A CRITICAL ANALYSIS OF THE BAYONNE BRIDGE, NEW JERSEY

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Abstract: This paper contains a detailed study and analysis of the Bayonne Bridge over the Kill Van Kull, including an aesthetic study based on Fritz Leonhardt’s aesthetic criteria, simplified structural analysis to British Standard 5400, and a consideration of the measure of success of Othmar Ammann in the design and construction of this bridge. An account is given of the design, construction, history and possible future of this highly accredited bridge.

Keywords: Bayonne Bridge, arch, steel truss, spandrel.

1 Introduction

The Bayonne Bridge is a steel trussed-arch bridge, designed by Othmar Ammann and opened on 15 November 1931, spanning the Kill Van Kull channel from Bayonne, New Jersey to Staten Island, New York. At the time of its completion it was the longest steel arch bridge in the world, with a central span of 510m [1] – 7m longer than the Sydney Harbour Bridge, which was completed the year after – a title which it held until the completion of the New River Gorge Bridge in 1977.

Despite being longer than its Australian cousin, the Bayonne Bridge has never received the same level of public recognition as the Sydney Harbour Bridge, and was generally overshadowed by the completion of another of Ammann’s creations; the longer and taller George Washington Bridge, which opened less than a month later. Although it may have failed to standout in the public eye against the surrounding New York monuments, the bridge was immediately recognized among academic circles as a work of structural art, and awarded the “most beautiful steel bridge” prize by the American Institute of Steel Construction in the year of its completion [2]. It was later designated a National Historic Civil Engineering Landmark in 1985 by the American Society of Civil Engineers [3].

Bayonne Bridge was only the second bridge that Ammann had been chief engineer of, after the George Washington Bridge, which was still under construction at the time, and although he may have been an unlikely choice for the Port Authority, Ammann had previously worked under Gustav Lindenthal on the Hell Gate Bridge, and brought with him to the new project a wealth of inspiration from this design, and a desire to improve on it both structurally and aesthetically [2].

The appointment was hugely successful, with the bridge being brought in ahead of time and under budget, and Ammann went on to become a prolific bridge builder in the New York area, and the Master Bridge Builder of the Port Authority [4].

2 Aesthetics

Ammann was heavily influenced by the design of the Hell Gate Bridge, which he had worked on previously with his mentor, Lindenthal. This influence is apparent in the similar designs of the two bridges (see Figures 1, 2), but Ammann brought his own views on the aesthetic quality of bridges to the design of the Bayonne Bridge, views which differed from Lindenthal in certain respects.

One difference that is instantly recognisable between the two designs is Bayonne Bridge’s lack of stone clad piers. Ammann’s original design did in fact include stone cladding for the piers (although they were never on as grand a scale as those of Hell Gate Bridge and the Sydney Harbour Bridge) but the stone
cladding befell the same fate as that of the George Washington’s; due to the great depression, costs were cut wherever possible, which meant that the stone, which served no structural purpose, never came to be.

Figure 2: Hell Gate Bridge, New York

Lindenthal believed a bridge should appear stiff and monolithic – demonstrating its ability to span the distance with strength and stability. Ammann was more committed to structural efficiency, and therefore went for a more slender design, avoiding the counter-curvature at the ends of the upper chord and large ornamental piers, as they served little structural purpose (see Figure 2). This stoic commitment to efficiency and structural form may have been a mixed blessing, because although the bridge was structurally efficient and cost-effective, it has never enjoyed the same recognition as its more flamboyant contemporaries.

2.1. Fritz Leonhardt’s Analysis

There are no set rules as to what constitutes a beautiful bridge, but Fritz Leonhardt, one of the most recognized and respected bridge engineers of the 20th century, proposed 10 areas that make up a bridge’s aesthetics. A brief consideration of each of these can afford us an idea of the measure of success of the aesthetic quality of the bridge.

2.1.1 Fulfilment of function

The bridge clearly demonstrates its function in the striking and strong form of the trussed arch. It is clear to the casual observer that the arch is the dominant structural form, carrying the slim deck below. Neither does the bridge lie, by placing large piers at each end which do not offer the structural support that they suggest. The arch in fact appears to have very little support at its ends, with only the lower chord supported by abutments (see Figure 3). This demonstrates that the lower chord members are the main load carrying members. The overall form of the bridge is simple, but the elements themselves are relatively complex.

