ANALYSIS OF THE TASMAN BRIDGE, TASMANIA, AUSTRALIA

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Abstract: This article provides an analysis of the Tasman Bridge, located in the city of Hobart, Tasmania, Australia. The article includes a brief history of the bridge, and an analysis of the aesthetic, strength, construction, serviceability, temperature, creep, wind, susceptibility to damage and possible future changes. It should be noted that not all points in this article are strictly fact, as some are be based on engineering judgement based on evidence.

Keywords: Tasman, Bridge, Prestressed, Concrete, Highway.

1 Introduction

Tasmania is a state on Australia and is an island located off the south eastern tip of mainland Australia, separated from the mainland by the Bass Straight. The island has a population of fewer than 500,000 and an area three and a half times the size of Wales.

The Tasman Bridge is located in Hobart, the states capital and lies on the Derwent River, connecting the Eastern shore suburbs with the CBD of the city on the western shore. It was completed in 1964 and replaced a two lane concrete floating bridge called the Hobart Bridge, which was the first of its type ever constructed.

2 Aesthetics

Through its form, a bridge should show that it is stable, and it should be obvious how forces are transferred through the structure, simplicity is important for good aesthetics. As you approach the Tasman Bridge, the piers appear to be very solid, due to their cross section, giving an impression of stability. It is obvious to see how forces are transferred through the deck to these piers and it is in this way that the bridge clearly shows its function.

In general the proportions of this bridge feel correct, the span to depth ratio appear to be right. However, it could be argued that, if viewed in plan, the piers appear too slender in comparison to the deck and overall span; this is because of the shape of the pier columns, which are rectangle as apposed to square, and have a longer breadth than width. Figure 2 also shows there is a pier missing on the right hand side; this is after the bridge collapse. The pier could not be replaced and as it was not part of the original design, it should not be criticized.

Figure 1: Photo showing ‘solid piers’

Figure 2: Piers appear slender
It could be argued that, because the breadth of the columns does not change along the length of the bridge, the end piers look quite broad. This is because the shorter piers are stiffer and thus attract more moment to them, thus they cannot be made thinner. Not changing the thickness of the piers would also simplify construction.

Figure 3: Piers appear stocky

The large number of repeated piers is what gives the bridge a lot of its character. However the piers continue onto the deck and break the line of the deck, there are also a large number of vertical breaks in the central span on the deck that could easily have been covered. These breaks in the line of the deck detract from the aesthetics of the bridge by preventing the eye from moving easily across the bridge. It would appear that these ‘vertical breaks’ are I beam webs, suggesting these beams were designed like steel beams.

The central navigation span has to be wider to allow ships to pass under the bridge. As the span is longer, the deck must be thicker to support the extra load. The other reason that this part is thicker is because it is that this part of the bridge is a cantilever and requires the extra depth to resist bending moment. However, the way this has been achieved means the deck abruptly changes in depth, detracting from the aesthetics. This would be difficult to overcome, it could have been solved by using a concrete box section and varying the thickness internally, thus not visually changing the depth of the section.

Figure 4: Abrupt changes in deck thickness

It is going to be difficult to build a bridge which blends with such a beautiful environment. The bridges greatest strength in this respect is that it complements the surrounding environment rather than competing with it, which may happen with a cable stayed or suspension bridge, which have large towers.

Figure 5: Curvature of bridge blends with environment

The bridge consists of matt finish smooth concrete. The texture of this bridge doesn’t really add anything in terms of aesthetics. The colour of concrete doesn’t really add anything to the aesthetics of the bridge, the stain on the concrete is not particularly appealing.

3 Construction

Tasmania had a population of 484,700 in 2005, and is over three times larger than Wales. The island is separated from the mainland of Australia by the Bass Straight.

These two factors make the choice of construction of very important in terms of economics and what could actually be achieved. As the local workforce would have been relatively unskilled and small, compared with a larger country, it would have been important to choose a construction method that wasn’t too labour intensive or complicated.

Tasmania hadn’t built any bridges on this scale previously and so it is unlikely that there would be the equipment located locally, this means that any equipment required for the construction would have to be imported, adding to the bridges cost.

Under these circumstances, using pre-cast sections would have been the economical and practical option, as casting is more labour intensive and using pre-cast elements would make construction times faster.

Economics would most like have been the biggest deciding factor, as the bridge was being built for a very small population, therefore making it difficult to justify spending a lot of money. For these reasons pre-tensioned pre-cast concrete was chosen for all major parts of the bridge, including beams, abutments and piers [4].

