CRITICAL ANALYSIS OF THE RIO-NITERO BRIDGE IN RIO DE JANEIRO, BRAZIL

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Abstract: This paper provides a critical analysis of the Rio-Niteroi Bridge, connecting Rio De Janeiro and Niteroi across the Guanabara Bay. The paper begins by outlining the history of the bridge before going on to analyse the bridge on its success both aesthetically and functionally. Much of the data in the paper has come from researched sources; however, some of the claims made in this paper are based on educated assumptions. There are sections of this paper that outline the bridges construction, analyse its loading, strength, serviceability and foundations. The paper also describes how the bridge deals with effects such as temperature, creep and wind. Finally the paper finishes by outlining the future of the bridge and hypothetical improvements and changes that could be made.

Keywords: Baia de Guanabara, Brazil, Highways Bridge, Reinforced Concrete, Rio-Niteroi Bridge

1 Introduction

The Rio-Niteroi Bridge as it is commonly known is actually officially called the ‘President Costa e Silva Bridge.’ It was named after the president who ordered its construction.

The bridge spans the Guanabara Bay in Brazil and connects the cities of Rio De Janeiro and Niteroi, thus its commonly used nickname. Its total length is an impressive 13.3km. It was the longest bridge of its kind when it was completed in 1969[1]. The bridge is part of the BR-101 highway. It provides a vital connection between the capital city and the second biggest city in the state, Niteroi. The journey for commuters would otherwise be a drive in excess of 96km around the Baia de Guanabara which the bridge crosses.[2]

The Bridge also boosts the highest central span of its kind. At 72m high, the steel central span allows hundreds of ships to pass into the bay each month. The bridge has eighteen access points and eight overpasses.[3]

The bridge is a three-lane highway in both directions making the deck 26.6m wide. Management of the bridge is the responsibility of Ponte S.A. who has the lease for its maintenance or twenty years since June 1995 [3][4].

The bridge is a reinforced concrete structure. On their first and so far only state trip to Brazil, Queen Elizabeth II and Prince Philip officially commenced the construction on August 23, 1968. Work only began in January 1969 and was not completed until March 1974[3][4].

2 Aesthetics

In the following sub-sections the bridge has been analysed on its aesthetics based on Fritz Leonhardts ten rules for a beautiful bridge laid out in his book ‘Brücken.’ Obviously there are some of his rules that
either don’t apply or are inappropriate for criticising this bridge. Rules such as those on colour and surface texture were obviously not overly considered in the design of this bridge, but the order and refinement of the bridges design and its incorporation in its surroundings are key to the bridges aesthetics. The following two sections split up Leonhardt's rules into two, one concerning the bridge itself and the other concerning its surroundings.

2.1 Physical Appearance of The Bridge

The bridge is not a complicated one to understand on first appearances. It is quite clear that the large piers carry the load to the ground. The presence of two piers across the breadth of the structure give a sound impression of stability. Although it sounds trivial, part of the bridges success is this impression of stability that is given by the substantial structure that makes you feel comfortable were you to cross it.

The bridge is well proportioned. The size of the piers compared to the size of the deck seems to be in a ratio that appeals to look at. The majority of the piers are quite short. Where the main span is higher to create access to the bay the ratio of span to bridge height is kept the same. This may have been intentional to keep the proportions the same, but it may have just been necessary to make it that width for the ships access. However, in the run up to the main spans the width doesn’t seem to gradually increase, except for the two spans either side of the central span as seen in figure 1. It may have been that the low gradient up to the central span was too small to warrant changing each span up the gradient slightly longer than the last. Cost may have been a governing factor.

Figure 2: A view of one of the central spans

This lack in gradually widening spans up to the central span and the change in material and texture causes a break in the line when you look across the bridge, figure 2. This is a shame, because it lets down the other parts of the bridge where the road level is constant where the bridges order is a great success to its aesthetics.

The bridge could be described as quite standard in its design. There doesn’t appear to be much refinement. It could have been possible to just add something as subtle as tapering the columns. This is usually done in bridge piers to remove the illogical image created when looking up at a pier and thinking it is wider at the top than the base. One could argue that this was not necessary with the Rio-Niteroi Bridge as there are few positions from land where you could get this impression, as you mainly look at the bridge from the distance from the shores of the bay.