2.1.2 Proportions

The bridge is generally very beautifully proportioned, with great care and attention given by Ammann to the shape and dimensions of the arch. The lack of stone piers means that the strong form of the arch is seemingly not matched by the abutments, which means the arch appears to be uncomfortably disappearing into thin air (see Figure 3). Perhaps if the piers were present the bridge would appear better balanced. On the other hand, perhaps, since there is no structural benefit to the spandrel form of the arch, a crescent form would have had better balance, and been more appropriate to Ammann’s desire for structural efficiency (see Figure 5).

People are used to such grand arches being the focal point of the surrounding area, such as with the Sydney Harbour Bridge. At the time of its completion, the Bayonne Bridge was the longest arch span in the world, and would have been a grand sight for the ships passing underneath towards the ports upstream, but in the grander scheme of the New York and New Jersey area, it does not stand out as a significant monument. In that sense the arch seems too grand a gesture. Over time I believe some of its grandeur may have been lost.

2.1.3 Order

The trussed arch cuts a clear parabolic curve over the Kill Van Kull. Ammann was hugely successful in creating a pleasing pattern of regular triangles through the articulation of the arch into 40 segments [1].

The pure form of the arch is unbroken along its entire length, and the grid of the truss forms a clean order. On closer inspection the discovery that many of the members are themselves trusses gives the bridge an almost fractal quality (see Figure 4).

Figure 3: View of arch descending to abutment
2.1.4 Refinement of design

Ammann experimented with various forms for the design of the Bayonne Bridge, including the cantilever truss, suspension and arch forms. He eventually decided upon the arch because he determined it would be the most economical and structurally sound, but it is known that he also considered the arch to be the aesthetically superior form for the low-lying industrial landscape it would visually dominate.\(^2\)

He considered both a crescent arch and a spandrel arch form (see Figure 5), and was influenced by the Hell Gate Bridge in his choice of spandrel form, as can be seen from his writing on that bridge regarding the same design process: “Although both designs are pleasing in appearance, the spandrel arch, owing to its height increasing from the center toward the ends, is more expressive of rigidity than the crescent arch, the ends of which appear to be unnaturally slim in comparison with the great height at the center.”\(^6\)

Figure 4: View inside the arch, showing truss elements

2.1.5 Integration into the environment

As previously noted, Ammann considered the steel arch design to be best choice to fit in with the surrounding industrial area. Unlike Manhattan, the areas of Bayonne and Staten Island near the bridge are relatively low lying, and so the arch has a lower curvature than that of the Sydney Harbour Bridge, giving it a lower central height.

The steel industry in New York was thriving around the time of the bridge’s construction, and so building with steel stood as a great demonstration of the possibilities of the material, and also responded to local expertise and manufacturing capabilities.

2.1.6 Texture, colour, character

The bridge’s coarse texture, dull matt grey paint and exposed bolts and rivets, which show honestly how the bridge was put together. Its character is in keeping with its industrial surroundings.

The character of a bridge is very much one’s own opinion, but I believe the Bayonne Bridge has a subdued character, which sits happily in its surroundings, and doesn’t try to garner attention and praise from a wider audience.

2.1.7 Complexity in variety

The overall pure an simple form of the arch is contradicted beautifully with the intricately trussed members that make up the structure.

2.1.8 Incorporation of nature

Bayonne Bridge is if anything the opposite of a natural form. It is the culmination of the industrial revolution, and man’s urge to build bigger and longer. It is proud to be industrial, and doesn’t try to hide it. New York is entirely a manmade environment, so this approach fits in its location perfectly.

Figure 5: Sketches of Ammann’s design as built, and if designed as a crescent

2.1.9 Summary

This analysis has shown that whilst the Bayonne Bridge is not above criticism, it is an accomplished piece of structural art, very highly regarded within the bridge engineering industry – as shown by its recognition by multiple prestigious awards – along with its maker, Othmar Ammann who, off the back of the success of his two first bridges, the Bayonne and George Washington Bridges, went on to become a major figure in the engineering profession\(^4\).