The bridge was relatively expensive compared to similar concrete bridges at the time, for example the Medway Bridge in Kent. This was because all pre-stressing steel and equipment, including jacks, strands and wires, had to be imported from the UK, along with some engineering staff, while all reinforcing steel had to be imported from the Australian mainland [4].

The Hobart Bridge, the bridge which the Tasman Bridge replaced, had a lift span at one end. It was located close to the western shore and the span lifted to allow ships to pass. The Tasman Bridge took four years to complete and for the sake of the local economy, ships would have to of been able to pass during that time, and the old Hobart Bridge would have to remain so that Eastern shore residents wouldn’t get cut off from the CBD of the city. The Tasman Bridge’s span to allow ships to pass is located in the centre of the river. It would have been unrealistic and probably impossible for bulk carriers to pass through the Tasman Bridge, turn towards the western shore, and then pass through the Hobart Bridge, this is best illustrated in the picture below. For this reason the bridge was constructed from the Eastern
shore so that the shipping lane would only be closed for a small amount of time at the end of construction.

**Figure 6:** Construction starting from eastern shore

**Figure 7:** Ship passing through Tasman bridge and Hobart Bridge

The founding conditions on the river bed are generally quite good. Bedrock, dolerite and basalt, are at depths of approximately 20 metres below silt, making a bridge made up of many small spans feasible. 1.37 metre diameter steel tube piles were driven using a template pontoon to bedrock [5] [1]. The steel piles were then filled with concrete. Difficulty during construction came from a section where the bedrock depth increases to approximately 90 metres; this was unexpected and added time and cost to the construction of the bridge. This problem was solved by installing the piles to over 60 metres, each pile was then ‘cyclically preloaded up to its design specification’ to remove the excessive deflection of the piles under load [6]. If the bridge was of different design, i.e. cable stayed or suspension, then the piers could have been founded on the shallow bedrock only, simplifying the construction.

From photos of the piers it can be seen that the piers are made up from many smaller units, as the joint lines can be seen. This suggests that the piers are made up of pre-cast units. The likely method of construction would have been to place pre-cast section using a floating crane. Joints would likely have been formed using a resin. Continuous stressing wire would have been run through the units and tensioned with the use of hydraulic jacks, thus putting the pier into compression. As mentioned previously, this method of construction was probably the best choice as it was the probably the most economical, as it requires a much smaller labour force than if the piers were cast in situ, and is a faster method of construction.

The construction of the main viaduct is typical of many highway bridges. One contractor is responsible for construction of the piers, and once this process has started, the contractor responsible for placing the pre-cast can start, making for an efficient construction process. The crane used for placing the beams on their bearings was a 100 tonne capacity Goliath crane. The Goliath crane was supported two floating steel tower formworks which were linked by steel girders [4]. The formwork described weighed on the order of 800 tons and was imported from Germany [4]. The steel girders allow the Goliath crane to travel along the length of the span to pick up and place concrete beams.

The height of the bridge over water varies from approximately 15 metres to 45.7 metres. To deal with the changes in height, the formwork was able to change height also. This was achieved by using ‘lift slab jacks’ which raised the height of the formwork. Formwork units 2.3 metres height were inserted or removed to raise or lower the formwork respectively.

As the bridge is on a gradient, it is very unlikely that roller bearings were used in construction as they cannot support any horizontal load. Steel rocker bearings would be the most likely choice of bearing, providing a pin jointed connection with the pier. Rubber pad bearings would also be unlikely because of their short life span.

The main navigation spans were assembled on continuous formwork [4]. This would have been the same formwork used for the construction of the viaduct spans. It is possible that the reason this was done was to join together the pre-cast sections while supported on the formwork and once this was complete the continuous stressing wire placed through the sections would have been put into tension using hydraulic jacks to put a compressive force into the concrete so that the cantilever could support itself and the formwork could be removed.

More likely the formwork was required to help maintain balance during construction, as the piers are quite slender and temporary supports may have been necessary. A construction photo is shown below, also shown in the picture is a walkway between the two cantilevers [5].
A potentially cheaper and simpler method of constructing the cantilever sections could have been to use a launching girder to place sections and use a temporary fixing at the pier head to eliminate the need for any temporary support. The viaduct beams could then have been lifted by cranes on floating barges, which would have eliminated the need for all of the formwork used in construction.

