A CRITICAL ANALYSIS OF THE MARTIN OLAV SABO BRIDGE, MINNEOSOTA

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Abstract: This conference paper will provide a detailed study on the Martin Olav Sabo Bridge in Minneapolis, Minnesota. The cable-stayed design will be critically analysed in aesthetics and the structural form will be assessed considering a full range of loadings. The paper will also evaluate the construction of the bridge, its durability and the possibility of future changes.

Keywords: Minneapolis, Cable-stayed, Asymmetric, Semi-Harp, Pedestrian

1 Introduction

The Martin Olav Sabo Bridge is situated in the city of Minneapolis and is the first cable stayed bridge to be built in the state of Minnesota. The whole bridge has a length of 671m and has a main span of 65.5m [1]. The height of the pylon is 30m. The bridge is named after the US congress member Martin Olav Sabo who campaigned for the construction of the bridge.

Its primary function is to provide a crossing for cyclists and pedestrians over a busy 6-lane highway. By doing so it eliminates the need for pedestrian/cyclists to cross the highway using a dangerous at-grade crossing. The Sabo Bridge is part of the ‘Midtown Greenway’ pedestrian trail and upon completion in 2007 it linked up the southern and northern sections of the route. This allowed safe continuous passage for pedestrians through the city.

The cable-stay design of the structure presents superb aesthetic qualities while allowing the bridge to suitable lie within the constraints of the site (Section 3). Particular

emphasis has been put on the slenderness of the design so that the bridge provides a landmark for the Midtown Greenway and essentially for the city.
2 Aesthetics

The ‘Martin Olav’ Sabo Bridge has a very interesting aesthetic appeal which can be attributed to many features of its design. These features should fulfill criteria described by the 20th century bridge engineer Fritz Leonhardt. It is the fulfillment of these criteria which in theory leads to an aesthetically pleasing bridge design. It is upon Leonhardt’s assumptions that the aesthetic analysis of the Sabo Bridge will be based.

Firstly it must be noted whether the bridge fulfils its function. The beauty of a cable stay bridge is that its structure is in plain view and the Sabo Bridge is no exception. The inclined pylon and cable-stays are obviously key structural elements and they have been designed so that they stand out. This is emphasized at night when the pylon and cables are illuminated to enhance their aesthetic appeal. Also as a pedestrian bridge the structure of the bridge has been made gracefully slender. This is a key element which makes the aesthetics of the bridge work. If a heavy structure had been used it would have been overdesigned structurally and would look out of place on the pedestrian route. The dramatic crossing also acts as a stimulant for motorists on the highway below.

The bridge is also well proportioned. The stays themselves act as stabilizing elements of the structure which allows the deck and pylon to become slender as shown in Fig. 3. This creates a balance between the deck and pylon masses. If alternatively the deck was too massive, it would make the pylon and cables seem ineffective. The bridge also has a balance between its depth and height. This ensures that the pylon does not seem out of place on the deck and allows it to stand out to show its functionality. The inclined pylon acts much like a large sundial casting interesting shadows throughout the day. The bridge also uses shadowing to emphasise certain elements. The positioning of the bridge with reference to the sun means that the deck is rarely cast in shadow. This creates a light deck and a shadowed soffit. This emphasizes the slenderness of the deck.

The asymmetric design of the bridge means that many of its features are unique in relation to the rest of the bridge. This includes the pylon, the pier under the pylon, the back-stays, and the main stay. However the bridge manages to keep its order through other design features. The cylindrical style of the piers is repeated throughout the bridge design. Also unnecessary edges and lines have been removed by cambering the tops of piers so they smoothly link into the soffit of the bridge. The fact the cables are arranged in a semi-harp design ensures a more ordered cable stay arrangement. The cables are parallel and cross each other at constant angles which give the bridge an order which is aesthetically pleasing. It also minimizes the crossing of cables when the bridge is viewed from oblique angles as shown in Fig.4.