3 Structural Design

All of the sizes quoted in this report are approximate unless otherwise stated, estimated from various sources including photography and comparison with other similar structures.

Bayonne Bridge is a two-hinged spandrel-braced through-arch bridge\(^5\), so called because the road deck travels through both chords of the main arch. It was the first long span bridge to be built of manganese steel, which was chosen due to its high strength and cheap price compared to nickel steel\(^7\). Its main arch is made up of 40 linear segments, which vary in height from 11.4m at the centre to 20.6m at each end\(^2\). The lower chord is the main load carrying element of the structure, with the upper chord subject to 33% of the axial force in the lower chord, on average\(^2\). The upper and lower chord members are riveted hollow box girders, 1.6m deep\(^8\), and the vertical and diagonal truss members are square riveted trussed box members.
The deck is suspended from steel cables hung from the lower chord of the arch in groups of four, at approximately 12.5m centres. The load is transferred from the deck to the cables by a system of secondary cross beams, stringers and girders riveted together (see Figure 7) [9]. It has been assumed that the asphalt and 200mm concrete road deck sits on a corrugated steel floor plate, which is the method of construction used in the Sydney Harbour Bridge. This is supported by the secondary cross members, 500mm deep and at 1m centres. These cross members sit on 1.5m I-beam stringers which run longitudinally the length of the span. These are riveted to the 3m main riveted girder cross beams, which are attached at either end to the bridge cables which suspend from the arch above.

The bridge carries four lanes of traffic – though there is no central reservation – with space for two more lanes of road or light rail. A pedestrian footpath is cantilevered off each side of the bridge (see Figure 7). The level of traffic over the bridge has never been as high as the surrounding bridges and therefore the planned expansions never took place [9].

4 Construction

![Figure 6: Bayonne Bridge under construction, showing falsework.](image)

The prevailing method of construction for arch bridges at the time was cantilevered construction, where the base at each end is secured in place and cantilevered over the span until the two meet in the middle. This requires expensive, substantial anchorages and ties to secure the arch during construction. Ammann was able to save costs by utilising the bedrock found in the channel to place falsework on, which supported the arches during construction (See Figure 6). This enabled him to do away with large piers, which he saw as unnecessary, and he saved further cost by reusing the material of the falsework in the eventual bridge structure [2]. Falsework had never before been used on a project of this scale.

![Figure 8: The 3-hinge phase of construction of Bayonne Bridge, showing asymmetrical falsework](image)

Because of the importance of the Kill Van Kull as a shipping channel, the Port Authority required that it be kept open for the duration of the construction period. Ammann’s original symmetrical construction plan was altered to have the two arches meeting to the south end of the bridge, thereby allowing the falsework to avoid the main shipping channel (see Figure 8).

Once the arch had been closed, one member of the upper chord was left out at the point of closure, forming a 3-pin arch (see Figure 8). The falsework was then removed, with the bridge being lowered into its final position, and the final member was added, resulting in the finished 2-pin arch. The steel cables were the hung in place, and the road deck craned up in sections from barges and attached, starting with the central section.

The entire structure was bolted and riveted together, the higher strength of the manganese steel allowing fewer rivets than would otherwise have been
required. The strength of the carbon-manganese steel used in the bridge was 193.1N/mm² [2]. This method of construction proceeded faster than was expected, and the project completed many months ahead of schedule and $3 million under budget [10].

5 Foundations and Geotechnics

The New Jersey Palisades run under the Bayonne Peninsula and under Staten Island, therefore the whole run of the bridge is above solid bedrock, providing excellent foundation conditions, particularly suitable for withstanding the lateral loads exerted on foundations by arch structures. Abutments were constructed on top of the bedrock to transfer the loads from the lower chord of the arch.

Because of the shallow depth to bedrock, double-wall cofferdams were built for the abutments, with the double sheet-piles being driven first, then the space between them excavated, braced and refilled with clay and gravel [8].