Once the cantilevers are completed, a 29 metre drop in span was placed to complete the central navigation span. There would have been a designed time for the placement of the drop in span, to account for temperature effects, so that the span would fit into the gap. It appears from photos that the drop in span is joined to the cantilevers with a halved joint.

If the centre span of the bridge was fully loaded while the anchor spans had no load on them, there is the potential that the live load on the central span would outweigh the deadweight of the anchor span and cause collapse. To prevent this the anchor span is held down by the dead weight of the deck of the adjacent viaduct span via a halved joint, providing a factor of safety [4].

4 Susceptibility To Damage

The River Derwent is a shipping lane for tourist and industrial ships. Located a few miles up river from the bridge is Zinifex Limited Zinc refinery, which is located on the river so that raw ore can be easily transported by ships to the refinery [7].

The design of the bridge has taken this into account by having a much wider, taller central span, known as the central navigation span of 94 metres and a clear height of 45 metres, and two flanking secondary navigation spans of 60 metres. The spans allow for the passing of small bulk carriers like the one shown below.

The pile caps of the piers which make up the navigation span are the most susceptible to damage from large vessels, as this is the only span which these ships are allowed to go through. These pile caps are protected by concrete gravity fenders [3], concrete rings are hung from the pile caps and move if impacted, absorbing energy. These two are the only protected piers of twenty one.

On the evening of the 5th of January 1975, a bulk carrier named ‘MV Lake Illawarra’ carrying ore to the Zinc refinery drifted off course and tried to correct its course to get through the central navigation span but was unable to. The ship collided side on with the bridge, taking out piers 18 and 19 and brought down 3 spans of deck totally 128 metres. The deck fell onto the ship sinking it below the bridge and killing seven crew members, along with five killed who drove off the bridge [8].

The force of a ship colliding with a bridge pier is quite large and is shown in a rough calculation below (the average speed of a bulk carrier is 11 knots = 5.6m/s):

\[ F = m(v - u)/t \]
\[ F = 7274 \times 1000 \times (5.6 - 0)/1 \]
\[ F = 40.7 MN \]

Assuming collision occurs at 15m above water, and force impacts single column acting as a cantilever.

\[ M = FL \]
\[ M = 40.7 \times 15 \]
\[ M = 608 MNm \]

\[ \sigma = M/l \]
\[ \sigma = 608 \times 10^9 \times 1500/(750 \times 3000^3)/12 \]
\[ \sigma = 540 N/mm^2 \]

The impact time may have been faster than 1 second, making this calculation less conservative. The stress shown would easily be enough to remove the column, causing collapse.

The design could have been improved by making the bridge deck redundant i.e. placing enough hogging and sagging reinforcement in the deck so that if one pier is removed by collision the deck would remain.

To protect all 21 piers of the bridge would be uneconomical compared to the risk to life and property. Reducing the number of piers would reduce the risk of a
ship colliding with a pier. Based on this a different bridge type may have been more suitable, such as a cable stayed bridge. This type of bridge could have achieved the height clearance while reducing the number of piers required and the main piers could have protected, therefore reducing the risk of collapse from ship collision.

5 Reconstruction and Changes

The collapse of the bridge left the eastern shore resident cut off from their jobs and important services such as the hospitals. During the reconstruction a ferry service transported people across the river and a temporary single lane floating bridge was constructed in 1975.

The reconstruction of the bridge would have posed many problems for the designers and contractors. The first task involved clear debris from the river bed to make room for new foundations. Special techniques had to be developed to locate debris as the water depth was over 30 metres and the visibility was only a few metres [9].

Pier 19 was rebuilt. The Lake Illawarra sank directly below pier 18, where it lies today [10], along with a large amount of debris, and it was decided not to rebuild it. This meant that a normal 42 metre span had to be rebuilt as well as designing a deck to span 84 metres. Reconstructing the bridge made more sense than demolishing the bridge and starting again with a new design, as the bridge is iconic of the city and it would have been far cheaper to repair the bridge than demolishing and constructing a new design. Reconstruction cost $40 million (Aus) and was completed in 1977.

Engineers reconstructing the bridge were concerned about the vulnerability of the remaining piers, if they were impacted, it could cause a domino collapse of the bridge [John Holand link]. Temporary steelwork was erected to protect the piers and is shown in the image below:

Figure 11: Thicker deck on span

Before the bridge collapse, the bridge was approaching full capacity [Statistics link], and so it was decided that as part of the reconstruction the bridge would be widened from four lanes to five lanes. As the piers were unaltered during the widening, it can be presumed that they were designed with the possibility of widening in the future.

