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A CRITICAL ANALYSIS OF THE AUCKLAND HARBOUR BRIDGE, NEW ZEALAND

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Abstract: This paper critically assesses the Auckland Harbour Bridge in New Zealand to analyse it in terms of aesthetics, structural design, construction and modifications. Comparisons will be made between the structure as it was built, and the form it is in today. Calculations are carried out throughout to demonstrate the methods which were, or could have been used for the design, these will provide an insight into the various loads acting on the structure.

Keywords: *Auckland, Steel truss, Nippon Ties*

1 Introduction

The Auckland Harbour Bridge is a steel truss structure with a total length of 1020m designed by British engineers Freeman Fox and Partners (now Hyder Consulting Engineers). It is currently an eight lane road bridge, made up of the original four lane truss structure completed in 1959 and a four lane cantilever structure from extensions in 1969. The reason for the bridge was to carry the high population from the North Shore into the centre of Auckland, preventing the previous car ferries or 25 mile route needed; this resulted in the expansion of the city.

The Auckland Harbour Bridge crosses the Waitemata Harbour between Northcote and Westhaven and was initially planned to be a road, rail and pedestrian bridge, however to reduce costs and due to concerns of a rail crossing not being needed for some time, the final design was a four lane road bridge.

Construction was between 1955 and 1959, the capacity was then doubled in 1969 using a clip-on design to add a further two lanes to each side of the structure. The final completion of the bridge was as a result of many proposals, commissions and debates dating back as early as 1860.

The total truss consists of a number of suspended and cantilever spans supported by a steel truss below the deck, with the exception of the navigation span. The navigation span is of length 243m with a height clearance of 43m for water traffic [1]. The extension sections follow the same line and height of the truss.

Figure 1: Auckland Harbour Bridge in use

Many options were considered for the design of the bridge, preliminary plans had been drawn up from as early as 1890, but commitment to the construction of a crossing was not found until 1929 [2]. The most favoured design was a suspension bridge with two piers, and many wished for a single arch bridge such as the Sydney Harbour Bridge. These options were primarily rejected due to financial reasons.

Figure 2: Auckland Harbour Bridge

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2 Aesthetics

Fritz Leonhartdz's 10 criteria for bridge design will be used for the aesthetic assessment. This is a widely used list of criteria for bridge design from the famous engineer, but not a necessity or guarantee of a fine bridge.

2.1 Fulfilment of function

The function of the bridge is clearly displayed to convey the image of a strong sturdy structure for the heavy looking truss. The design of the bridge was not to show simplicity but to fulfil its requirements. The purpose for the rise of the bridge in the centre for the navigation span is clear to all, as is the general bridge layout.

2.2 Proportions of the bridge

The increasing spacing of the piers towards the centre creates good, varying proportions (Fig. 2). The wider higher spans keep a more uniform rectangular shape. From a distance this spacing may appear equal, which causes confusion as to why a deeper truss is needed; however upon inspection the varying span lengths become clear.

The original truss structure (Fig. 3) appears to have a very deep section, which looks too high for the crossing it is as well as appearing heavy and cumbersome. With the addition of the side sections (Fig. 4) the truss is obscured much more, this forms a wider bridge which has more appealing dimensions.

The truss beneath appears deep to support the slender looking deck in comparison. The image of this deep truss is somewhat improved by the ability to partially see through it, making it appear less onerous.

Figure 3: Original structure [2]

2.3 Order within the structure

The large number of elements used to create the truss may seem too many however the repetition of these throughout the truss reduces the apparent number of elements, despite their differing sizes. The order is somewhat interrupted when viewing the truss from certain oblique angles where the truss elements merge into a criss-crossing confusing image (Fig. 4). The addition of the clip-ons has improved the order by providing a continuous line along the entire length of the bridge, this draws attention from the many truss elements.

Figure 4: Truss elements at oblique angle

2.4 Refinement of design

With the addition of the extra lanes, the edge view on the main span of the deck has a wider section over the supports which creates a curve along the span (Fig. 1). The addition of dummy members [1] over the pinned connections has provided a cleaner appearance to the bridge, but it is a disappointment to hide the true structure of the bridge from view.

2.5 Integration into the environment

The large bridge structure impedes the beautiful view across the harbour. The scenery beyond is almost entirely blocked from certain angles except for the high navigation span where one can catch a 'snapshot' of the view beyond.