Figure 3: The colours and shadows are key in analysing the aesthetics

It has already been mentioned that the colour and texture don’t play a major role in the bridges aesthetics. It is worth noting however, that as you cross the bridge when you have the view shown in figure 3, the colour of the fittings is in stark contrast to the bridge and looks quite ugly. If they had been a duller colour it would have blended into the bridge much better. This is not a problem from a distance as you cannot make out the details and so you only see the bridge.

Another interesting thing we can take from figure 3 is effect of having wide piers in section. It provides deep shadows as one looks along the bridge, which cause the deck to look a lot deeper than it is. The other problem with having wide piers is that an oblique barrier is created. Even though there are still only the standard two piers across the breadth, the angle that you have to be at to see through the piers is relatively large.

2.2 The Surroundings

It is always quite difficult for a bridge spanning a large expanse of water to integrate into its surroundings, especially when there are so many views of the bridge where you see it crossing the bay with the mountainous landscape in the backdrop. However, the Rio-Niteroi Bridge seems to work against the landscape. In its crudest form, one could make the comparison that the rise and fall of the central span mirrors the mountainous landscape and shows (in figure 1) that although you would expect a bridge of such a cumbersome structure to stick out like a sore thumb, this bridge attempts, and succeeds to some extent, in feeling quite comfortable in its setting.

The Rio-Niteroi Bridge was built in a time and culture where cost was probably a governing factor in its design. The fact that it was the longest bridge in the
world when it was constructed and it had the highest central span of its type was a big enough achievement in itself. It would be harsh to criticise the bridge for not showing some complexity in its design, although, the absence of this one factor certainly doesn’t give the impression that the bridge is lacking a certain aesthetic aspect.

Despite this lack of complexity it is definitely not without character. In summary, despite a few minor flaws, it would be fair to say that this bridge has some success in its aesthetics. It is a lovely bridge to look at and it is easy to admire it for the feats it achieved when you consider the age in which it was built.

3 Choice of Design

The bridge was built as a vital link between the capital and the second largest city in the state to create easier and possibly greater commercial and economic trade between the cities.

One could be quick to jump to the conclusion that economic considerations were the key factor in deciding the bridges design. However, one of the major restrictions on the bridges design was in fact its location, or more specifically, its surroundings. It was prescribed to the designers that towers were completely out of the question as the bridge lay in the flight path and approach to the Rio De Janeiro airport.

This limited the designers significantly and so from the offset a short span pier bridge was pretty much the only option, which meant making sure that other conditions (such as geological) would also permit this design. The designers then had to do all they could to minimise the number of piers.

The problem with having lots of piers is that the ground conditions have to be suitable along the entire length of the bridge, opposed to a few carefully selected positions for towers in a suspension bridge for example. Foundation conditions are discussed later in the paper.

The next determining factor would have been economic conditions. Being that steel would have been a cheaper resource in south America, one can assume that the labour costs of a concrete construction would result in a cheaper bridge than an equivalent made from steel.

The access points for trade shipping routes, the entry points to the bridge and the use of the Ilha da Conceição as an intermediate point governed the bridges geometry in plan and elevation. The bridge could not be straight. It is safe to assume that a bridge of constant radius would have been preferable, but as it not a constant curve in plan, it was probably the location of the shipping routes and founding conditions that prevented it from being so.

Loading influences would have governed the sizing and detailing of the bridge and is discussed later in the paper.

The bridges live loading was pretty prescribed by the need to have six lanes of traffic, three in each direction.

4 Construction

4.1 Key persons, dates and costs

Construction on the bridge officially started in August 1968 but works didn’t get underway until January the following year. The bridge took over five years to complete. It was completed in March 1974.

The president at the time, President Costa e Silva, was responsible for starting the project. The following table shows key figures in the bridges construction:

<table>
<thead>
<tr>
<th>Client</th>
<th>Ecex</th>
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</thead>
<tbody>
<tr>
<td>Designers</td>
<td>Antonio.A, Noronha Serviços de Engenharia S.A</td>
</tr>
<tr>
<td></td>
<td>Howard, Nettles, Tammon &amp; Bergendoff</td>
</tr>
<tr>
<td>Contrator</td>
<td>Construções e Comerico Camargo Correa S.A.</td>
</tr>
<tr>
<td></td>
<td>Constructura Rabello S.A.</td>
</tr>
<tr>
<td></td>
<td>Mendes Junior S.A.</td>
</tr>
<tr>
<td>Steel Construction</td>
<td>Cleveland Bridge &amp; Engineering Co. Ltd.</td>
</tr>
<tr>
<td></td>
<td>Dorman, Long &amp; Co. Ltd.</td>
</tr>
<tr>
<td></td>
<td>Montreal Engenharia</td>
</tr>
</tbody>
</table>