Refinements on the bridge help to enhance its aesthetic appeal. The piers follow the Greek idea to taper the cylindrical columns to prevent the optical illusion that the top of a parallel column looks wider than its base. The use of one column per pier also ensures that an oblique view of the bridge will not create an opaque barrier of sight. Also the largest span of the structure is in the middle of the bridge with the piers holding smaller spans towards the edges of the bridge. Another refinement is that the bridge deck becomes wider at the pylon so that the bridge is not restricted by the structural element.

The bridge is in an unusual environment, it crosses a busy highway which is situated on the outskirts of downtown Minneapolis. This means it has a backdrop of skyscrapers, office building and modern apartment blocks. Therefore the bridge needed to have eye-catching features to make it stand out against this modern landscape, yet not be a heavy and solid to look out of place next to the local neighbourhood houses. The Sabo Bridge utilized a distinctive asymmetric shape to stand out in front of the city backdrop but its slender structure is also graceful enough not to impose on the surrounding areas. However although it is suitable for this environment, the Sabo Bridge would look much more elegant if it was used as a crossing for a river.

The texture of the bridge is very important, due to the fact that as pedestrians cross the bridge they experience it firsthand. The pylon is screened by polished stainless steel screens and the rest of the top decks components follow this metallic texture. The abutments will be of polished steel and so the benching on the bridge is made of aluminium. This all helps to portray a sleek and modern appearance. The concrete beams and soffit of the bridge have been given a smooth finish to indicate their slenderness.

During the day, the bridge follows a specific colour scheme of metallic greys and white. The piers, deck, and surfacing of the bridge have all been constructed using a white concrete. This gives the piers a clean and sleek appearance. The pylon, abutments and benches have followed a metallic colour scheme with all materials being polished to bring out their natural colours. At night

![Figure 3: Close-up of bridge deck](image3)

![Figure 4: Oblique views of the Sabo Bridge](image4)
the bridge is illuminated in blue lights as shown in Fig. 5. This lighting lends well to the metallic finish due to the resulting reflections.

![Figure 5: Sabo Bridge at night](image)

The Sabo Bridge has aesthetic character due to many reasons. The inclined pylon gives the bridge an element of uncertainty which adds a degree of character. The slenderness of the structure gives the bridge a very sleek modern feel and combined with the lighting at night, the bridge comes alive with character. Particularly notable are the illuminated steel cables which are reminiscent of blue lasers.

The bridge is simple in the way that its structural components are very obvious. For example it is blatant that the steel cables are holding up the bridge deck. However there is a visual complexity to the bridge in that the visual arrangement of cables changes when the bridge is viewed at different angles. This can be a negative feature due to the fact that the cables can look chaotic from some perspectives.

A specific landscaping plan was drafted for the bridge which incorporates nature into its modern design. This compromised of trees, shrubs and grasses. This greenery assists with the efforts of volunteers on the Midtown Greenway who are attempting to establish nature in the more urban areas of the trail [2].

3 Structural Form

The structural elements of the Sabo Bridge are typical of a cable stay design and were influenced by a number of constraints at the development site. Firstly the bridge needed to span the Hiawatha highway without situating large piers in the middle of the road. Secondly, a thin deck was required to provide adequate spacing between the bridge and power lines which would travel above and below the deck. Thirdly, there were vertical restrictions on the eastern side of the development due to the power lines. These constraints all lead the design down a specific route and an ‘asymmetric cable-stay’ design was chosen. This would utilize available space around the highway and also satisfy aesthetic criteria.

The need for a slender deck and piers eliminated many cable stay designs. Structures which used the deck or the piers as stabilization would not be suitable. This is because they would require the elements to be made very stiff to take the moments they were subjected to. Therefore the Sabo Bridge uses a structural design which utilizes the cable stays themselves as the determining stabilizing element. This only generates a moderate compression in the deck which allows itself and the pylon to be slender.

