arch base. One set of columns either side were designed to act onto the arch springing points, this was to reduce the number of seperate foundations required. The first stage of construction is shown diagrammatically in Fig. 3 below:

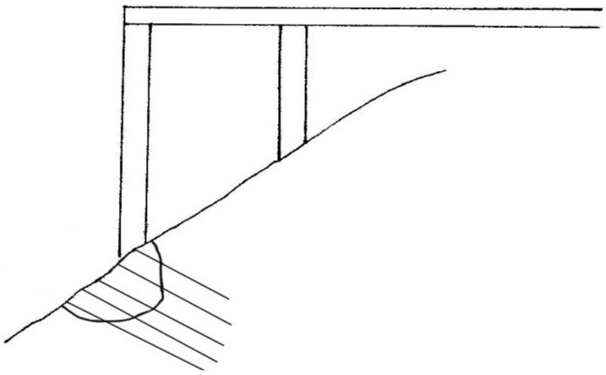

Figure 3: Diagram of the first construction phase

At this stage the construction of the arch began from either side. The suspended cantilever method was adopted and the concrete poured in situ with travelling formwork. This is shown in Fig. 4. Also it can be seen that a cableway has been erected across the valley, this will act as a crane to carry materials out to the construction, this is an important feature which would speed up the construction.

Figure 4: Photo showing the construction of the arch using the suspended cantilever method; Ref. [7]

Using a suspended cantilever method was most suitable in this location. Alternatively steel centring could have been erected across the entire valley. This method was used extensively by the Swiss engineer, Maillart who designed many arch bridges in Switzerland. In modern days this system is uneconomic due to the long construction time required and the extra material needed.

The main advantages of the suspended cantilever method are the long distances spanned and the ability to accurately adjust the height of the arch with the suspension cables, in the case of the Bloukrans as construction was nearing completion one side of the arch needed to be raised by over 600mm. The main drawback of this method is the cost associated with erecting the temporary piers for the suspension cables. Also the downward force caused by the suspension cable piers would create high vertical loads acting at the arch foundations and through the columns at this location. This is shown in Fig. 5.

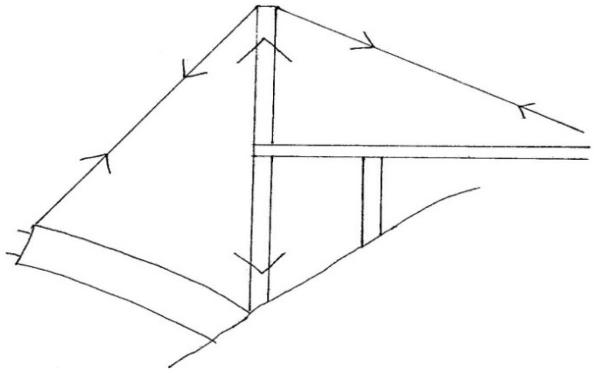

Figure 5: Diagram showing forces during construction

Simple statics can be used to show the force in the suspension cables and the force in the column acting on the abutment. By treating the situation as a propped cantilever with the suspension cables acting as a single cable the forces in the cable can be attained. The worst loading on the section will occur just prior to the joining of the two segments when the length of the cantilever will be 135m. For the purpose of this simple calculation it is assumed that the cables meet the column at an angle of 70°, this would give a value for the tension in the cable of 87155.6 kN at the Ultimate Limit State (ULS). If it is then assumed that the tie back cable meets the column at the same angle as the suspension cable the force acting through the columns can be shown to be 59618 kN. This high force would be a significant design consideration for both the column and the foundations at the arch abutment. It is worth mentioning that as can be clearly seen from Fig. 4 that many suspension cables have been used. A single cable would yield under such large forces and the extension would increase disproportionally to the applied load, leading to significant deflections and possible collapse of the cantilevered section.

A problem existed during the construction of the two segments with the high wind load. During the construction the segments were effectively pinned in the horizontal plane. This meant that the structure had no way of resisting horizontal wind loading. One solution to overcome this would be to tie the front of the cantilevered arch back to the valley sides, this can just be seen on the right segment in Fig. 4.