Table 1: Notable persons/companies involved in the Bridges construction [4]

The total build cost of the Rio-Niteroi Bridge came to over $22 million. It was funded by British banks. The bridge was tolled in one direction to re-coup the build costs. Currently it would cost you $1.80 (Aug ’07) [3] to cross the bridge into Niteroi.

Figure 5: The toll station heading into Niteroi following the bridges passing over Ilha da Conceição.

4.2 Materials

It is possible that the choice of material was influenced by the presence of British designers who would have been used to building in concrete. It is a plausible claim when you note that a widely available natural resource of South America is iron ore, thus making steel construction possibly cheaper, as it is in North America. It may have been that at the time the unskilled nature of the work needed meant labour costs of a concrete structure were cheaper, therefore making it a more viable construction material. Also concrete
has its obvious advantages when using it in a marine environment.

4.3 Details and methods of construction

The Bridge has a total length of 13,260m. The central spans are 200m either side of a 300m central span. These central spans are made from a steel hollow box girder supported on a cantilever system on reinforced concrete piers.

The bridge was constructed using segmental methods of construction. In Figure 4 you can clearly see the sections of the bridge being floated out into the bay where they are lifted into place on site.

You can also see on the right hand side that the central span is a cantilever system. Methods of construction such as launching girders or incremental launching weren’t possible because of the not uniform geometry in plan or elevation.

Figure 4: Construction of the central spans

The shorter spans over lower parts of the bridge are a smaller 80m and are made from reinforced prestressed concrete hollow sections. All the piers are reinforced concrete. Although it is apparent in figure 4 that all the piers have some widening at the base for protection and it is quite probable that beneath the waters surface this is widened further, more substantial protection from collision from large shipping vessels can be seen surrounding the piers of the central access spans as shown in figure 5.

These ‘buffer’ areas around the pier are designed to absorb the load of any accidental impacts so that the structure of the bridge is retained. It may be possible that on the lower areas of the bridge where the protection is not so apparent, that the bridge has been designed so that in the unlikely event of a collision, should one of the piers be wiped out, the bridge will not collapse. This is just conjecture, but would have been standard practice for bridge design at the time of the Rio-Niteroi’s construction.

Figure 5: Collision protection at the bases of central span piers.

4.4 Bearings

Given the period of construction the bearings would not be the Teflon rockers that are commonly used today. It is also unlikely that concrete hinges are used as they are cheap and susceptible to cracking and corrosion. It is also unlikely that these were used as they need to be cast in-situ. On a project this large it would be sensible to suggest that sliding/expansion bearings were used, although not on the rise and all to the central spans as they can not support horizontal load. It is possible a system similar to the steel rocker that was commonly used in the seventies and eighties would have been used, although the bridge may have been slightly too early for these to be around at the time of construction. If and when they were needed, there might be some steel fixed bearings, but again, these are unlikely to be used, as the climate would prescribe that the effects of temperature would deem them unsuitable.

4.5 Sections and post-tension girder boxes

The majority of the bridges deck is made from reinforced concrete girder boxes that have been post tensioned and mate cast so they can be assembled easily on site.

Figure 6: Annotated diagram of a concrete box girder

Figure 7: Diagram showing the principle of post-tensioning
The principle of post-tensioning is a relatively simple one, but remarkably effective nonetheless. As shown in figure 7 a thin tube is cast into the beam/web/girder etc, and is allowed to set. Once set a tension bar is fed through and locked off with a steel plate on one end. The bar is put into tension and locked off at the other end. The two plates and the bar then put the concrete into compression. If enough compression is put into the concrete, when the member is put into situation where it would otherwise of been in tension, all that happens is the compression is relieved from the concrete. If done well, you can prevent the concrete from ever going into tension, as shown in figure 8.

**Figure 8:** Top: BMD of normally loaded continuous bridge. Middle: BMD of post tensioned structure. Bottom: Stress state showing stages of pre-stressing.