3.1 Cables

The ‘stays’ used on the bridge are locked coil cables as shown in Fig. 6. The wires in these cables are arranged in successive layers which are wound around a central core consisting of parallel wires. On the outside of this core are wires with elongated S-sections. These lock together to form a watertight barrier [3]. Using this type of cable has many advantages. The cables are economically viable due to the fact that ducts and grouting are unnecessary. They are also easy to place and have good flexibility. The anchorage spaces needed for the cables are also much smaller than those for other cables types.

Due to the asymmetric design of the bridge, backstays are needed to transfer the tensional loading of the main stays to the ground. This is done by securing them to earth anchors which will be discussed in Section 3.4.2. In this design it is very important that all cables are in tension and do not become slack when live loads are applied. This is achieved by pre-stressing the cables during construction and also by designing the length of the side span (16.8m) to be less than half of the central span (65.5m).

The bridge is a variation of a double plane cable stay bridge. This means it uses two sets of cables attached to either side of the bridge to take bridge loadings. Using two sets of cables also provides torsional restraint to the deck and this contributes to the slender deck design. The cables on the bridge are arranged in a semi-harp formation. This formation is not as efficient as a fan system however it provides more aesthetic appeal as discussed in Section 1.

![Cross Section](image)

Figure 6: Locked Coil Cable Details

3.2 Deck

The deck is made of reinforced concrete sections. This material allows the deck to vary in size along its length. For example the deck become thin towards its midsection and thicker towards the piers as shown in Fig. 7, 8. This takes account of the varying bending moments within the deck. For example the thicker deck component will be required to take more compression than the midsection which will primarily be in tension. The deck will also be cambered over the length of its span to compensate for deflection during loading. The camber was calculated using dead and superimposed loads and...
used a 3D time dependent analysis to account for creep and shrinkage over time.

![Figure 7: Deck section at mid-span](image)

Figure 7: Deck section at mid-span

Figure 8: Deck section at pier approach

The main cable stays are attached to the bridge deck at either side of its width. This is done by first securing the cables in anchor sockets situated within specially designed steel anchor tubes. These anchor tubes are then attached to the deck sides using an embedded steel bearing plate. The bearing plate is secured to the deck using pins and pre-stressed steel reinforcement bars.

### 3.3 Pylon

A key feature of the bridge is that a singular pylon is used to provide a structurally elegant feature. This means that the two sets of both main stays and backstays are all supported by the same pylon which increases the stresses it needs to bear. The pylon is made of steel and is a variation of an I-beam with inclined flanges for aesthetic reasons. The top of the pylon is covered by a steel cap plate and the bottom is secured to the deck and pier using a steel bearing plate. Stiffeners are used to provide rotational restraint to the pylon. To reduce bending moments the pylon is inclined at 20° to the vertical.

The cables will be attached to the pylon at specific points along the pylon length. The first cable will be attached 17m from the bridge deck to provide adequate space for pedestrians and cyclists to pass underneath. The cables will be anchored using open spelter sockets. These will then be attached to steel diaphragm plates situated within the pylon.

### 3.4 Foundations and Geotechnics

The geological make-up of the site was found using several boreholes at specific points. It was discovered that the soil made of ‘poorly graded sand/gravel’ and there are layers of limestone beneath this at a depth of about 15 meters. Due to the nature of the project and ground conditions, it was an obvious choice to use piles. The piles used for the structure are cast in place concrete piles and are 0.25m in diameter. They are constructed by boring into the ground using steel shells. These shells are then filled with concrete.

#### 3.4.1 Pier 2

Pier 2 is the pier on which the pylon sits and is ultimately the pier which deals with the majority of moments generated by the pylon. As seen in Fig. 9, 25 piles have been used to anchor the pier into the ground. Due to the fact that the soil is poorly graded means that its strength is not as high as densely packed sands. This is due to the fact that the angle of shearing resistance and bearing capacity factor are lower values in loose soil [4]. Therefore to increase strength the piles were bored to a depth of 15.25m where they become end bearing and can utilize the strength of the limestone layers. Loads are transferred to the piles through a pile cap with dimensions 1.5 x 4.5m x 4.5m.