The joining of the two arch segments was a complicated and delicate procedure but on completion the arch had gained its structural integrity and the next stage of construction could commence.

6.2. Erection of Columns

With the arch in place and the target strength of the concrete achieved, work could begin on the erection of the columns. The construction was carried out using a traditional approach to reinforced concrete construction. The rebar was installed, form-work erected and the concrete then poured.

The columns were spaced at 19m centres with a 15m span at each end. Three sizes of columns were used throughout the bridge, with the most common being 2.5m by 1m with the slender dimension visible on elevation this works well from an aesthetical point of view but also makes the column more resistant to the bending moments induced by the longitudinal expansion of the deck due to temperature increases. Two columns were erected side by side to support the decks two twin box section. The relatively close spacing of the columns helps to make the arch loading more uniform and also allows for a more slender deck, this also works best with the deck construction method. The maximum height of any column was 65.5m.

6.3. Deck construction

The deck is constructed from two twin box sections side by side, each supported by one of the columns. This eliminates the need for a cross beam, saving the material requirements and reducing construction time. The structural arrangements and dimensions are shown in section in Fig. 6.

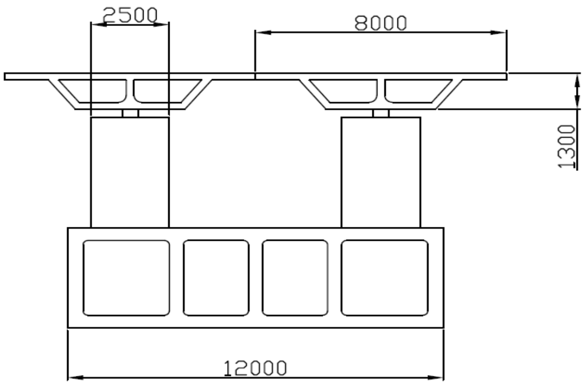

Figure 6: Transverse section showing two twin box sections

The twin box section has been assumed based on construction trends at the time the Bloukrans was built.

An incrementally launched system was used to construct the deck. The constant depth of the deck is a clear indication that this construction method was used. This system requires factory type conditions to be set up on one embankment. The sections are then cast in this environment on site and simply pushed into place. The process has a cycle time of between 7 and 10 days and typically sections of 15m-30m are constructed at one time; Ref. [8]. Using this system ensures a high quality deck construction. Environmentally it is a better method than using precast elements as the transport costs for precast elements is much higher than those for wet concrete. Other key advantages of this are that a large amount of concrete is required for the arch so economies of scale can be created with site mixed concrete in a controlled environment and a high quality product can be assured. As the span of the bridge is relatively long forming the deck with in-situ cast concrete could lead to problems with concrete disagregation.

The max span/depth ratio for such a construction method is approximately 15 therefore, the depth of the deck would therefore be in the region of 1.3m.

The main issue that needs to be overcome with this construction method is ensuring the transverse stability of the deck during construction and ensuring the stability of the columns as the deck is pushed into place. The transverse action of the wind on the longitudinal face would be particularly problematic in this location. Stability of the columns can be achieved by tying the columns to one another in a mesh of temporary cables. One major drawback of the method is that as the deck is pushed out into place each section will at some point experience maximum hogging and sagging moments. Each section must therefore be designed for this and would usually be constructed with both top and bottom flanges of equal thickness. However in its working state a high percentage of the concrete in the section is doing very little work, particularly concrete on the tension side.

The bearings used in this type of construction are essential to ensure the launching of the deck takes place effectively. The bearings are designed to allow for longitudinal sliding during the launching procedure this is done by making the top surface of the bearing from stainless steel, during the launching sheets of very low friction PTFE are fed between the bearing and the deck. Two types of bearing exist for this operation, both operating in a similar manner. Temporary sliding bearings can be used during the construction phase, on completion the deck is lifted and the bearings removed and replaced with permanent bearings. The other type combines the two features, on completion the deck is once again lifted but this time only the steel top plate and PTFE pads are removed, the gap between the deck and the bearing is then filled with grout.