Figure 9: Diagram showing the arch base connection at the abutment

A steel frame was cast into the abutment, and the main structural shoe of the arch was riveted to it. The end member of the lower chord then connected to the steel shoe by means of a 16" pin, creating a joint which allowed rotation in the vertical plane (see Figure 9) [8].

During the 1970’s remedial repairs were carried out on the north abutment, where one inch of shotcrete was installed to encase it, but cracks propagated out on the north abutment, where one inch of shotcrete was installed to encase it, but cracks propagated to the point loads generated through the shotcrete, necessitating further repair work in 2008. The contractor, VSL, carried out major works on the abutment, including concrete removal and encasing, and installing post-tensioning cables to ensure the continued integrity of the structure [11].

6 Loading

The simplified load analysis in this paper is based on the British Standard specification for loading of bridges, BS5400 [12]. All of the nominal loads determined in this analysis will need to be factored by a partial load factor, $\gamma_n$, and an accuracy factor to allow for inaccuracy in the analysis, $\gamma_a$. $\gamma_a$ varies depending on the material and is given in BS5400. $\gamma_n$ is 1.10 for this elastic analysis at ultimate limit state [12].

6.1 Dead and Superimposed Dead Load

The total weight of structural steel used in construction of the bridge was 16520 tonnes. It can be assumed for the purposes of this analysis that the steel is distributed evenly along the entire span of the bridge, equating to an unfactored load of 317kN/m for the entire steel superstructure, with the partial load factor, $\gamma_n = 1.05$. This will overestimate the load at the centre of the span, since it neglects to take into account the steel used to construct the piers at each end, and so is a conservative assumption.

The bridge deck, which is 12.2m wide, consists of a concrete and asphalt road surface (originally tarmac) on steel cross beams, supported by stringers and cross-girders, which are attached to the cables (see Figure 7). The concrete and asphalt road surface is approximately 300mm thick, and if the density is assumed to be 2400kg/m³, the unfactored weight per unit length of bridge is 86.2kN/m ($\gamma_n$concrete = 1.15). The proportion of the steel used in the bridge deck could be very approximately assumed to be 1/5th of the total steel in the structure, considering the majority is within the arch. This would give the weight of the steel in the road deck as 73kN/m bridge length.

6.2 Vehicular Load

Following the procedure in BS5400, the road width of 12.2m gives 4 notional lanes of 3.05m each. A loaded length of 504m gives an HA loading of 19.3kN/m. In load combination 1, with HA and HB loading, $\gamma_n = 1.30$, therefore the design HA loading is 27.6kN/m. The KEL is taken as 120kN per notional lane.

Due to the arterial function of the bridge, and the location near large ports in New York, the maximum 45 units of HB loading should be applied. With a cable spacing of approximately 12.5m, the most onerous HB vehicle lengths would be an inner axle spacing of 11m for the hogging case and 6m for the sagging case. For the purpose of this analysis, the point loads generated by the wheels on each axle will be generalised to one point load acting at the centre of the overall footprint of the HB vehicle.

$$\text{HB}_{\text{point load}} = 45 \times 10 \times 4 = 1.8 \times 10^3 \text{kN}.$$ 

For the combined HA, HB load case, the loads need to be positioned on the notional lanes in order to generate the greatest stress in the member or section of the structure undergoing analysis. The HA load is also multiplied by a lane factor, with the lanes being ordered from the most onerous to the least. The design HA load then becomes: $\text{HA}_{\text{design}} = \gamma_n \gamma_f \rho_n \text{HA}$.

Where $\rho_n$ is the lane factor for lane $n$, and is obtained from table 14 of BS5400, depending on the...
number of notional lanes and bridge length. In this case $\beta_1 = 1.0$, $\beta_2 = 0.67$, $\beta_3 = 0.6$, $\beta_4 = 0.6$. The worst case load configuration is shown below in Figure 10:

![Figure 10: Live load distribution on road deck for ultimate limit state design](image)