The long and high navigation span which is located to the north of the harbour is considered as an unsuitable place in terms of aesthetics, symmetry and the view beyond. It is unfortunate since this location was needed to allow the required water depth and headroom

2.6 Surface texture

The common understanding of using rough finishes for piers and a smooth finish for the deck has not been met with the bridge. The smooth concrete finish of the piers which are large elements draw some attention from the line of the truss and put the dark underside of the bridge in further display. This detrimental visual effect is somewhat relieved by the additional lanes added; the box sections attract the eye to their smooth line and draw interest along them while hiding the piers.

Considerable care took place during construction to get the concrete quality in the piers to a high standard of uniform colour [2], this has been successful and prevented the common sight of poor quality and stained undersides of bridges (Fig.1).

2.7 Colour of components

The colour of the bridge is a light grey colour, which appears a little 'dull' at many times of the day. The interesting shape and design of the truss could have been highlighted in a bright colour in a similar way to the Forth Rail Bridge. However, at certain times of the day, the play of shadows and sunlight create a varying and interesting effect on the colour of the bridge. The illumination of the truss from the lighting creates interesting patterns on the roadway as well as illuminating the piers and guiding the eye under and through the bridge (Fig. 5).

Figure 5: Auckland Harbour Bridge at night

2.8 Character

The position of the truss creates a certain character in the bridge. This switching of position from under to over the deck for the main span makes the truss stand out and allows the eye to follow it along the entire length (Fig. 5). This is further accentuated by the curved edge to the steel box sections as mentioned in 2.6.

The approach to the bridge is from a curved road which allows the driver to see the bridge they will be crossing. This makes the bridge much more noticeable and interesting, allowing the driver to appreciate its character.

2.9 Complexity in variety

The main complexity of the structure is that of the truss, which looks complicated but with a clear function. The truss creates interest while remaining simple enough from the repetition of the elements. The truss switching from under the deck to over adds interest to the previously continuous and mundane repetition. The addition of the clip-ons adds further interest and variety raising the question why they are of completely different construction.

2.10 Integration into nature

The crossing is in the centre of a city across the busy harbour. For these reasons, the bridge materials and components are suitable for the surroundings and it is not trying to be something it is not. The bridge is not a grand statement of its surroundings, nor is it an unsightly part of the landscape.

2.11 Summary

The bridge is perhaps not the most ideal type for the large harbour crossing; however the designers have achieved an interesting and memorable bridge from the truss design, which is loved by so many local residents. Its original appearance had several elements which made is appear unsightly, yet as mentioned in this section these have been successfully disguised by the clip-on additions, helping to make the current structure visually appealing. Whether this was an intentional visual improvement or just good fortune is unknown.

3 Structural Design

The road surface of the bridge is supported by the steel truss superstructure. The truss is a 'deck truss' for its entire length except for the main span which is a 'through truss' with the traffic travelling through the structure. The truss throughout the whole structure is a subdivided warren truss as shown in Fig.6. The steel truss is supported on 6 reinforced concrete piers which are founded on the bedrock. The balance between the cost of the superstructure and number of piers decided the number of spans needed, which is seven of varying length due to the geology and navigation access. Table 1 shows the spans and their lengths with reference to Fig. 6.

Table 1: Span lengths

Span Lengths							
Label	A	B	C	D	E	F	G
Span	176	244	177	124	114	103	80

The original truss bridge is 4 lanes wide of width 12.8 metres and deck construction of steel and concrete as shown in Fig. 7. The 6 steel stringers support the 170mm reinforced concrete deck and work compositely together due the bonding and overlapping of the steel and concrete. The deck is a series of simply supported sections to prevent any bending of the cross girders in the truss [1].

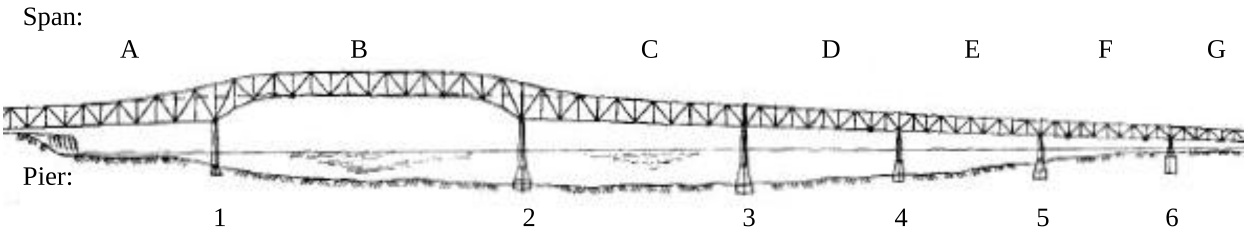

Figure 6: Elevation of bridge [1]