As you can see in figure 6 the width of the reinforced concrete is not constant. The advantage of varying the width is that at the mid-span (right hand side of the section in figure 6) it can have a thicker top to take the compression. Over the support where this is not needed, extra concrete can be put in the base and in the webs (left hand side) to take hogging moment.

Box girders are strong against torsional effects because of their closed geometry. Diaphragm walls would be placed above the piers to prevent punching, with a hollow section for inspection access.

**Figure 9:** Post-tensioning tendons shown on the inside of girders could keep members in tension outside of the concrete rather than within it as in figure 7.

As already discussed the concrete box girders are reinforced. As the sections are precast and floated out and lifted into position, it is likely that this is achieved using post tensioning tendons and deflectors (shown above). Each section would be mate-cast and assembled on site where they can be tensioned together and glue put in the joints. This is the cheaper option rather than casting them in situ. Adding the post tensioning has the effect of reducing the maximum sagging moment in the center of the span, and allowing the girders to span longer than they otherwise could. This is shown in the two bending moment diagrams in figure 8.

It is the fact that the top flange is in such a high hogging moment over the supports that results in the large base and webs. As the depth of the girder changes or the widths of its webs and flanges vary, the EI value changes. The means the stiffness will vary. The stiffer areas attract more of the moment.

It is plausible to suggest that this method is used over most of the bridges structure, although in some areas the external dimensions of the deck change to accommodate for the hogging moment (rather than the change in thickness being internal) taken over the piers as seen in Figures 4 and 5. This is where the cantilevered steel box girders are used.

### 4.6 Geological conditions and foundations

A study carried out by the Ciade Universitaria tells us the geological conditions in the area. Despite the apparent mountainous landscape in the background of the pictures of the bridge, the bay itself is a relatively flat and uniform coastal plain with a bedrock consisting of Pre-Cambrian crystalline rocks [6]. This would have been great news for the design of the foundations. Pre-Cambrian rock is generally speaking, crystalline granite rocks deposited millions of years ago and, geotechnically speaking, provides a very high quality hard stable base for the piers.

Covering the bedrock is a layer of silty deposits, mainly sandy clays that ranges from 10m-15m [6]. The environment is constantly changing due to the tidal behaviour in the bay, but generally, changes are not overly varied, and the 10-15m limits remain roughly the same.

With the piers being very regular distances apart it would be safe to assume that conditions are relatively constant along the length of the bridge, with little need to specifically place piers in locations where the conditions are better than other places along the bridges length. The foundations of the piers likely pass through the silty layers to the strong bedrock below and being drilled into the granite where needed.

It is possible that the central spans with longer distances between piers have been positioned as such as this was where the conditions made building foundations most difficult, so by placing the larger...
spans in these locations, less piers were needed making construction slightly quicker, easier and cheaper.

5 Loading

This paper analyses the bridge to limit state (ULS) criteria using BS5400., and Serviceability Limit State (SLS).

The following load factors are used throughout the analysis:

\[ \gamma_{f3} = 1.00 \quad \text{SLS factor} \]
\[ \gamma_{f3} = 1.10 \quad \text{ULS factor for steel} \]
\[ \gamma_{f3} = 1.10 \quad \text{ULS factor for Concrete} \]  

The bridge must be analysed at both SLS and ULS for the following five conditions:
1. All permanent loads
2. Same as 1) plus wind
3. Same as 1) plus temperature
4. All permanent loads plus motional live loads
5. All loads plus friction at supports

The carriageway it just over 26m wide, which means that notionally we have 7 lanes, 6 traffic lanes and the rest is distributed accordingly:

5.1 Dead load

The following load factors are used:

\[ \gamma_p = 1.05 \quad \text{ULS factor for steel} \]
\[ \gamma_p = 1.00 \quad \text{SLS factor for steel} \]
\[ \gamma_p = 1.15 \quad \text{ULS factor for concrete} \]
\[ \gamma_p = 1.00 \quad \text{SLS factor for concrete} \]  

The following are assumed weights for dead and superimposed loads:

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>7850 (8243 factored)</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>2400 (2760 factored)</td>
</tr>
<tr>
<td>Asphalt</td>
<td>2300 unfactored</td>
</tr>
</tbody>
</table>