In the original design of the bridge, the four lanes went almost to the edge of the deck, with the footways on the edge of the deck and the beams were the end of the pier cap; this can be seen in the image below. In order to widen the bridge to accommodate an extra lane, the deck would have to cantilever transversely over the end beam on both sides; seen below.

Figure 12: Transverse cantilever deck on reconstructed bridge

The widening of the bridge would have the most affect on the outer beams of the deck, as these two beams now support the transverse cantilever section of the deck. As a result, post tension cables and deflectors have been added to the two outer beams to increase their bending capacity. The reconstructed span has no post tension on it and must have therefore been designed to support five lanes of traffic. Before and after photos of the deck between pier 20 and 21 can be seen below.
As this bridge has already been widened once it is unlikely and probably impossible to increase its capacity any further. Since the reconstruction of the Tasman Bridge another bridge has been constructed up river, called the Bowen Bridge to try to reduce congestion at peak times on the bridge. The beam-slab design of the Tasman Bridge was probably among the better choice of bridge designs to widen, when compared to other designs such as an A frame cable stayed bridge.

6 Bridge Geometry

Bridge dimensions have been determined by analysing photos to get approximate lengths. The figures below are used in all following calculations.

First Moment of slab = \((400 \times 2890) \times 1375 = 1589 \times 10^6 \text{ mm}^4\).

Area of section = 
\((1.44 \times 10^6 \text{ mm}^2) + (400 \times 2900) = 2.6 \times 10^6 \text{ mm}^2\).

\[ \Delta z = 1589 \times 10^6 \text{ mm}^4 \div 2.6 \times 10^6 \text{ mm}^2 = 606 \text{ mm} \]  
(Towards slab).

\[ I_{centre} = (\text{Beam} \text{ I Local} + A y^2) + (\text{Slab} \text{ I local} + A y^2) \]

\[ I_{centre} = (7.5 \times 10^{11} \text{ mm}^4 + 5.4 \times 10^{11} \text{ mm}^4) + (1.54 \times 10^{10} \text{ mm}^4 + 6.9 \times 10^{11} \text{ mm}^4) \]

\[ I_{centre} = 2.0 \times 10^{12} \text{ mm}^4 \]

\[ \Delta z = 1045 \times 10^6 \text{ mm}^4 \div 2.18 \times 10^6 \text{ mm}^2 = 480 \text{ mm} \]  
(Towards slab).

\[ I_{End} = (7.5 \times 10^{11} \text{ mm}^4 + 3.4 \times 10^{11} \text{ mm}^4) + (1.01 \times 10^{10} \text{ mm}^4 + 6.1 \times 10^{11} \text{ mm}^4) \]

\[ I_{End} = 1.7 \times 10^{12} \text{ mm}^4 \]

\[ A_{deck} = A_{slab} + A_{beam} \]

\[ A_{deck} = (400 \times 15800) + 6 \times 1.44 \times 10^6 \text{ mm}^2 \]

\[ A_{deck} = 7.79 \times 10^6 \text{ mm}^2 \]

\[ I_{Deck} = 4 \times I_{centre} + 2 \times I_{End} = 1.14 \times 10^{13} \text{ mm}^4 \]
7 Loading

Loading values for the analysis of this bridge were obtained from the British Standards. The various loading conditions are detailed below.

7.1 Dead Load

Concrete Density = 2400 kN/m³

\[
W_{\text{dead}} = W_{\text{dead}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{dead}} = A_{\text{dead}} \times \text{Density}
\]

\[
W_{\text{dead}} = 15 \times 2400 = 360 \text{kN/m}
\]

\[
W_{\text{deadULS}} = W_{\text{dead}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{deadULS}} = 360 \times 1.15 \times 1.1 = 455\text{kN/m}
\]

\[
W_{\text{deadServ}} = W_{\text{dead}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{deadServ}} = 360 \times 1.1 \times 1 = 396\text{kN/m}
\]

7.2 Superimposed Dead Load

2% Road rise over half of road.