Two methods for launching the deck currently exist. One method jacks the deck up and pushes the deck. The other method patented by prestressed bridge experts VSL pulls the deck.

7. Loading

The loading of the Bloukrans can be broken down into two parts; the load acting on the deck and the load acting on the arch. In full design both would need to be assessed with different load cases to determine the worst combination acting on the structure. For the arch the worst case loading is likely to come from a uniform load over half the arch, capacity checks for this are carried out in a later section. Worst case loading and stresses will mostly likely occur in the deck during the launching procedure; however these checks will not be attempted in this paper.

All loads used in analysis for the purpose of this paper are in accordance with BS 5400-2:2006. For bridge design loads are multiplied by 2 partial safety factors, γ_{f3} and γ_{fl} at both the ULS and the Serviceability Limit State (SLS).

An important aspect of BS 5400-2 is the concept of notional lanes, these differ from marked lanes and for analysis vehicular loading is applied to the notional lanes, not the marked lanes. The Bloukrans Bridge has a carriageway width of 15.05m, which equates to 4 notional lanes.

BS 5400-2 outlines 5 main load combinations which should initially be checked in bridge analysis, these are as follows:

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.

7.1. Dead Load

The Dead load of the bridge is taken as the weight of all the structural members. For the purpose of this paper it was assumed that prestressed concrete has the same weight as reinforced concrete (2400 kg/m^3). The dead weight of each component is shown in Table 2.

Table 2: Dead weight of each element of the bridge

Component	Dead Weight
Deck	163.6 kN/m length
Arch	410 kN/m length
Columns	58.75 kN/m height

7.2. Super Imposed Dead Loading

A separate classification applies to dead load acting on the structure which itself is not structural. This includes the road surface, the fill material for the road surface and any services that my run through the bridge deck. The amount of super imposed dead load is relatively difficult to predict. Because of this BS 5400-2 applies high load factors (γ_{fl}) of 1.20 at SLS and 1.75 at ULS. This takes into account the degree to which the load increases during the roads life, largely due to new layers of black top and increasing amounts of services in the deck. A value of 9.25 kN/m^2 has been calculated for example calculations in this paper. This comprises of approximately 300 mm of saturated fill material (density 2000 kg/m^3), 100 mm of black top (density 2400 kg/m^3) and 1 kN/m^2 additional loading for services and road furniture.

7.3. HA Loading

The Highways Agency's HA loading is used as the basic vehicular live load applied to a bridge. For bridges greater in length than 380m the HA live load is taken as 9kN/m. Therefore this is the value that will be used in analysis for the Bloukrans (length 450m). The value of 9kN/m takes into account the low probability of having the full span loaded with heavy lorries simultaneously. As part of the HA loading specification an extra KEL knife-edge load of 120kN is applied where it would have the most adverse effect for the appropriate assessment.

7.4. HB Loading

HB loading simulates the action of an abnormally heavy truck. In HB loading, a basic layout for trucks is taken. For full HB loading 45 units of load is applied. This is equal to 450 kN per axle or 1800 kN for the entire truck. The length of the truck can then be varied to produce the most adverse effect on the structure. This is shown in Fig. 7.

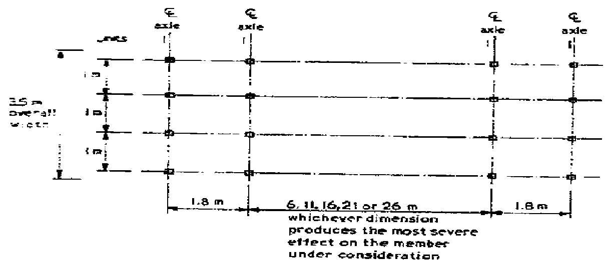

Figure 7: Diagram of HB loading

HA and HB loading are applied simultaneously, BS 5400-2 outlines a number of ways to apply the two loads together, one is illustrated in Fig. 8. Full HA loading is applied to two notional lanes and ⅓HA is applied to the remaining lanes.