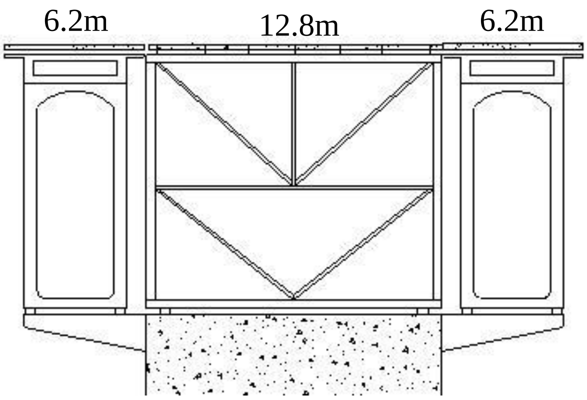

Figure 7: Current bridge cross section

3.1 Earthquake Design

Despite the area having the lowest earthquake activity in New Zealand [5], the structure has to withstand a moderately high earthquake. This led to the structure having all directions braced and being made of elastic material. Large movement should be allowed in the bearings while the bridge remains well anchored. On-shore anchorage points provide this and prevent lateral forces acting on the piers. Further earthquake resistance is provided from the superstructure being divided into two sections at pier 3.

3.2 Connections

The superstructure is resting on rockers on the piers, creating a pinned connection to prevent lateral loads and allowing these to be taken by the anchorage points of fixed connection at either end of the bridge. Pier 3 has large steel toothed expansion joints to allow movement between the two sides of the bridge. This is only expansion joint in the entire truss length.

3.3 Piers and Foundations

The essential criterion of the piers was to withstand a severe earthquake by remaining fully fixed to the bedrock and allow pinned connections of the superstructure. The height of the piers ranges from 16.8 to 31.7 metres and are constructed of high density reinforced concrete on the outside and low density material in the centre. The slender piers of 3.3m width have large bases of 14m to spread to forces over a wide area.

Full width piers were used for visual appearance as well as allowing the large bearings required in the design. The bearings could then be fixed using hidden sway frames in the hollow piers [1]. The complicated rocker bearings were required to allow for large lateral movements and high forces in the event of an earthquake. The longevity of these was critical to ensure the expected behaviour in an earthquake. The high quality concrete used was needed to protect the structure from scouring due to the high tidal currents in the area.

The varying ground strength allows the differing spans and pier depths. These all go at least 1.8 metres into the bedrock which limits the bearing pressure to 750KN/m^2 under normal loading and 1500KN/m^2 under combined earthquake and wind loading. Little settlement has occurred with the exception of pier 2 which as a settlement of 25mm. The effect of this is shown in section 8.3.

4 Geology

The harbour has bedrock of slate and sandstone at approximately 33 metres below mean sea level. The bridge site has up to 15 metres of sediment above the bedrock except a few locations where the bedrock is exposed due to scouring. The bedrock provides a pile end capacity in the range of 6-16MPa. Borehole data used for design shows the bedrock has a moderately uniform strength with occasional lenses of stronger material, tests during construction showed this to be slightly higher than anticipated [1].

The sediment was not used structurally due its very poor strength, and issues of liquefaction in the event of an earthquake.

5. Extensions

Soon after the bridge was opened, there was a requirement for further capacity of up to 100 percent. The increase in traffic in 10 years from opening of 13,000 to 45,000 vehicles per day [6] shows how clear this need was. Initially, this was not possible so a duplicate bridge was planned. The issue was that the new bridge would need to be beside the existing one for cost reasons and being the best route, which many argued would spoil the appearance of the Auckland Harbour Bridge. This led to a more in depth enquiry into the capacity of the existing structure. It was found the piers could carry a slightly increased load due to an overly conservative wind loading design, so light extensions were designed.

A system of cantilevered sections from the existing piers was needed to remain entirely separate from the truss structure, evident by the visible gap. The extension was completed in 1969 using a system known as ‘Nippon Ties’ [6]. This system exerts no extra forces to the superstructure itself since the load is transferred directly onto the piers. The new sections were hollow steel box sections prefabricated in Japan and were placed onto steel cantilever extensions of the piers (Fig. 8).