Fill Density = 2200 kg/m³

\[
\text{Area} = (2/100) \times (13.4/2) \times 13.4 = 0.9 \text{m}^2
\]

\[
W_{\text{fill}} = 0.9 \times 2200 = 19.8\text{kN/m}
\]

Bitchumen Density = 2400 kg/m³

\[
W_{\text{bitchumen}} = 0.08 \times 13.4 \times 2400 = 25.3\text{kN/m}
\]

\[
W_{\text{deadULS}} = W_{\text{dead}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{deadULS}} = (19.8 + 25.3) \times 1.1 = 83\text{kN/m}
\]

\[
W_{\text{deadServ}} = W_{\text{dead}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{deadServ}} = (19.8 + 25.3) \times 1.2 \times 1.0 = 52\text{kN/m}
\]

7.3 HA Live Load

2 lanes @ 9 kN/m + 2 lanes (1/3) 9 kN/m = 24 kN/m of deck.

\[
W_{\text{HAULS}} = W_{\text{HA}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{HAULS}} = (24) \times 1.5 \times 1.1 = 39.6\text{kN/m}
\]

\[
W_{\text{HAServ}} = W_{\text{HA}} \times \gamma \times \gamma_{f3}
\]

\[
W_{\text{HAServ}} = (24) \times 1.25 \times 1 = 30\text{kN/m}
\]

\[
\text{KnifeLoad}_{ULS} = 2 \times 120 + 2 \times (120/3) \times 1.5 \times 1.1
\]

\[
\text{KnifeLoad}_{ULS} = 528\text{kN}
\]

\[
\text{KnifeLoad}_{Serv} = 2 \times 120 + 2 \times (120/3) \times 1.25 \times 1
\]

\[
\text{KnifeLoad}_{Serv} = 400\text{kN}
\]

7.4 HB Live Load

\[
H_{\text{BULS}} = W_{\text{HA}} \times \gamma \times \gamma_{f3}
\]

\[
H_{\text{BULS}} = (450) \times 1.5 \times 1.1 = 743\text{kN}
\]

7.5 Load Analysis

\[
M_{\text{max}} = (wL^2) / 8 \times (pL) / 4
\]

\[
M_{\text{max}} = 128 \times 10^6 + 5640 \times 10^6
\]

\[
M_{\text{max}} = 133\text{MNm}
\]

Long Term Deflection

\[
\delta_{\text{Long}} = \delta_1 + \delta_2 + \delta_4
\]

\[
\delta_{\text{Long}} = 0.81 + 2.52 + 21.7 = 24.3\text{mm}
\]
Beams would be pre-cambered to take account of creep deflection.

Short Term Deflection
\[
\delta_{\text{Short}} = \delta_1 + \delta_2 + \delta_3 + \delta_4
\]
\[
\delta_{\text{Short}} = [(0.3w + p/2)(0.2l)^3 / 3EI) + (w(0.2l)^3) / 8EI + (p(0.6l)^3) / 48EI + (5w(0.6l)^3) / 384EI]
\]
\[
\delta_{\text{Short}} = [(384 + 200)\times10^3(8540)^3) / 3EI) + (30(8540)^3) / 8EI + (400\times10^3(25600)^3) / 48EI + (150(25600)^3) / 384EI]
\]
\[
\delta_{\text{Short}} = 0.35 + 0.35 + 0.4 + 0.49 = 1.59\text{mm}
\]

There are two temperature effects which would have been considered in analysis. One is effective temperature and the other is temperature difference.

9.1 Temperature Effects
\[
\Delta l = \Delta T J'\alpha = 20\times42.7\times10^3 \times 12 \times 10^{-6}
\]
\[
\Delta l = 10.2\text{mm}
\]

Pier Deflects 10mm
\[
P = \delta \times 3EI / l^3
\]
\[
P = 2.16kN
\]
\[
M = P \times l
\]
\[
M = 2.16\times45.7 = 98.7kNm
\]
\[
\sigma = My / l = 0.17N / \text{mm}^2
\]

The value of stress in the calculation above is very small, so the pier is unaffected by temperature effects.

9.1 Temperature Difference
\[
T_1 = 8.4, T_2 = 6.4
\]
\[
\epsilon_1 = 8.4 \times 12 \times 10^{-6} = 101 \mu \epsilon
\]
\[
\epsilon_2 = 86.4 \times 12 \times 10^{-6} = 76 \mu \epsilon
\]
\[
\sigma_1 = 101\times10^{-6} \times 30 \times 10^3 = 3.03N / \text{mm}^2
\]
\[
\sigma_2 = 76\times10^{-6} \times 30 \times 10^3 = 2.2N / \text{mm}^2
\]
\[
\sigma_M = (3.03 - 2.2) / 2 = 0.38N / \text{mm}^2
\]
\[
\sigma_A = 3.03 - 0.38 = 2.65N / \text{mm}^2
\]

This bending moment is not as critical as Load case 1 as live load gives a bending moment of 9MNm.