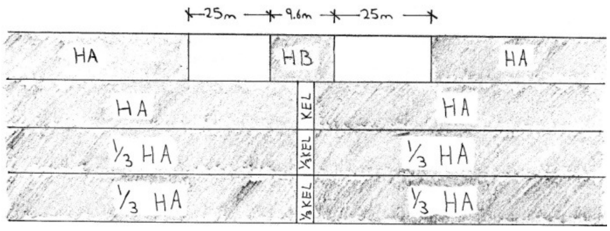

Figure 8: Diagram showing HA and HB loading on deck

7.5. Wind Loading

A v-shaped valley has the effect of funnelling the wind, increasing the wind load acting on the bridge. In the case of the Bloukrans, the close proximity of the bridge to the sea (2km south) will also play a role in increasing the wind design load. The wind loading on the structure is particularly significant during the construction of the deck and the arch. The design of the low friction bearings is aimed at reducing friction in the longitudinal direction allowing for the deck to be pushed into place. This has the associated problem of reducing friction in the transverse direction creating problems in gusty conditions. As mentioned above, the wind acting on the cantilevered sections of the arch during construction would be particularly problematic as the structure had no way of resisting such actions itself.

The wind loads are calculated in accordance with BS 5400, the first stage is to calculate the maximum wind gust (v_c) using Eq. (1). The wind loads on the deck, columns and arch will be calculated separately. For the purpose of this paper the effect of the transverse wind on vehicles will not be considered.

$$v_c = vK_1S_1S_2. \quad (1)$$

v is the local wind speed in m/s. Preliminary site research showed the wind speed was 25.6 m/s; Ref. [2], based on a 120 year return period. K_1 is a wind coefficient, 1.86 in this case, S_1 is a funnelling factor for the topography, 1.1 in this case and S_2 is a gust factor, taken as 1.66.

$$\therefore v_c = 86.9\ m/s$$

For analysis this value is converted to a dynamic pressure head, q, using Eq. (2).

$$q = 0.613 v_c^2. \qquad (2)$$

This gives an overall dynamic pressure of 4.63 kN/m^2. A number of factors are applied to the dynamic pressure to give the characteristic wind load. These factors depend on the characteristics of the structure. The final loads are shown in Table 3.

Table 3: Characteristic Horizontal Wind Loads

Member	Wind load
Deck	5.1 kN/m^2
Columns	6 kN/m^2
Arch	6.49 kN/m^2

The wind also has the effect of inducing both downward pressure and upward, relieving pressure on the structure. The values of these loads are shown in Table 4.

Table 4: Upward and Downward Characteristic Wind loads

Member	Wind Pressure
Deck	3.47 kN/m^2
Arch	3.47 kN/m^2

8. Strength

8.1. Deck at Serviceability

A simple check for the purpose of this paper was carried out on the adequacy of the prestress tendons to resist bending in the deck. In the absence of exact details a number of assumptions were required to carry out the check. For simplification, only one of the decks two box sections has been checked as both are identical.

The section was checked for load case 1 (section 6) at the SLS. In full analysis the ULS would also need to be checked but the calculations for this are beyond the scope of this paper as the effect of the reinforcing steel and prestressing steel both need to be taken into account. However, the calculations outlined in this section serve as an example of the process required. The following assumptions were made:

- Position of neutral axis is 0.45d
- Area of prestress steel is 0.5% of effective area = 22750 mm^2
- Total section area is 3475000 mm^2
- Prestress steel located at centre of flange
- 60% of maximum theoretical prestress level is achieved
- Yield stress of prestressing tendons is 1700 N/mm^2
- I value of section 8.14x10^{11} mm^4

The capacity of the section is checked at the supports where bending moments will be highest. The bending moment at this location is calculated from Eq. (3)

$$M = \frac{wl^2}{12}. \qquad (3)$$

The force in the prestress tendons can be calculated from Eq. (4).