![Figure 9: Pier Details](image)

Figure 9: Pier Details

#### 3.4.2 Earth Anchorage

The forces carried by the backstays need to be transferred to the ground and this was done using large concrete blocks as pictured in Fig. 3. The cables are anchored to the blocks using open bridge sockets which are then attached to pre-stressed rock anchors situated within the concrete.

![Figure 10: Photograph of concrete earth anchors](image)

Figure 10: Photograph of concrete earth anchors

The concrete blocks also need to be anchored to the ground. Due to the fact the cables are in tension means that the foundations of the concrete blocks need to withstand this tensile load and resist the associated uplift. This is achieved in 3 ways. Firstly the sheer mass of the concrete block resists uplift force. Secondly 6 piles are used for each rock anchor. In tension the piles bear no capacity from the base and therefore loads are shed through skin friction. Lastly 3 rock anchors are used per block to attach the blocks to the limestone layers under the ground. This allows tensile loads to be transferred to the limestone as this cannot be done through the piles.

The earth anchors are tied to the foundation of pier 2 using compression struts. These are made from reinforced concrete and will take the lateral forces which would be imposed on the ground by the foundations. This ensures that sliding of foundations will not occur.
4 Loadings

It is important to design bridges to withstand all of the various loadings which will be applied during their lifetime. The means the bridge was designed using a limit state philosophy. By use of codes, loadings are predicted and multiplied by corresponding factors of safety. Engineers can then design the structural components of a bridge to withstand these loadings. This ensures that the bridge is serviceable and will not collapse.

The Sabo Bridge was designed using “AASHTO design specifications” and imperial units. However due to the unavailability of these codes, the analysis of loadings in this paper will use ‘BS5400-2:2006’ and SI metric units.

4.1 Dead and Super Imposed Load

Dead Loads consider the self-weight of the bridges’ structural elements. The reinforced concrete deck of the bridge is the main structural dead load and the pylon will be assumed hold its own weight.

Concrete deck:
\[
= \text{Density} \times g \times \text{crosssectional area.} \quad (1)
\]
\[
= 2400 \times 9.81 \times 3.97 = 93.47kN/m. 
\]

Superimposed load considers self-weight of non-structural elements. They have a more conservative factor of safety due to possibility that they will change during the bridges life.

Parapets:
\[
= 1.25kN/m(\text{assumed}). \quad (2)
\]

Concrete Surfacing(100mm):
\[
= 300 \times 9.81 \times 0.55 = 1.62kN/m. \quad (3)
\]

4.2 Live Loading

4.2.2 Primary Pedestrian Loading

The Sabo bridge is a pedestrian/cycle bridge and therefore its primary live loading will not consider vehicular traffic. The live loading on the bridge due to pedestrian traffic will be uniformly distributed over the whole span. As the bridge is over 36m long the live load is found using Eq.(4).

\[
W_{\text{PEDST.}} = \left( \frac{\text{nominal HA UDL} \times 10}{L+270} \right) \times 5kN/m^2. \quad (4)
\]
\[
= \left( \frac{151 \times 10}{65.5 + 270} \right) \times 5 = 3.08kN/m^2.
\]

4.3 Wind Loading

In this paper the wind loading on the Sabo Bridge is to be analysed using static procedures based on Ref. [5]. It will also be assumed that the wind acts only on the main span of the bridge as this is where loadings will become significant. Wind loading is an important consideration as it can cause unexpected effects on bridges. If further information is required on the effects of dynamic response due to turbulence, then specialist advice should be sought.

Wind loading is based on the maximum gust (Vc) found in Eqn.(4). This is based on local hourly mean wind speeds which are usually documented in the relevant standards. However as the Sabo Bridge is based in Minnesota, the mean hourly wind speed is taken as ‘4.8m/s’ from Ref. [6].