Table 1: Summary of loads acting on the bridge

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unfactored $\gamma_f$</th>
<th>K/N/m</th>
<th>$\gamma_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL_T</td>
<td>Overall dead load</td>
<td>12.2</td>
<td>317</td>
<td>1.05</td>
</tr>
<tr>
<td>DL_J</td>
<td>Road deck dead load</td>
<td>5.2</td>
<td>63.4</td>
<td>1.05</td>
</tr>
<tr>
<td>SDL</td>
<td>Superimposed dead load</td>
<td>7.1</td>
<td>86.2</td>
<td>1.15</td>
</tr>
<tr>
<td>HA</td>
<td>HA load (per lane)</td>
<td>6.3</td>
<td>19.3</td>
<td>1.30</td>
</tr>
<tr>
<td>PL</td>
<td>Pedestrian load (per path)</td>
<td>1.2</td>
<td>1.8</td>
<td>1.5</td>
</tr>
</tbody>
</table>

6.3 Pedestrian Load

From clause 6.5.1 of BS 5400-2:2006, the nominal pedestrian live load can be calculated for Bayonne Bridge, assuming no exceptional crowd gatherings, of 1.2kN/m$^2$. With a width of approximately 1.5m, and $\gamma_n$ = 1.50 for pedestrian live loads, the load per unit length in the direction of bridge span is 3.0kN/m. This is an insignificant load compared to the dead and vehicular loads, and therefore needn’t be considered in the large scale approximate analyses carried out in this paper.

6.4 Secondary Load Effects

6.4.1 Parapet Collision

Vehicle collision loads with the parapet do not need to be considered in combination with any other secondary load effects. Due to the bridge’s location high above a body of water, the parapets should be designed to a high level of containment, which according to BS6779-1:1998 means they can withstand an impact from a four-axle rigid 30 000kg HGV travelling at 64km/h at 20° from the direction parallel to the parapet. This can be simplified according to BS5400-2:2006 to a single horizontal transverse load of 500kN applied at 1.65m above the road surface [12]. This load is applied uniformly over a 3m length along the parapet, and this equates to a moment at the base of the parapet, $M_p$, of 275kNm/m.

6.4.2 Pier Collision

The arch abutments are at the edge of the navigable channel of the Kill Van Kull, and therefore the risks of collision by vessels should be considered and the local authority should be consulted.

6.5 Other Load Effects

6.5.1 Erection Loads

During construction, the arch was supported by falsework, and was used to support cranes and other construction machinery. The top chord of the arch will therefore have been in tension at certain times. These cases need to be designed for also.

6.5.2 Fatigue

Steel is not generally susceptible to creep or shrinkage, but may be affected by fatigue if subject to cyclical loading. British Standard BS5400-10:1980 gives guidance on the code of practice for fatigue analysis of bridges, and is based on the Palmgren-Miner rule for damage calculation.

7 Strength

Using the load cases determined above, a simplified calculation of the bending in the deck, and compression in the arch, can be carried out, along with a consideration of the wind and temperature effects acting on the bridge.

7.1 Road Deck

7.1.1 Transverse

Looking at the bridge in cross section the road deck is effectively a simply supported beam, suspended from a set of cables at each edge (see Figure 7). The worst case loading for sagging in the span can be drawn using the vehicular live load shown in Figure 10 and is shown below (see Figure 11):

![Figure 11: Ultimate limit state transverse deck loading](image)

Taking moments about A, the reactions can be calculated as: $V_A=2810kN$, $V_B=3283kN$. The maximum moment can be assumed to be at the location of the HB point load. Taking moments about that point gives: $M_{max} = 21756kNm$.

Assuming all of the moment is carried by the large transverse I-beam girder, the maximum stress can be found at the extreme fibres, assuming member sizes as shown in Figure 13:

$$\sigma = \frac{My}{I}$$
7.1.2 Longitudinal

Longitudinally, the deck is a continuous member suspended by cables under the arch, with discontinuity at each end where expansion joints are located, presumably where the deck crosses within the arch. The end conditions are therefore assumed pinned, and the 26 sets of cables act as supports that are not entirely rigid. Therefore in reality the bending moment diagram will have slight overall sag. The worst case loading for sagging is to have every other span fully loaded, and the ones between simply with an unfactored dead load applied (see Figure 12).