$$F = 0.6 f_y A_{ps}. \qquad (4)$$

Values for f_y and A_{ps} are given above.

$$\therefore F = 23 \times 10^6 N$$

This in then calculated as a stress from Eq. (5)

$$\sigma_c = F/A. \qquad (5)$$

$$\sigma_c = \frac{23 \times 10^6}{3.475 \times 10^6} = 6.6\ N/mm^2$$

The prestress force also has the effect of causing an extra component of compression at the top of the section over the support and tension at the bottom. These stress levels are calculated from Eq. (6).

$$\sigma_{c,t} = \frac{(p.e)y}{I}. \qquad (6)$$

Where p is the prestress force; e is the eccentricity of the prestress force; y is the distance from the N.A. to the position where the stress level is required and I is the second moment of area. It can be shown from Eq. (6) that σ_c = 14.1 N/mm^2 and σ_t = 9.65 N/mm^2. These forces are combined with those calculated from Eq. (5) to give the final prestress state. These are represented by stress blocks in Fig. 9.

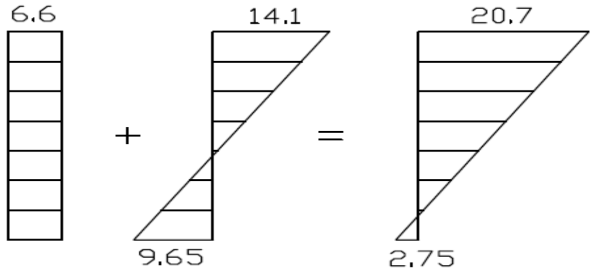

Figure 9: Stress blocks for prestress state of deck

The loads from the SLS, load combination 1 can be shown to give a moment of 6466 kNm at the supports. The resultant stress generated by this moment can be calculated from Eq. (7).

$$\sigma_{c,t} = \frac{My}{I}. \qquad (7)$$

This gives σ_c = 4.2 N/mm^2 and σ_t = 6.13 N/mm^2. Combining these with the prestress state gives the overall stress state in the deck. This is shown in Fig. 10.

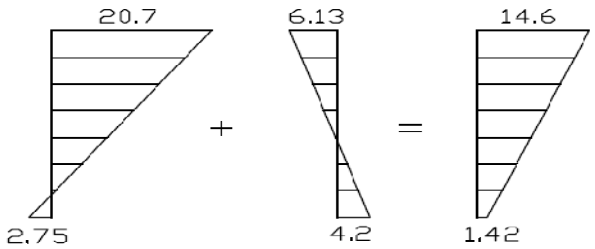

Figure 10: Final stress state for Deck at SLS

Fig. 10 shows that at the SLS the deck experiences no tension and a maximum compression force of 14.57 N/mm^2. The maximum allowable compressive stress is equal to 0.33f_{cu}. Assuming a grade C45 concrete has been used, the maximum allowable stress is equal to 14.85 N/mm^2 proving that the prestressing in the section is adequate at the SLS for this particular load combination.

8.2. Arch at ultimate limit state

Similar checks to those shown in section 8.1 can be carried out to find the capacity of the reinforced concrete arch. During normal uniform loading the parabolic shape of the arch means that it is always loaded axially and experiences no bending. However the application of non-uniform loading will cause bending in the section. For the purpose of this simple check the arch has been treated as a three pin arch.

The calculations outlined in this section serve as an example of typical design checks. The calculations are carried out for load combination 1 (section 6) at the ULS. For the purpose of this design check fully factored live (HA and full HB), dead and super imposed dead loads were applied as shown in Figure 8 to one half of the arch, and un-factored dead and superimposed dead loads were applied to the other half. It can be shown that this out of balance of loads will cause a maximum bending moment at the quarter span. For the loads mentioned above this moment was calculated to be 1.17x10^6 kNm. The sections bending capacity can then be calculated and compared with this value.