Table 2: Unit Weights used in analysis

The area of reinforced concrete in 11m² in section for half the bridge deck, the dead load on each pier is as follows:

\[ W_{\text{dead}} = 11 \times (2760 \times 10)_{\text{(factored)}} = 303kN/m \]  

5.2 Super-imposed dead load

In addition to the unit weights shown in Table 2 above, the following load factors are used for super-imposed live load:

\[ \gamma_p = 1.75 \quad \text{ULS factor} \]
\[ \gamma_p = 1.20 \quad \text{SLS factor} \]

\[ W_{\text{s.i.d}} = 1.75 \times ((2300 \times 10 \times 1.5) + (2200 \times 10 \times 1)) \]
\[ W_{\text{s.i.d}} = 98kN/m \]  

5.3 Live load

The following load factors are used for accidental skidding and collision with piers:

\[ \gamma_p = 1.25 \quad \text{ULS factor} \]
\[ \gamma_p = 1.20 \quad \text{SLS factor} \]  

5.3.1 HA Loading

HA Loading consists of the factored ULS load and a knife-edge load KEL applied in each lane over the supports.

8.25kN/m² (factored ULS - intensity of live load)

\[ W_{\text{HA}} = 8.25 \times 12.3m = 101.5kN/m \]

165kN per lane KEL.

5.3.2 HB Loading

HB Loading is the load from an abnormally large truck:

154kN per wheel (factored ULS) placed on either side of a lane and

354kN load acting horizontally from each wheel under the HB load to simulate accidental breaking.

The above conditions represent the worst case scenarios on a concrete section of the bridge midspan on an 80 span.

5.3.3 Accidental impact

The Rio-Niteroi Bridge is susceptible to horizontal loads on the piers due to impact from boats. Using the laws of momentum, I can find that the force (that acts roughly 3m above water level) acting on the pier on impact is nearly 50MN. The bridge has been designed to cope with this by providing protection at the pier bases.

5.4 Wind load

The following load factors are used during erection when:

a) Erecting bridge and superimposed loads are in combination with wind and other loads:

\[ \gamma_p = 1.10 \quad \text{ULS factor} \]
\[ \gamma_p = 1.00 \quad \text{SLS factor} \]
b) Superimposed and wind effects are only considered:
\[ \gamma_{ULS} = 1.40 \quad \text{ULS factor} \]
\[ \gamma_{SLS} = 1.00 \quad \text{SLS factor} \]

To find the wind gust we use the equation:
\[ V_c = V_c K_1 S_1 S_2 \]
(8)

where \( V_c \) is the maximum gust we expect, \( V \) is the local wind speed (25m/s), \( K \) is the wind co-efficient (1.81), \( S_1 \) is the funneling factor and so can be taken as 1.00, \( S_2 \) is the gust factor (1.42). All of this gives:
\[ V_c = 25 \times 1.81 \times 1 \times 1.40 \times 1.40 = 90.0 \text{m/s} \]

The horizontal load acting at centroid of the section of the bridge concerned is:
\[ P_h = q A_1 C_D \]
(9)
\[ P_h = \sum (0.613 V_c) A_1 \]
\[ P_h = \sum 100 A_1 N \text{m}^{-2} \]

where \( q \) is the dynamic pressure head, \( A \) is the area of the bridge in question, \( C_D \) is the drag co-efficient.

The uplift or down force is given by:
\[ P_u = q A_3 C_L \]
(10)
\[ P_u = \sum (0.613 V_c) \times 0.4 \times 1.4 \times A_3 \]
\[ P_u = \sum 31 A_3 N \text{m}^{-2} \]

For an example calculation I have analysed the bridge with \( P_h \) and \( +P_u \), combined as this is the expected worst case scenario.

5.5 Temperature load

The following load factors are used when movement is restricted (including frictional):
\[ \gamma_{ULS} = 1.30 \quad \text{ULS factor} \]
\[ \gamma_{SLS} = 1.00 \quad \text{SLS factor} \]

However, if it is a case of a temperature differential we use the following factor:
\[ \gamma_{ULS} = 1.00 \quad \text{ULS factor} \]
\[ \gamma_{SLS} = 0.80 \quad \text{SLS factor} \]

Assume the maximum temperature is 30° and the minimum is 17° [8].

Using the British standards we need to add 13.5° to the deck and 2.5° on the underside. The reverse temperature difference is 1.5° on the exposed surface of the upper slab and 3.5° on the underside of the deck slab. On the bottom slab the underside differential is 6.5° and 6.3 on the inside of the box girder.