\[ y = 1.5\text{m} \]
\[ I = \sum l_{\text{local}} + a^{2}y \]
\[ = 0.167\text{m}^{4} \]
\[ \therefore \sigma = 192.0\text{N/mm}^{2} < 193.1 \therefore \text{PASS.} \]

For simplification, only the HA live loading will be considered, without knife edge loads, and the loads will be applied uniformly along the length of the bridge. This will allow us to consider the deck as a simple continuous beam, with uniformly distributed load, such that the maximum sagging and hogging moments are found in the first span and at the first interior support. Their approximate value is given in BS8110 as 0.09FI sagging moment and 0.11FI hogging moment, where F is the uniformly distributed load times the span length, and l is the span length. The moment is then assumed to be taken equally between the 6 stringers, and their member size taken from Figure 13:

\[ w = y_{f1}y_{f3}b \left( \sum \beta_{n}HA + SDL + DL \right) = 547\text{kN/m} \]
\[ M_{\text{hog}} = 0.11Fl \]
\[ = 9402\text{kNm}. \]
\[ y = 0.75\text{m} \]
\[ I = 0.05\text{m}^{4} \]
\[ \therefore \sigma = 23.5\text{N/mm}^{2} < 193.1 \therefore \text{PASS.} \]

This is an unconservative approximation, which doesn’t take into account HB loading, knife-edge loading or the elasticity of the cables.

\[ \sigma = F/a \]
\[ a = 4\pi r^{2} \]
\[ \therefore \sigma \approx 418\text{N/mm}^{2} < 460 \therefore \text{PASS.} \]

7.2 Cables

The tension in the cables can easily be calculated from the reaction at support B found from the transverse load case, of 3283kN. There are four cables grouped at this point of diameter \( \sim 50\text{mm} \), and yield strength \( \sim 460\text{N/mm}^{2} \):

\[ \sigma = F/l \]
\[ a = 4\pi r^{2} \]
\[ \therefore \sigma \approx 418\text{N/mm}^{2} \approx 460 \therefore \text{PASS.} \]

7.3 Arch

The total height at midspan is 81.1m, where its depth is 11.43m. The distance between the bottom of the arch and the road deck is 23.95m.

The lower chord of the arch is the primary load carrying component of the entire structure, transferring the loads from the suspended cables as axial compression to the abutments. The final member of the lower chord sits on the abutment at an angle of 30°, and if we assume that all of the thrust of the arch is carried through that lower chord member, we can calculate the axial force from the vertical reaction at the base, which we can approximate for the dead load case because we know the total weight of steel used was 16520 tonnes.

\[ V = 16520 \times 1000 \times 9.81 \div 4 \]
\[ = 40.0\text{MN} \]
An area of 15600mm$^2$ equates to a hollow steel box section 1.6m deep, 1m wide with a thickness of 50mm.

A. Thrall and D. Billington conducted a detailed analysis of Bayonne Bridge and found that the lower chord arch members were working at an average efficiency of 0.98 of their yield strength [2]. This suggests a very good use of material and very considerate and thorough design and planning.

8 Wind Loading

Wind loading is particularly crucial on a suspended arch bridge, because the cable offer no lateral support, so all of the lateral load must be taken in shear by the deck and transferred at either end to the superstructure. The analysis taken to BS5400 will still apply to the Bayonne Bridge because the conditions are similar to those found in the UK. A 120-year return period value of wind speed, $\nu$, will be assumed of 30m/s.

$F = \frac{40.0 \times 10^6}{\cos 30} = 46.8\text{MN}$.

If $\sigma_{\text{max}} = 193.1\text{N/mm}^2$

\[ F = \frac{46.8 \times 10^6}{\sigma} = \frac{300}{242000\text{mm}^2}. \]

$\gamma = \frac{1.59}{\sqrt{1.75 \times 1.00 \times 0.99 \times 1.00}} = 1.3$.