For analysis an area of steel reinforcement had to be assumed in the absence of accurate information. A value of 6% of the total effective area was assumed. The steel reinforcement was also assumed to be located at the mid depth of the flange.

Fig. 11 Shows the typical strain diagram and forces arrangement for a reinforced compression member. In pure bending the compressive forces will be equal to the tensile forces. This would represent one point on the M-N interaction diagram for the section.

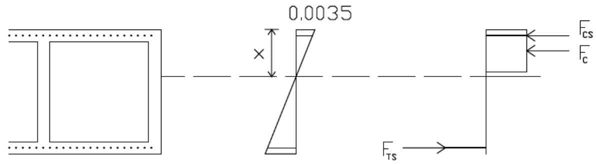

Figure 11: Stress block diagram for reinforced section

Due to the fact that there is no axial force Eq. (8) can be deduced.

$$F_{ST} = F_{SC} + F_C. \quad (8)$$

Where F_{ST} is the force in the tension steel, calculated from Eq. (9). F_{SC} is the force in the compressive steel. The compressive force depends on the strain in the steel. If the strain is less than the yield strain (0.002) F_{SC} is calculated from Eq. (10), otherwise it is calculated from Eq. (11). The value of the strain is dependent on the depth (x) of the neutral axis. Finally F_C is the compressive strength of the concrete, calculated from Eq. (12).

$$F_{ST} = {A_{ST} f_y}/{1.15}. \quad (9)$$

$$F_{SC} = \varepsilon_{SC} E_S A_{SC}. \quad (10)$$

$$F_{SC} = {A_{SC} f_y}/{1.15}. \quad (11)$$

$$F_C = 0.45\, f_c A_c. \quad (12)$$

A_{ST} and A_{SC} are the areas of steel reinforcement, taken as 523800 mm^2; f_y is the yield strength of the steel, 460 N/mm^2; E_s is the Youngs modulus of steel, 200000 N/mm^2; f_c is the characteristic compressive strength of the concrete, 45 N/mm^2 and A_c is the gross area of concrete in compression.

Altering the depth of the Neutral Axis (NA) allows a solution to Eq. (8) to be found iteratively. It was found that the NA was 354mm below the top of the section. This gave the following set of results:

$$F_{ST} = 209.5\ MN$$
$$F_{SC} = 134\ MN$$
$$F_C = 75.5\ MN$$

The bending capacity of the section can be found by taking moments about the bottom steel. This gives a moment capacity for the section of 1.18x10^6 kNm, proving that the section is adequate to withstand bending with this particular load case.

9. Temperature effects

Fluctuations in the temperature of the bridge is an important aspect in the design of the bridge. The expansion of the material leads to associated movements in the deck or an increase in the residual stress if the movement of the bridge were to be restricted. The behaviour of the bridge deck with temperature involves a complicated analysis due to the non-linear effect of temperature increase across the deck. For the purpose of this paper a simplified analysis has been carried out assuming the temperature rise is constant across the bridge deck. The strain (ε_T) caused by temperature increase is shown in Eq. (13)

$$\varepsilon_T = \alpha\Delta T. \quad (13)$$

Where α is the coefficient of thermal expansion for concrete, $12 \times 10^{-6}/°\mathrm{C}$ and ΔT is the increase in temperature in the deck, taken as 25 °C

$$\therefore \varepsilon_T = 300\mu\varepsilon$$

The expansion of the deck (ΔL) can be calculated from Eq. (14).

$$\Delta L = \varepsilon_T L. \quad (14)$$

Where L is the full length of the bridge deck, $450 \times 10^3\ mm$

$$\therefore \Delta L = 135\ mm$$

If no expansion joint existed or it were to become blocked the increase in temperature would cause an increase in the residual stress (σ_c) within the deck. The increase in compressive stress can be calculated from Eq. (15)

$$\sigma_c = \varepsilon_T E. \quad (15)$$

Where E is the Youngs modulus of concrete, $30000\ N/mm^2$

$$\therefore \sigma_c = 9\ N/mm^2$$

This value of stress is very high and combined with the stress from the loading and prestressing would cause significant problems in the bridge. This shows that having at least one expansion joint is essential and maintenance must be carried out to ensure its continual operation. The design of the bearings requires careful consideration in this instance. If the decks movement is restrained by the bearings a large moment will be produced in the columns.