The following calculation outlines the expansion of bridge:

\[ \Delta T = 20°C \]
\[ \alpha = 12 \times 10^{-6} /°C \]
\[ \varepsilon = \Delta T \times \alpha = 240 \mu\varepsilon \]
\[ \alpha = 240 \times 10^{-6} \times 1.3 \times 10^7 = 3.12 m \]
\[ \therefore \]
\[ \varepsilon_{c} = 240 \mu\varepsilon \]
(13)
\[ \sigma_{c} = 240 \times 10^{-6} \times 30,000 = 7.2 N / mm^2 \]

The bridge needs to be allowed to expand by 3.12m or else huge stresses will be induced in the bridge. It is most likely that a number of expansion joints would be placed along the bridge for example at the beginning and end of the bridge. Other places that are quite likely are where the bridge passes over the island.

5.5.1 Temperature difference

\[ T_1 = 13.5° \quad T_2 = 2.5° \]
\[ \varepsilon_1 = 13.5 \times 12 \times 10^{-6} = 162 \mu\varepsilon \]
\[ \varepsilon_1 = 2.5 \times 12 \times 10^{-6} = 30 \mu\varepsilon \]

as \[ E_{concrete} = 3000; \]
\[ \therefore \sigma_1 = 162 \times 10^{-6} \times 30 = 4.86 N / mm^2 \]
\[ & \sigma_2 = 30 \times 10^{-6} \times 30 = 0.9 N / mm^2 \]
(14)

If I assume that the axial stress is constant and the stress caused by moments induced by the temperature difference is zero about the neutral axis, the axial stress is half way between \( \sigma_1 \) and \( \sigma_2 \):
\[ \sigma_{A} = \frac{4.86 - ((4.86 - 0.96) / 2) = 2.91 N / mm^2}{2} \]

\[ \text{Force}_{(axial)} = \sigma_{(axial)} \times A \]
(15)

\[ \text{Moment} = \frac{\sigma A l}{y} \]

The second moment of area of the concrete box girder is 10.6x10^{12}mm^4 and has a cross sectional area of 7.56x10^6mm^2. Using equations (15):

\[ \text{Force}_{(axial)} = 2.91 \times 7.56 \times 10^6 = 22 MN \]
\[ \text{Moment} = \frac{2.91 \times 10.5 \times 10^{12}}{2300} = 13.3 MN m \]

These loads are significant and can not be ignored in the analysis.
5.5.2 Temperature effects on the piers

The following calculations check that the pier is not overly affected by temperature. One would not expect it to, because one would expect two piers that are along side each other to expand or contract at roughly the same amount being exposed to the same conditions. One would therefore expect minimal differential behaviour as a result of temperature change. Assuming a twenty degree change in temperature and applying it to the tallest pier:

\[ \Delta H = \Delta T \times a \]
\[ \Delta H = 20 \times 72m \times 12 \times 10^{-6} \]
\[ \Delta H = 0.017m \]

This is a minimal change so one can expect the stresses to be equally insignificant:

\[ F = \Delta H \times \frac{3EI}{l^3} \]
\[ F = 17 \times \frac{3 \times 3000 \times 10.6 \times 10^{12}}{72000^3} \]
\[ F = 4.3kN \]
\[ M = 4.3 \times 72 \]
\[ M = 309kN.m \]
\[ \sigma = 3N.mm^{-2} \]

It is clear to see that the loading is minimal and can be neglected.

5.6 Other load factors

There are many other forms of loading that can affect a bridge. This paper covers the most important. It goes without saying that a bridge in Brazil need not look at snow loading or the possibility of loading from water freezing around the piers.

It is also safe to discount the affects of vandalism and small vibrations from things such as pedestrians on a bridge of this scale because their effects will be negligible compared to the other loads.

6 Structural Analysis

Now that all loadings have been calculated it is possible to analyse the bridge. A full set of calculations would include effects of the shrinkage and creep of the concrete. As steel tendons are present in the structure stress relaxation would also have to be considered. It is obviously safe to exclude loadings such as snow loads when you consider the climate and location of Rio de Janeiro. Figure 12 Shows assumes dimensions and calculated loads.