The lateral force acting on the bridge, $P_e$, is given by:

\[ P_e = qA_s \cos \theta = 13.3\text{MN}. \]

Where $q = 0.613\nu^2$. $A_s$ is the solid horizontal projected area and $C_D$ is the drag coefficient. The depth of the deck is 3.7m (see Figure 7), and with live loading depth of 2.5m, the total depth is 6.2m. The loaded length between the arch is 400m, therefore $A_1 = 2480\text{m}^2$. $C_D$ would normally have to be determined using wind tunnel tests for this type of bridge deck, but here we can assume a value of 1.3, taken from the graph for a b/d ratio of 5.76.

The total lateral force imparted on the deck by the wind is therefore 13.3MN, which can be assumed to distribute equally 6.65MN to each end of the bridge. It can then be assumed that the steel I-beam stringers transfer the shears to the bridge arch superstructure, and have shear strength 0.4 times their tensile strength, to give a total shear resistance, $S_K$:

\[ S_K = vA_t = 22.8\text{MN}. \]

22.8MN $> 6.65$ : PASS.

In reality, the entire system of the bridge deck, including diagonal members and concrete road deck, would act as a system to transfer the shear load, depending on their relative stiffness.

9 Temperature

A worst case scenario can be envisaged where the expansion joints at each end of the bridge have seized entirely due to lack of maintenance, and the bridge deck undergoes a large temperature variation. The temperature variations in New York can be quite severe, and assuming the joints seize at the coldest point in winter, the temperature may increase by 50°. This is a highly simplified analysis, and in reality, temperature effects will require other considerations, such as buckling of the deck.

\[ e = k\Delta T/l = 0.24\text{m} = 0.24\text{m} \]

\[ \sigma = \frac{eE}{l} = 120\text{N/mm}^2. \]

10 Natural Frequency

The natural frequency of a bridge should be between 5Hz and 75Hz. The lower limit is to avoid wind induced vibrations, and the upper limit exists for the comfort of bridge users.

An approximate calculation of the natural frequency can be carried out based on the following equation:

\[ f_0 = \omega_n = (\beta_n l)^2 \sqrt{EI/ml^4}. \]

If we make the assumption that the stringers, which run longitudinally along the bridge span, are the most significant source of resistance. The continuous bridge is a clamped-clamped condition, therefore $\beta_n l = 22.37$.

\[ l = 6 \times 0.05\text{m}^4 \]

\[ E = 200\text{kN/mm}^2 \]

\[ m = 63.4\text{kN/m} \]

\[ I = 400\text{m}. \]

$\omega_n = 3.0\text{Hz}$. This is lower than the desired 5Hz, so the vertical acceleration of the deck must therefore be limited to $0.5\sqrt{f_0}$ m/s$^2$, as specified in BS5400 [12]. With a deck of the stability and mass of the Bayonne Bridge, it is likely that this criteria can be met.

11 Serviceability

It is also necessary to check the bridge under the serviceability limit state, to ensure that deflections occurring under normal circumstances are within reasonable limits. In the serviceability limit state, $\gamma_f$ and $\gamma_s$ are usually taken as 1.00.
A simplified analysis can be carried out to determine the extension of the steel cables supporting the bridge deck at the midspan. Assuming full HA live loading uniformly over the bridge, but neglecting knife edge loads and HB loading for simplicity, the load per cable group can be determined using the values for loads in Table 1, as 3456kN.

Assuming equal distribution between the two cables, and the area of steel previously calculated is 7854mm$^2$. The Young’s Modulus of steel is 200kN/mm$^2$, and the length of the cables is the distance between the bottom of the arch and the deck, which has been previously stated as 23.95m:

$$F_{cable} = \frac{1782kN}{F_1}$$

$$e = \frac{1}{AE} = 26.4mm \ll \text{span/360.}$$

12 Durability

Figure 14: Detail of the arch ending at the abutment, behind a pile wall

The bridge’s 1997 biennial inspection identified numerous locations where members had suffered corrosion, including stringers, diaphragms, rocker bearings, anchor bolts, lateral bracing and stiffeners [13]. The Port Authority approved major rehabilitation work in order to modernize the roadway to accommodate increasing vehicle loads, and stabilize the bridge structure in order to provide users a ‘smooth ride’. The work carried out included selected replacement of support steel, and reinforcing the structure with welded plate girders and box beams using high performance HPS70 grade steel [13]. The presence of corrosion in structural members can clearly be seen on inspection of the arch base at the abutments, shown in Figure 14.