Figure 8: Pier extensions

The steel hollow box sections were a modern construction technique which allowed the lightweight design along with other weight saving techniques such as the thin surface laid onto the flange. Cantilevered construction was used for this by lifting sections into place (Fig. 9).

The entire section is continuous over the 1020m length, supported on rockers which allow longitudinal movement, and have a single expansion joint at one end. Transverse movements in the event of an earthquake are transferred to the piers, however this is well within the required limits.

Figure 9: Steel box sections during construction

6. Loading

Despite the bridge designers using the American AASHTO codes for live loading design, BS5400 will be used for this analysis since other British Standards such as BS153 were also used in the design and to allow comparison between the two. The design must check ULS and SLS of all elements.

For the purpose of the calculations following, the original four lane truss structure will be analysed to assess the methods used in the main design since the analysis will provide more information than the structurally independent steel box sections.

The design factors γ_{fl} and γ_{f3} are used for partial load and inaccuracy factors where γ_{f3} is taken as 1,10 for steel bridges and γ_{fl} values are shown in table 2.

Table 2: Design factors for ULS [4]

Component		γ_{fl}
w_d	Dead Load – Steel	1.05
w_d	Dead Load - Concrete	1.15
w_{sd}	Superimposed Dead Load	1.75
H_A	H_A Loading	1.5
H_B	H_B Loading or $H_A + H_B$	1.3

6.1 Dead and Superimposed Dead Loads

The loadings to be used for the calculations which follow are outlined in Table 4. These are based on the loadings as described in BS5400 with densities in Table 3.

Table 3: Material densities

Material	Density (kg/m^3)
Concrete	2400
Steel	7850
Asphalt	2300

Table 4: Dead weight loadings

Component	Load (KN/m)
Concrete deck (w_d)	144.4
Steel truss (w_d)	159.7
Superimposed Dead (w_{sd})	4.6

6.2 Vehicle Live Loading

The original four lane bridge has a carriageway width of 6.4m so has 2 notional lanes per carriageway to design for. The bridge will be assessed as a continuous section and the worst hogging and sagging cases chosen which will result in being for spans A-C.

The maximum sagging case will be with full loading on span B with unfactored, dead load on all other spans. The associated H_B loading would be a 6m heavy vehicle in the centre of span B. (Fig. 10)

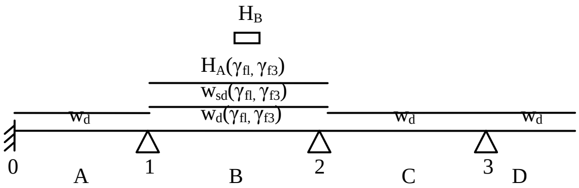

Figure 10: Max sagging case

The maximum hogging case will be with maximum load on spans B and C with unfactored, dead load on all other spans to produce the hogging over support 2. The associated H_B loading would be a 26m heavy vehicle over support 2 (Fig. 11).

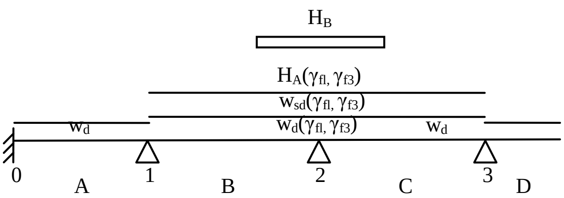

Figure 11: Max hogging case

The H_A loading to be used varies with the length of the span which equates to loads 20.8KN/m and 27.1KN/m per lane for spans B and C respectively, Eq. (1). A KEL of 120KN is to be applied at the centre of the span to assess the worst bending, and close to the support to assess the worst shear.

$$W = 36\left(\frac{1}{L}\right)^{0.1} \quad (1)$$

The largest loading on the outer lane will cause the worst torsional effects which is where H_B loading will be, with H_A loading over the first two lanes and 1/3 H_A over the remaining two (Fig. 12). Consideration must also take place for an exceptionally large vehicle straddling two lanes

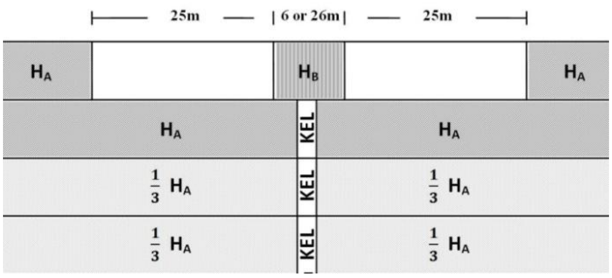

Figure 12: Loading combination

6.3 Braking and Acceleration Forces

BS5400 states the most severe of 250KN plus 8KN per metre length or 25% of the total H_B force. The load used for design following the AASHTO codes was 5% of the total live loading [1]. This equates to loads of 8410KN and 6120KN for BS5400 and AASHTO codes respectively, which shows a strong relation to one another considering the variations in loadings between the two codes.