The fact the bridge is a footbridge means Vc can be reduced by a factor of 0.77 due to wind break action.

\[
Vc = \text{Mean Speed}(Vs) \times \text{Gust Factor}(Sg). \quad (5)
\]
\[
= 6.55 \times 1.12 = 7.33 \text{ m/s.}
\]

\[
Vc \times \text{reduction factor}(0.77) = 5.65 \text{ m/s.} \quad (6)
\]

Where,

\[
Vs = Vb \times Sp \times Sa \times Sd,
\]
\[
Sg = Sb \times K_f \times T_g \times Sh'.
\]

Table 1: Values for Eqn. (5)

| Vb|=4.8m/s | Sp|=1.05 | Sa|=1.3 | Sd|=1.0 | Vs|=6.55m/s |
| Sb|=1.55 | K_f|=0.86 | T_g|=0.84 | Sh|=1.0 | Sg|=1.12 |
4.3.1 Transverse Wind Loading

Transverse wind loading ($P_t$), calculated in Eqn. (7) will occur over the normal elevation of the bridge and will act at the centroid of the main span deck. Guidelines for the area at which the wind acts are given in Ref. [5]. As the bridge has open parapets, the wind load is assumed to act over the combined vertical length of the deck and the pedestrians upon it.

\[ P_t = q \times A_1 \times C_D. \]  
\[ = (0.613 \times 5.65^2) \times 121.83 \times 2 = 4.77 kN. \]  

4.3.2 Longitudinal Wind Loading

Longitudinal wind loading is transferred to the bridge directly ($P_{LS}$) or indirectly through pedestrian/ cyclists ($P_{LL}$).

\[ P_{LS} = 0.25 \times q \times A_1 \times C_D_1. \]  
\[ = 0.25 \times 19.57 \times 121.83 \times 2 = 1.19 kN. \]  

\[ P_{LL} = 0.25 \times q \times A_2 \times C_D_2. \]  
\[ = 0.25 \times 19.57 \times 81.9 \times 1.45 = 1.16 kN. \]

4.3.3 Vertical Wind Loading

The vertical wind loading ($P_v$) on the substructure can cause uplift or a downward vertical force. It is important to find the extent at which the wind load will either add to bridge loading or relieve it. The area over which wind load acts, is the plan area of the main span.

\[ P_v = q \times A_3 \times C_l. \]  
\[ = 19.57 \times 425.75 \times 0.4 = 3.33 kN. \]

4.4 Temperature Effects

4.4.1 Effective Temperature

The increases and decreases in the overall temperature of the bridge cause the structural components to expand and shrink. This is directly associated with the extreme temperatures at the bridges location in Minneapolis, Minnesota. According to Ref. [7], ‘Minnesota has a continental climate with cold winters and hot summers. The state’s location in the Upper Midwest allows it to experience some of the widest variety of weather in the United States’. This means that temperature fluctuations are very high and from Ref. [8] we can find the maximum and minimum temperatures in Minneapolis are 40°c and -34°c respectively. Using Ref. [5] we find that these temperatures can be relieved by 2°c due to the fact that a 50-year return on the bridge is adequate. To find the effective temperatures we use the tables in Ref. [5], which gives 38°C for maximum effective temperature and -24°C for minimum effective temperature. This gives a very high effective temperature difference ($\Delta T$) of 62°C. This can then be used to find the strain in the bridge ($\epsilon$), the length of expansion/shrinkage ($\delta$) and the stress imposed ($\sigma$). Eqn. (11, 12, 13)

\[ \epsilon = \alpha \times \Delta T. \]  
\[ = 12 \times 10^{-6} \times 62 = 744 \times 10^{-6}. \]  

\[ \delta = \epsilon \times L. \]  
\[ = 744 \times 10^{-6} \times 65.5 = 48.7 mm. \]

\[ \sigma = E \times \epsilon. \]  
\[ = 30000 \times 744 \times 10^{-6} = 22.3 N/mm^2. \]

This stress imposed on the deck is an unwanted effect which could cause cracking. To minimise the temperature effects, the bridge uses expansion joints as shown in Fig. (13). These are built into the deck at specified points and allow the deck to expand up to 57mm without inducing stress.