![Figure 12: Dimensions and loading diagram of the concrete box girder.](image)

The following equations outline the bending moment that would be felt at mid span if the post-tensioning were not present. This is the worst-case situation for the bridge with all loads being applied simultaneously.

![Figure 13: Load Case 1](image)

\[ M_{mids.} = \frac{w_{live} l^2}{8} + \frac{KEL l}{4} \]
\[ M_{mids.} = \frac{(6 + 25) \times 80^2}{8} + \frac{13 \times 80}{4} \]
\[ M_{mids.} = 250MN.m \]

Load case 2 where there is no KEL load, only a HB load in the central bay is not critical as the KEL loading is far larger.

If there was a full set of data available it would be possible to size the members fully in a full report. It is also worth noting that this bridge may not have been built to British standards, even though the designers were British, so member sizes may be a lot different if the bridge had been constructed in England.
7 Cantilever Steel Span

Figure 14: A sketch of the central span, which is a steel box girder section.

Having the Central span as a cantilever system has the advantage of being able to span further than a continuous deck. It is safe to assume that the designers positioned the steel box girder in place where there was going to be zero moment, making the steel girder a simply supported deck, as shown in figure 15 below.

Figure 15: Likely bending moment diagram for central span.

8 Durability and Serviceability

The Rio-Niteroi bridge has the unfortunate position of being located in an environment that isn’t what one could describe as moderate. The salt water, the waves, sea air, humid climate with hot temperatures mean that the bridge is constantly being attacked.

The two biggest culprits for the corrosion of materials is most definitely the salt quickening the rate at which the steel can corrode if left exposed by anything as seemingly insignificant as a nick in the paintwork and the heat causing cracks in the concrete which in turn expose reinforcement to the elements for them to corrode.

Figure 16 shows the gantry under the steel section of the bridge so that workers can continually keep the steel painted. The hollow girders provide ease of access to inspect the bridge of signs of cracking which could be early signs of the bridge failing.

9 Improvements

It would be very easy to confuse this with future changes. Comments made in Future changes are necessary works, improvements are suggestions that could be made to better the bridge either from an aesthetic or functional point of view.

When one looks at the steel section of the bridge it does definitely stand out like a sore thumb. It could be possible to disguise this change in material more subtly, possibly by a different colour of paint that was closer to its neighboring bays made of concrete.

Currently the bridge is not lit up in the evenings. The bridge is a symbolic link, an engineering achievement and as such it should be shown of for all to see. Maybe something simple like lighting the piers up in the evening to show off the bridge would just add something to it to show that everyone see its importance within their communities.

As I mentioned in the second part of the paper, the fittings (lamp-posts etc) are all painted white and stick out very awkwardly against the sea, landscape and the bridge itself, if they were painted a duller grey then maybe they would blend in more successfully.

It is very easy to criticise something that was not your own work once it has been completed, so I believe it is slightly unfair to lay into the bridge too much. The bridge is a success and deserves some praise. It was a crowning engineering accomplishment when it was built and can still be appreciated by so many today.

10 Future changes

The Construction of the bridge doesn’t lead itself to being expanded very much. I doubt very much if the designers built the bridge with the idea of expanding the bridge in the future. That is not to say that it probably couldn’t do with expansion. The bridge provides an inter-city link that is used by 140,000 vehicles a day. The bridge is not wide enough to house another lane of traffic without expansion unless it was to use up all remaining hard shoulder space and some of the central reserve.

If this was feasible it is most likely that work would need to be carried out to strengthen the bridge. The use of fibre composites with concrete is becoming increasing popular. In some cases it can be used as a strengthening tool simply by wrapping it around existing members. It can also be used in repair work, for example to replace or repair damaged post tension tendon bars [10].

Fibre composites also serve as a seal if the concrete has cracked, and could be used to prevent the elements reaching the reinforcement and corroding it.
11 Acknowledgments

A special note of thanks should go to Sarah Huelin for providing the initial idea of compiling this paper on the Rio-Nieroi Bridge following our travels to South America in the summer of 2005. Unfortunately we did not stay in Rio long enough to explore the bridge.

The template of this paper is the work and property of Professor Tim Ibell of the University of Bath. A note should be made that if it was not for the work that has gone into the preparation of his Bridges 1 notes the basic understanding needed to write this paper would not have been there.

12 References

[9] Ibell, T., Bridge Engineering, University of Bath, Department of Architecture and Civil Engineering, 2008