6.3 Wind

Frequent wind gusts of over 30ms^{-1} led to the designers using a wind pressure of 1.46KN/m^2 when acting alone, and 1KN/m^2 when acting in an earthquake [1]. For wind alone, this results in a load per metre (based on the truss area) of 6KN/m.

A short calculation into BS5400 wind loading shows a similar result. Calculating the wind gust from Eq. (2) using a 30m height gives K_1 of 1.40, S_1 of 1.10 due to funnelling, S_2 of 1.21 for the gust factor and a mean hourly speed of 22ms^{-1}shows the gust speed, v_c to be 40.1ms^{-1}.

$$v_c = vK_1S_1S_2 \tag{2}$$

The dynamic pressure head, q is calculated as 1.03KN/m^2 using Eq. (3). This can then be put into Eq. (4) to calculate the horizontal wind load acting on the bridge.

$$q = 0.613{v_c}^2 \tag{3}$$

$$P_t = qA_1C_D \tag{4}$$

The C_D value found to be 1.8 [4] is related to the solidity ratio of the truss, taken as 0.2. Both trusses are exposed to the wind however the rear truss is partially shielded, so the C_D ratio for the second truss will be multiplied by the factor η, taken as 0.95 for a truss of large spacing [4]. Using Eq. (4) to form Eq. (5) gives a horizontal wind load, P_t simplified to act over each metre length of 8.3KN/m. This has a clear relation to the designers method, which is likely to have consisted in a more detailed site analysis preventing the perhaps overly conservative value of 8.3KN/m

$$P_t = qA_1C_D(1 + \eta) \tag{5}$$

Other wind loading effects to consider will be loading of piers and longitudinal wind on the bridge and traffic (P_{LS} and P_{LL}) as well as the vertical force. The vertical force is considered below using Eq. (6).

$$P_v = qA_3C_L \tag{6}$$

With q as 1.03KN/m^2 from Eq. (4), A_3 being taken as 13056m^2, and C_L being 0.75 [3], the vertical force, P_v acting over the entire bridge is 10085.8KN, which can be simplified to 9.9KN per metre length for analysis.

6.4 Earthquake

The designers used the live load and 10% of the superstructure self weight in any direction as the forces for earthquake design of the superstructure. The design of the foundations included the 10% weight above plus the foundation and virtual mass to take into consideration the inertia of the mud and water surrounding it [1].

The volume of the virtual mass to be used is a cylinder whose height is that of the pier, and of diameter of the dimension of the pier which side is under consideration. This is shown diagrammatically in Fig. 13 where the solid rectangle represents the pier and the shaded area indicates the area of virtual mass to consider for the direction of movement demonstrated by the arrow.

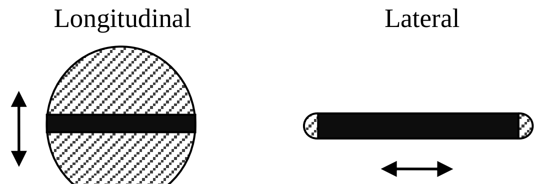

Figure 13: Virtual mass area to consider

6.5 Temperature Effects

Assessment must take place to consider the effects if the expansion joints and bearings do not perform as designed.

The deck and the 6 I-beams acting compositely with the concrete slab will be investigated for this calculation, ignoring any effects of the truss. Assuming the whole deck rises to 15^{o}C above the datum, using α from Table 5, the expansion needed in this joint is 183.6mm, Eq. (7)

$$e = \propto TL \tag{7}$$

Assuming the only expansion joint (at pier 3) is not functioning, the possible stresses can be calculated over the 1020m span. Using the values in Table 5, the axial compressive force will be calculated for a temperature variation where the top surface is 15°C above datum temperature and the bottom is 5°C below.