4.4.2 Deck Temperature Difference

Due to the way heat travels through the deck materials, temperature differentials are set up within the structure. These will be analysed using Ref. [5]. A ‘positive’ difference is created when the top of the deck gains heat whereas a ‘negative’ difference is created when the deck looses heat. Our deck is made of concrete with a surfacing of 100mm and has a temperature profile as shown in Fig. 14. The deck height changes (Fig. 8) according to where it is along the bridge length and therefore so does the temperature differences. Table 2 shows the temperature difference at mid span.

\[ Figure 14: \] Temperature profiles of concrete deck

### Table 2: Temperature differences at mid-span (h=0.2m)

<table>
<thead>
<tr>
<th>Positive Temperature Difference</th>
<th>Negative Temperature Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_1 = 0.07m$</td>
<td>$h_2 = 0.10m$</td>
</tr>
<tr>
<td>$T_1 = 2.0^\circ C$</td>
<td>$T_2 = 0.5^\circ C$</td>
</tr>
</tbody>
</table>

4.5 Accidental Collisions

The Sabo Bridge has minimum headroom of 6.9m. This means the substructure does not need to be designed against vehicle collision [5].
Figure 15: Vehicle collision loads on bridge piers

4.6 Parapet Loading

The parapets on the bridge need to hold the weight of leaning pedestrians and of bikes. They therefore need to withstand a nominal load of 1.4kN per meter run.

4.7 Other Load Effects

The bridge will be subject to many other loadings which may affect the design of the bridge. The nominal loads associated with these loads can be found in specialist literature.

Due to the fact that the bridge is made from reinforced concrete means it could suffer from shrinkage, creep and stress relaxation of its steel tendons. These problems have the potential to severely affect the strength of the bridge. For example, creep can reduce the elastic modulus of the concrete bridge deck by a factor of 3.

5 Strength

Now that the loadings on the bridge have been calculated, it is important to analyse the strength needed in the bridge to hold these loads. This paper will determine the bridge's strength in bending, the capacity of the cables, and the effects of compression in the deck.

To do this the worst case loadings must be calculated. These are found by combining the loadings in Section 4 as specified in Ref. [5]. This paper will use load combination 1 to analyse the structure which involves all dead and live loads. The factored loading of 'combination 1' is calculated in Table 3. This factored load acts across the whole main span as shown in Fig. 16.

<table>
<thead>
<tr>
<th>Type</th>
<th>Unfactored Load (kN/m)</th>
<th>γf1</th>
<th>γf3</th>
<th>Factored Load (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead</td>
<td>93.47</td>
<td>1.15</td>
<td>1.1</td>
<td>118.24</td>
</tr>
<tr>
<td>Super imposed</td>
<td>2.87</td>
<td>1.75</td>
<td>1.1</td>
<td>5.52</td>
</tr>
<tr>
<td>Live</td>
<td>16.94</td>
<td>1.5</td>
<td>1.1</td>
<td>27.95</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>151.71</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 16: ‘Combination 1’ loading on main span

5.1 Bending

The Sabo bridge deck is split into various sections which are supported in different ways. The first section of the bridge acts as a continuous beam from the West abutment through to pier 2. This section of the structure is a simple slab bridge and its strength will not be analysed in this paper. The third or main span is supported by the cable stays and will experience the highest bending moments on the bridge. Therefore it is important this section is analysed. The main span is fixed at pier 2 and is pinned at pier 3. This means its bending strength can be found in isolation from the rest of the bridge. This is shown in the form of a simplified moment diagram in Fig. 17. The loadings correspond to Table 3.