10. Serviceability

10.1. Initial Deflections

The prestressing force in the tendons will control the deflection of the deck when load is applied. However, as the arch is constructed from reinforced concrete its deflection is not controlled. Calculations involving virtual work can be carried out to show the level of deflection expected in the arch but these calculations are beyond the scope of this paper so will not be attempted. A limited amount of deflection will be experienced by the deck due to creep of the concrete. This is discussed below.

10.2. Creep

All concrete bridges will undergo some element of creep during their working life. This is a function of the setting of the concrete, which continues for a long period of time and due to the constant loading applied to the bridge in the form of the dead and super imposed dead loads. Although creep will continue throughout the life of the structure the great majority will occur in the first year. To determine the amount of creep (δ) of the deck, Eq. (16) is used.

$$\delta = \frac{w(0.2l)^4}{8EI} + \frac{(0.3wl)(0.2l)^3}{3EI} + \frac{5w(0.6l)^4}{384EI}. \quad (16)$$

Where w is the load applied, 341 kN/m; l is the length of a span, 19000 mm; I is the second moment of area, 1.63 mm^4 and E is the long term Youngs modulus of concrete, 10000 N/mm^2.

The final deflection of the section due to creep is 7.34mm. Over a span of 19m this is a very small.

10.3. Fundamental Frequency

The fundamental frequency of a bridge can be a particular problem if the value lies outside the acceptable range of 5-75Hz.

Complicated analysis is required to find an exact value of the frequency, but this lies outside the scope of this paper. Other simplified methods exist, which give an approximate value and one such method is the Rayleigh-Ritz method. The fundamental frequency (ω_n) using this method can be calculated from Eq. (17).

$$\omega_n = (\beta_n l)^2 \sqrt{EI/ml^4} \quad (17)$$

Where: $(\beta_n l)^2 = 22.37$ for a clamped-clamped situation; E is the Youngs modulus of Concrete, $30\mathrm{x}10^9$; I is the second moment of area, 1.63 m^4; m is the mass density of the section, $16.7\mathrm{x}10^3$ kg/m and l is the length between supports, 19m.

$$\therefore \omega_n = 106\ Hz$$

This is not the true value for the section as the real case is a pinned-pinned situation. However, the Rayleigh-Ritz method has no value for $(\beta_n l)^2$ for the pinned-pinned case, therefore, some extrapolation is required.

It can be shown that for a clamped-pinned situation, $\omega_n = 73\ Hz$. Extrapolating from this value would yield a fundamental frequency in the region of 40 Hz for the pinned-pinned situation. This falls well within the acceptable limits, showing that the Bloukrans should experience no problems with vibrations.

11. Longevity

A number of issues may arise during the lifetime of the bridge that may affect its longevity. Many of these are designed against but some cannot be avoided. This section looks at the main issues affecting the life of the bridge.

11.1. Loss of Prestress

A well known problem with prestressing is that the tendons lose some element of the prestress force during loading. The loss of force in the prestress tendons can be caused by a number of factors. These include losses due to the friction with the ducts, the elastic shortening of the concrete during prestressing, stress relaxation of the steel tendons and creep losses due to shortening of the concrete; Ref. [8]

11.2. Accidental/Intentional Damage

The susceptibility of the bridge to accidental and intentional damage is relatively low due to the size of the structure. As the bridge is an under bridge there is no chance of a traffic accident leading to bridge collapse. Intentional damage in the form of a terrorist attack would be of concern to the designers, however low the perceived risk is. The composite construction and sheer size of the components makes the structure strong against impact loading meaning that the columns, deck and arch should be able to withstand a moderate blast load.