In one incident in 1990, one lane was closed overnight after a crane being towed by a tugboat struck the central span, but no significant damaged was caused by this or any other collisions, suggesting a sturdy deck design.

12.1 Vandalism

There have been no documented reports of problems with vandalism to date. It is unlikely that Ammann took into consideration potential vandalism in the same way that a designer may do today, since there were no spray cans in the 1930’s, but the bridge’s industrial appearance and style may have helped it avoid becoming a target, as there are no large blank walls particularly susceptible to defacing, and neither are there any weak elements to the bridge that could be damaged easily.

13 Future Changes

13.1 Height Restriction

Figure 15: A large container ship squeezing under the central span.

Because of the increasing size of ocean going vessels, tankers and container ships, the once plentiful central clearance height above sea level of 45.7m is now severely restricting ships heading for the ports upstream, with some of the largest only able to pass at low tide, and then only with special foldable masts installed (See Figure 15).

This is a major concern for the Port Authority, because the ports are such a significant feature of the local economy, with 5000 ships passing under it every year, and there is the danger that the New York ports will start to lose out to competitors [14].

Two options currently being considered are to raise the bridge deck, or replace the bridge altogether. The bridge deck could be removed; the piers at each end raised, and replaced 35 feet higher. This would cost less, at ~$500million, but may only be a short term fix, as ships are continuously increasing in size. It is also argued that raising the height of the deck within the existing arch would destroy the aesthetic of the bridge.

In order to make the most of this opportunity, it is considered that replacing the bridge would prove a more long-term solution to the problem, although it may cost twice as much. The replacement would most likely be a cable stayed bridge, much like many other recent bridges, and would certainly not possess the character of Ammann’s creation.

Either way, the future of the bridge is now looking far from secure, as expansion work on the Panama Canal is due to be completed in 2014, which will see
thousands of even bigger tanker ships visiting the east coast that won’t be able to fit under the bridge [14].

13.2 Suggested Improvements

Figure 16: Bayonne Bridge lit at night.

In many respects Ammann built on Lindenthal’s design for the Hell Gate Bridge, creating a more efficient arch, and attempting to avoid preconceptions of how a bridge should look; its monumentality and rigidity. He did away with the reverse curvature spandrel ends that are visible on the Hell Gate Bridge, but do serve no structural purpose, as he believed beauty was achieved through efficient structural design (See Figure 2). However, there are some concessions to the perceived aesthetic of the time, which in some respects contradict with the pursuit of efficiency.

One example of this is the spandrel form, which Ammann chose over a crescent arch for its aesthetic qualities (see Figure 5). This shape seems to fit better when enclosed with large piers, as in the Sydney Harbour Bridge, and although Ammann intended to have stone clad piers, they were not on the same scale. In hindsight, it is this lack of stone that has singled out the Bayonne and George Washington Bridges as beautiful expressions of pure steel.

Therefore, it could be argued that the arch would have been a purer form had it followed the crescent design, and omitted stone clad piers from the design, but this is light criticism of a bridge which has stood the test of time, and helped to launch the career of a great engineer.

14 Conclusion

A detailed aesthetic, historic and structural analysis has been conducted on the Bayonne Bridge, and it has been shown that both the design and construction of this bridge were thought through and carried out with a great deal of success. Since being completed ahead of time and under budget, the Bayonne Bridge has been a huge asset to the surrounding area of Bayonne and Staten island, and while it may not be well known among the wider population, there is much positive sentiment towards the bridge in both the local area and the bridge engineering field as a whole.

It is widely recognised that Ammann was a master builder of bridges, who came to exert an unrivalled influence on the bridge-scape of New York City. It is partly thanks to his success at Bayonne that he was able to continue on his path to notoriety.

This paper has shown that Bayonne Bridge was adequately designed to cope with the increasing traffic loads placed on bridges since the time of its completion, but that no one could foresee the even greater increase in the size of ocean going vessels that now threaten to close the bridge forever.

15 References