Figure 17: Simplified moment diagram of main span

However Fig.17 does not show the effect of the cables on the bridge. This is significant because the cable stays transfer moments to the pylon significantly reducing the maximum moment in the deck. The cable stays are elastic directional supports which means they move with the bridge deflections and are not restricted in any direction. This creates a bending diagram such as the one shown in Fig. 18. However due to the complexity of the supports it would take computer analysis to determine the maximum sagging or hogging moments in the bridge.

Figure 18: Final Moment diagram

5.2 Cable Strength

It is important to determine whether the bridge cable stays are suitable to carry the required loadings. This involves checking that the cross-sectional area of cable is large enough to take the tensional forces imposed on the cables.

Two cable stays support a section of bridge 6.55m long. Therefore the vertical load in each cable (Cv) can be found using loadings from Table 3 and Eqn. (14). The cables taking the most force will be the top cables because they have the shallowest angle of 20° to the horizontal. The tension force for this worst case scenario is found in Eqn. (15) and the minimum cross-sectional area for the cables is found in Eqn. (16). The ultimate strength of locked coil cables is taken to be 1570 N/mm² [3]. The actual diameter of the cables is 75mm and therefore the bridge cables are more than adequate to take the loadings on the bridge.
Compression \( \frac{9\, kN}{\sin 20°} = 1373.9\, kN \). Note that there are two
\[ \text{Horizontal forces will be imposed on the deck by the} \]
cables which will put the deck into compression. It is possible
that the deck may buckle due to these forces and therefore
it is important that these forces are evaluated. The horizontal
forces from each cable are different due to their varying angles
from the horizontal. For ease this conference paper will
assume that all cables act at an average angle of 31° to the
horizontal. Therefore the average compression imposed on the
deck at each cable anchorage is given in Eqn. (17). Note that there are
two cables for each anchor point. The maximum compression
(Eqn. (18)) will act just before the pylon and will cause a
stress (\( \sigma \)) in the deck of 2608.5kN as shown in Eqn. (19).
To cope with this stress the pier below the pylon has been
inclined towards the deck to aid in carrying the horizontal
compression loads.

\[
\text{Av. Compression} = \frac{496.9}{\cos 31°} \times 2 = 1159.4\, kN. \quad (17)
\]
\[
\text{Max. Compression} = 1159.4 \times 9 = 10434.6\, kN. \quad (18)
\]
\[
\sigma = \frac{\text{Compression}}{\text{Area}} = \frac{10434.6}{4} = 2608.5\, kN. \quad (19)
\]

Figure 19: Deck Compression

6 Serviceability

The serviceability of the bridge is very important
because as pedestrians use the bridge they will be very
aware of bridge movements. This includes the oscillating
of the structure due to wind. If the bridge exceeds its
serviceability limit state then it will be unsuitable for use
regardless of its structural integrity. This paper will
evaluate the effects of vibrations on serviceability.

6.1 Vibrations

Vibrations can have serious consequences on bridges
especially if the bridge is forced to vibrate at its natural
frequency. On footbridges vibrations are a key concern
due to the phenomenon of ‘Synchronous Lateral
Excitation’ [9]. This occurs when chance correlation of
footsteps cause a sway in the bridge. As more people start
to walk in sequence, the worse the sway of the bridge.

To analyse vibration effects on the Sabo Bridge, first
its natural frequency \( (f_0) \) is found using Eqn. (20). The
bridge is assumed to be simply supported between pier 2
and pier 3. The natural frequency of the Sabo Bridge is
under 5Hz which is due to the slenderness of the bridge
deck. This means it’s maximum vertical acceleration (a)
also needs to be evaluated and checked that it is restricted
to 0.5\( f_0 \) m/s². Equation (21) shows that the maximum
acceleration is indeed under the require3d value and
therefore the Sabo Bridge is adequately designed to
withstand vibrations.