8 Beam Pre-stress Requirements
\[
M_{\text{live+dead}} = 133MNm
\]
\[
P / A + P.e.y_{\text{hoi}} / I + Mv_{\text{hoi}} / I = 0
\]
\[
P((0.128 + 0.265)\times10^{-6}) = 20.84
\]
\[
P = 53000kN
\]
\[
P_{\text{beam}} = 53000 / 6 = 8830kN
\]

Cables @ 60% Prestress, py=1700 N / mm²
\[
A = 8830\times10^3 / 0.6\times1700 = 8657\text{mm}^2
\]

Use two strand 133 diameter cables in each beam.

9 Temperature
10 Pier Euler Buckling

\[ F = K \pi^2 EI / l^2 \]
\[ K = 2 \]
\[ F = 2 \pi^2 10 \times 10^3 \times 2.11 \times 10^{11} / (45 \times 10^{11})^2 \]
\[ F = 20 \text{MN (short)} \]
\[ F = 60 \text{MN (long)} \]

The pier loading from load case 1 = 12.5MN, this means that the long term loading factor of safety for the piers is only 1.6, which is quite low. The piers have been designed this way so that they are not very stiff and thus are not so affected by temperature effects.

11 Wind

Take \( V \) as 30ms\(^{-1}\), Tasmania has similar monthly wind averages to Anglesey, and from BS5400, Anglesey has an hourly wind speed of 30ms\(^{-1}\).

\[ K_i = 1.74, S_i = 1, S_{\sigma}=1.27 \]
\[ v_c = 30 \times 1.74 \times 1 \times 1.27 \]
\[ v_c = 66.3 \text{ms}^{-1} \]
\[ q = 0.613 v_c^2 \]
\[ q = 2.96 kN / m^2 \]

\[ b / d = 15.8 / 5.7 \]
\[ e_d = 1.35 \]
\[ P_t = q A_c e_d = 348 kN \]

On each deck section.

Transverse Bending Moment on Deck section
\[ M = 8.14 \times 42.7 / 12 = 1855 kNm \]

This isn’t very large considering the large transverse \( I \) value of the deck.

\[ 348 / 2 = 173 kN \]
\[ 173 / 6 = 29 kN \]

A horizontal load of 29kN must be resisted by each bearing.

![Figure 22: Deck wind load on piers](image)

Wind load on Piers, X direction
\[ P_t = q A_c e_d \]
\[ P_t = 2.96 \times 32 \times 0.8 = 75 kN \]
\[ 2 \times 75 \times (45 / 2) = V_b \times 9.95 \]
\[ V_b = 0.34 \text{MN} \]
\[ V_{Total} = 0.74 + 0.34 = 1.1 \text{MN} \]

This is more critical than load case 1, as live load gives a pier compression of only 0.85MN. In combination with dead load this is not enough to cause collapse as short term resistance on the piers is 60MN.

Wind load on Piers, Y direction
\[ P_t = 2.96 \times 3 \times 2.1 = 18.6 kN / m \]
\[ M = w l^2 / 2 \]
\[ M = 18.6 \times 42.7^2 / 2 \]
\[ M = 16900 kNm \]
\[ \sigma = M y / I = 29.8 N / mm^2 \]
\[ \text{Pr estress} = \alpha A \]
\[ \text{Pr estress} = 7.63 \times 4.5 \times (10^3)^2 \]
\[ \text{Pr estress} = 343556 kN \]

12 Cantilever Overturning Moment

The cantilever section of the bridge is balanced by the secondary navigation span, known as the anchor span in cantilever construction, and the adjacent viaduct span. The factor of safety is calculated below.

\[ \Sigma M = 0 \]
\[ M = 346 \times 42.7 \times 81.35 + 346 \times 60 \times (60 / 2) - 560 \times (94 / 2) \times (94 / 4) \]
\[ M = 1200000 + 622000 - 618000 \]
\[ M = 1200000 kNm \]

\[ F.O.S = 1200000 / 618000 = 1.94 \]

If viaduct span were to collapse:

\[ \Sigma M = 0 \]
\[ M = 622000 - 618000 \]
\[ M = 4000 kNm \]

\[ F.O.S = 4000 / 618000 = 0.06 \]

Cantilever is only just prevented from overturning if the viaduct span which contributes to holding up the cantilever was to collapse.
Acknowledgments

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