Table 5: Values for temperature calculations

Component	Value
E_C	30KN/m^2
E_S	200KN/mm^2
α	12 x 10^{-6} / °C

This leads to a strain and stress variation as shown in Fig. 14 with the simplification of the steel not overlapping with the concrete and assuming the simplified linear distribution. This equates to the compression forces in the deck shown in Table 6.

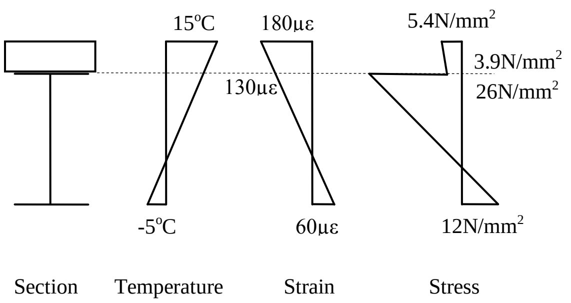

Figure 14: Temperature distribution

Table 6: Compression forces

Component	Compression Force
Concrete (F_c)	10.1 MN
Steel (F_s)	0.6 MN
Total	10.7 MN

These forces would be taken in the end anchorages if the bearings prevent this force being passed through to any of the six piers. This shows the huge effect temperature variation has on stresses.

6.8 Parapet Design

The parapets are 0.9m high of tubular steel which are bolted to 0.3m high kerbs. These kerbs are fully fixed and integral to the slab[1].

Assumptions of a 40 tonne truck travelling at the maximum speed limit of 40mph will be used.

This speed limit equates to 18ms^{-1} along the bridge, so a perpendicular velocity of 5 ms^{-1} will be used and the assumptions that 85% of the energy is absorbed in the trucks crumple zones which occurs over a time of 0.2 seconds.

$$F\Delta t = 0.15\, m\Delta v \tag{8}$$

$$v = u + at \tag{9}$$

$$s = ut + \frac{1}{2}at^2 \tag{10}$$

Eq. (8) shows FΔt to be 30,000kgms^{-1} which gives a force of 300KN. The acceleration is found to be 25ms^{-2}, Eq. (9) which gives a deflection of 0.75m, Eq. (10). This seems a very large deflection needed, however a fully laden truck going at the maximum speed limit is unlikely, and the assumptions used are likely to give a slightly exaggerated deflection.

6.9 Moveable Barrier

The moveable barrier plays a critical role in the running of the bridge. This is a tidal system which is used to separate opposite directions of traffic at the daily peak hours.

Previous collisions with the barrier have shown it moving 1m with car collisions [7]. Assuming a 1500kg mass moving at the above described speeds with s taken as 1, this gives t = 0.4 seconds equating to a force of 3.75KN. This shows how the design of the barrier to allow movement in the collision successfully absorbs the forces from this action.

6.10 Load Combinations

The load combinations used in the original design differ slightly from those outlined in BS5400 to allow for other local effects. The combinations which were considered are: [1]

1. Dead, live and impact loads
2. Temperature, longitudinal wind, bearing resistance plus combination 1
3. Earthquake loading, plus a limited loading from combination 2
4. Erection loading

7. Strength

7.1 Design Loading

The load combination in 6.2 for maximum sagging in span B has been used with a load of 328.5KN/m over span B and unfactored, dead weight elsewhere. Using moment distribution and fixed end moments (FEMs) from Eqn. (11) to moment diagram in Fig. 15 has been calculated using the knowledge that end 0 is fully fixed and end 3 is assumed free.

$$FEM = \frac{wl^2}{12} \tag{11}$$

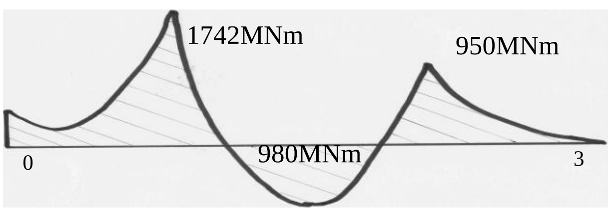

Figure 15: Bending moment diagram

By resolving the truss the moment equates to an axial force in the upper chord of 92.4MN.

Using Eq. (12) and assuming σ = 460N/mm^2 requires a cross sectional area of 200.8 x 10^3 mm^2 which equates to a square section of 840mm by 840mm with plate thickness of 60mm.

$$\sigma = \frac{F}{A} \tag{12}$$

7.2 Erection

During launch, the cantilevered sections would have caused large hogging moments over the supports. A short analysis of this is shown below, assuming a case where the truss of span A has been fully erected, and half the truss of span B is cantilevered from support 1. This assumes a UDL of 157.9KN/m (Table 3) which, solved by statics gives vertical reactions of 7.3MN and 40.3MN at piers 1 and 2 respectively, resulting in a bending moment in the truss of 1.19 x 10^{6}KNm over pier 2. This is comparable to the case of hogging in section 7.1 which indicates how important construction loads are.