11.3. Chemical Attack on Concrete

Chemical attack on concrete road bridges in the UK is a significant problem, largely due chloride attack from the de-icing salts applied during the winter. However, this is unlikely to be a problem in South Africa due to the temperate climate. There are many other causes of concrete damage which can affect the structure during its working life; Ref. [9], these include:

- Moisture penetrating capillary voids. Often excess water is added to the concrete to improve its workability, when this water dries out small voids are left through which moisture can penetrate.
- The concrete has a very high alkaline content and this acts to protect the steel from corrosion. However, carbon dioxide in the air reacts with the lime in the concrete, neutralising it. If the reaction penetrates to the level of the steel, corrosion of the steel will occur.
- Chemical corrosion which can come from a variety of sources.

Most of these problems occur over long periods of time. By providing suitable levels of cover for the prestressing and reinforcing steel many of these problems will be avoided, the prestressing itself helps due to the compression force it exerts throughout the deck.

11.4. Climate Change

One of the issues with climate change is that the earth is expected to experience a greater number of higher intensity storms in the future. This would significantly increase the wind loads acting on the Bridge.

12. Future Changes

The original tender documents required the design of a bridge twice as wide, it is therefore not un-imaginable that the bridge may exceed its capacity sometime in the future. If this were to happen creating extra traffic lanes may cause significant problems. One option would be to build a second bridge alongside the first. Another possibility might be widening the bridge. However, this would be incredibly expensive, time consuming and disruptive. An example of how the bridge could be widened is shown in Fig. 12.

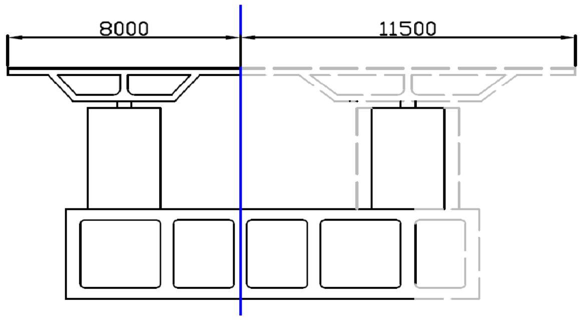

Figure 12: Section showing existing structure (left side) and proposed widened structure (right side)

The right side of Fig. 12 shows how the proposed widening of the deck may work. Construction of this proposal would cause severe disruption to the route but it may be possible to keep the bridge open to traffic whilst the work is carried out.

During the life of the bridge the road surface will need renewing on many occasions, this is taken into account with the high load factors applied to the super imposed dead load. It may also become necessary in the future to reinforce the bridge if it was found that the load on the structure had increased significantly above the design loads or the strength of the bridge had deteriorated. This could be done with a Fibre Reinforce Plastics (FRP) which is applied in the form of thin sheets. This method of bridge reinforcement has been used extensively in the USA.

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What is topological analysis ? A method of obtaining chemically significant information from the electron density Ï (rho) Ï is a quantum-mechanical observable, and may also be obtained from experiment. â€œIt seems to me that experimental study of the scattered radiation, in particular from light atoms, should get more attention, since in this way it should be possible to determine the arrangement of the electrons in the atoms.â€Â The traditional way of approaching the theoretical basis of chemistry is though the wavefunction and the molecular orbitals obtained through (approximate) solutions to the SchrÃ¶dinger wave equation. HÎ¨ = EÎ¨. The Hohenberg-Kohn theorem confirmed "Bloukrans Bridge - Le pond de Bloukrans". Arch 01. troisieme conference internationale sur les ponts en arc. p. 35. ^ http://www.murrob.com/projects_detail.asp?project=16. ^ Isaac, P.M. (16 April 2008). "A critical analysis of the bloukrans bridge". Proceedings of Bridge Engineering 2 Conference 2008. ^ My Destination Website, retrieved 26 April 2012. External links. Media related to at Wikimedia Commons. Bloukrans Bridge at Structurae. Face Adrenalin, the bungee operators at the bridge. Categories.