7.2 Parapet

This loading of 300KN from section 6.8, Eq. (8) will be assumed to be acting at the top of the 0.9m parapets which will exert a moment of 270KNm onto the member with an 'I value' of 173.3 x 10^6 mm^4, Eq. (13).

$$I = \frac{\pi}{4}({r_1}^4 - {r_2}^4) \tag{13}$$

Calculating the bending stiffness shows a 300mm diameter sections with wall thickness 20mm would provide the moment resistance.

$$\sigma = \frac{My}{I} \tag{14}$$

Assuming σ of 460N/mm^2 and using Eqn. (14) shows the member bending capacity of 531KNm.

8. Serviceability

8.1 Deflection

The truss and bridge deck were pre-cambered by the amount of deflection expected under dead loading by adjusting the length of the truss members. This did not take into account the deflection under live loadings so some deflection can be expected under heavy loading.

Assuming the truss as a beam with E value 200KN/mm^2 (Table 5) and I-value 6.7 x 10^{12} mm^4, Eqn. (15) shows the deflection in span B to be 442mm under full live loading.

$$\delta = \frac{wl^4}{384EI} \tag{15}$$

8.2 Frequency

Although the bridge has no pedestrian access, the 1975 demonstration of over 2000 protestors clearly showed the need for careful consideration into the effects of large crowds moving across structures. The main issue was with the steel box section extensions which experienced lateral vibration of 0.7Hz [5]. Analysis was not carried out for the extensions, however a approximate calculation using the Rayleigh Ritz method will be performed to show these effects.

$$w_n = f_o = (\beta_n l)^2 \sqrt{\frac{EI}{ml^4}} \tag{16}$$

Using a fixed-pin case and β_n value of 15.42, I value 3.88 x 10^6 mm^4, E value of 200KN/mm^2 (Table 5) and mass 6310kg/m a natural frequency of 0.16Hz is obtained, Eq. (16). This is an approximate calculation to demonstrate the susceptibility of the extensions due to their natural frequency and slender design and shows how a simple calculation if known at the time would have identified this, if it was thought to be an issue. Further calculations can be carried out to determine the vertical acceleration, Eq. (17).

$$a = 4\pi^2 {f_o}^2 y_s K\varphi \tag{17}$$

8.3 Support Settlement

The settlement of pier 2 by 25mm will be analysed to find the additional moments set up as a result. This will be calculated using moment distribution and the relevant fixed end moment (FEM) formulas. By treating the truss as a beam with an I value 6.7 x 10^{12} mm^4, E value 200KN/mm^2 (Table 5) and assuming no continuity over pier 3 due to the expansion joint and for earthquake resistance, where support 0 is fully fixed at the end (Fig. 16). The FEMs due to settlement are shown in Eqn. (18)

$$FEM = -\frac{6EI\delta}{l^2} \tag{18}$$

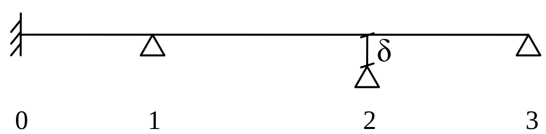

Figure 16: Settlement at pier 2

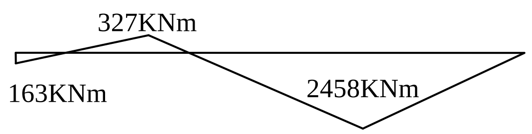

Figure 17: Settlement bending moment diagram

Following the moment distribution method gives the bending moment in Fig. 17 due to the settlement alone, resulting in a 2458KNm additional moment in the truss over pier 2. This will act to reduce the hogging moment under normal loading, but this shows the additional moment which will be exerted over pier 1 and at fixed end 0.

9. Construction

All construction work was carried out from moving materials and large sections into position from barges. This was a huge operation of high risk because of the large, heavy sections to be transported.

9.1 Piers

The construction of the huge concrete piers was with the use of steel caissons which were floated into position, and sunk using compressed air at pressures of up to 345KN/m^2 [1]. This allowed the visual inspection of the supporting rock to check for faults which were likely to be found. The original design of the piers had to be altered to allow for this method of construction and sinking into position.

9.2 Trusses

The trusses were constructed on-shore, launched and brought into position by barge. Once in position, floating cranes, temporary supports and the removal of ballast from barges could be used to position the section. Work was completed using cantilever and falsework erection. The design of pinned connections between the spans created a problem for the cantilever construction so "dummy" members were added over the pins only to be used during the construction phase.

Figure 18: Transportation of trusses

Difficult conditions were met during construction which almost caused the loss of a large truss section as it got dragged with the current. This shows the dangers in delivering large sections across the water and is an example of why this method is seldom used today.

Figure 19: Erection of trusses

A simpler and safer approach to construction would have been to launch the structure from the large reclaimed land at the bridge abutments. The straight bridge would allow this method of construction but extreme accuracy would be needed to keep the bridge in line, and the cross section variations in the design would have caused large problems. The moments experienced during construction may be well above the moments to be experienced during operation.

10. Alterations and Durabilty

10.1 Pedestrian Access

Since the design stages of the bridge, the access for pedestrians and cyclists has been an issue, and has been rejected on many occasions on cost grounds. There is still no route across the bridge without a vehicle which is currently an issue high on the agenda. Proposals have been put forward to cantilever another section past the 'Nippon ties' for walking and cycling (Fig. 20). As mentioned above, engineers are struggling to maintain the existing, possibly overloaded cantilevered sections, so these extensions are a controversial subject with protests still being planned. Transit New Zealand conducted a feasibility study into this, and it is likely the cantilevered sections will be possible. An option less likely to pose structural issues would be to allow access under the bridge, this would not be such a good experience however it would give the option of walking and cycling.

Figure 20: Proposed walkway

10.2 Lighting

Lighting on the bridge is bright to ensure a safe route however this was too bright for ships below the bridge meaning covers had to be added to prevent the spread from the roadway.

Navigation lighting and the illumination of the piers was then added to prevent accident under the bridge. Further lighting on the main span was added for aviation safety.

10.3 Maintenance

The salty environment of the structure poses a problem to the maintenance of the steel to prevent any corrosion. Zinc coating was applied as a sacrificial element to the steel. However, later it was discovered that the zinc was corroding too quickly due to the salty and humid atmosphere, and required its own protection however conditions at this time were particularly harsh.

10.4 Nippons

The 'Nippon' extensions have been discovered to be cracking and an investigation into their future is ongoing. Temporary measures have seen the removal of heavy vehicles over 13 tonnes from the outer lanes, but recent work has seen the strengthening of the sections, with mixed success. Further weight reduction techniques have been used with the testing of new, lighter surfacing materials to reduce the dead weight of the structure. Structural engineers at a large consultancy have noted that the clip-ons are at risk of catastrophic collapse under full H_A loading. This is unlikely, and preventative management measures currently ensure this does not happen.

The result of the frequency shown in 8.2 raises the question of the effect of this on the structure and what issues of fatigue could be expected.

10.5 Strengthening and seismic retrofit

In 2006 signs of fatigue and cracking were discovered in the clip-ons which resulting in a £17 million strengthening process starting in 2007. This work and another seismic refit in 1999 has added a large amount of bracing and further strengthening to the structure. The process involved further strengthening of the truss, added prestressing and reinforcement of the piers to resist further wind and seismic loading [9]. Additional methods which could be used are stiffeners and the strengthening of the diaphragms [8].

10.6 Second Crossing

With the population in the region set to rise a further 1.2 million by 2050, [7] a second crossing is likely to be needed. Looking at local maps and areas of high population it can be seen there is the option to locate a second bridge between 1 km and 10kms along the harbour. A feasibility study is also underway into a location just 500m from the existing structure [5]. If, upon further investigation this is not possible then the second crossing could be provided by means of a tunnel under the harbour.

11 Conclusion

Despite the several durability concerns outlined in sections 9 and 10, the Auckland Harbour Bridge should continue to be a well loved and well used structure in Auckland. The long challenge faced in allowing and financing such a structure makes it an even more appreciable feat.

A more striking bridge could have been achieved structurally, however if this was a requirement, a bridge may not have been constructed at all due to lack of money which would have stunted the growth of the North Shore and Auckland itself.

With the current strengthening and possibility of a second crossing, the Auckland Harbour Bridge should have less demand put onto it allowing it to be an Auckland landmark for years to come.

12 References

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