\[
f_0 = \frac{C^2}{2\pi^2} \sqrt{\frac{Elg}{M}} \quad (20)
\]
\[
= 3.66 \times 10^{-4} \times \sqrt{375.43 \times 10^3} = 0.224\, \text{Hz}.
\]
\[
a = 4\pi^2 \times f_0 \times y_s \times k \times \psi. \quad (21)
\]
\[
y_s = \frac{PL^3}{48EI} = \frac{0.7 \times 65.5^3}{48 \times 30 \times 10^6 \times 0.1229} = 0.0011\, \text{mm}.
\]

Table 4: Values for Eqn. (20, 21)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>65.5 m</td>
</tr>
<tr>
<td>I</td>
<td>60\times 10^5 kN/m²</td>
</tr>
<tr>
<td>g</td>
<td>9.81 m/s²</td>
</tr>
<tr>
<td>M</td>
<td>96.34 kN/m</td>
</tr>
<tr>
<td>( y_s )</td>
<td>0.0011 mm</td>
</tr>
</tbody>
</table>

7 Construction

Construction of the Sabo Bridge followed 8 main
stages (Fig. 23) which are detailed below.

The first stage involved the construction of the
bridge’s piers, abutments and the earth anchorages (for
the cables). This started by installing piles into the ground
to the required depth as stated in Section 3.4. Next the
piers, abutments and rock anchorages were constructed by
pouring concrete into wooden formwork. Steel bars link
the elements to the piles. It was important at this stage to
test the strength of the rock anchorages and make sure
they would be able to take the proposed cable tensions.

The second stage began with the erection of
falsework between the piers. Gaps were left in the
temporary framework to allow vehicular traffic to pass
through on the highway. It was very important that the
falsework was suitably protected from vehicular impacts,
so barriers were erected and a minimum clearance of 5m
between the highway and framework was provided. Bridge
bearings were installed and then the concrete
superstructure was cast in-situ.

Figure 20: Photograph showing falsework
The third stage involved allowing the concrete to set and removing the falsework from spans that did not require the cables for structural support.

The fourth stage saw the erection of the steel pylon using a crane to lift the pylon in place. Next two of the steel cables were attached and stressed. These cables balanced the pylon at its inclined angle and therefore it was important that the crane did not release the pylon until the cables were stressed. Installation of the pylon could not be performed if wind speeds exceeded 30mph.

Stage 4 involved the installation of the back stays between the pylons and earth anchors. These cables were then stressed sequentially from bottom to top. It is important to note that left and right cables were stressed simultaneously so as not to induce lateral moments in the pylon.

Stage 6 involved the installation of cables between the pylon and the deck. These cables were then stressed from bottom to top. The left and right cables were stressed simultaneously to minimize moments in the pylon. As the cables were stressed, they began to take the loading of the deck. This meant the superstructure lifted off of the framework in certain locations.

Stage 7 began with the removal of the falsework under the main span. This allowed the cables to take the full weight of the deck. The cables were then re-stressed using jacks by a specified force to counteract deflections in the deck. This ensured that tension in the cables was enough to hold the deck under and up to its fully loaded capacity.

Stage 8 of construction required applying the finishing non-structural elements to the bridge. The parapets were set in place, concrete surfacing applied, pylon screens fixed in place, and the bridge lighting elements inserted.

The method used for constructing the Sabo bridge had many advantages. The use of in-situ concrete method allows the changing cross-sectional area of the deck to be formed on site. This also meant that no joints were produced within the bridge spans, which makes the bridge substantially stronger. This method is also suitable for post tensioning which means when the cables are re-stressed the deck does not crack. Also no cranes were needed for the construction of the deck. This was important because some areas of the bridge were inaccessible to cranes due to overhead cabling.

However there were also many disadvantages to the construction method. Mainly the use of falsework to construct the deck is a problem because it is expensive and time-consuming to build. It also took up space on the highway which caused restrictions to vehicular traffic and had economic consequences. Also using in-situ concrete for the deck meant that the concrete had to be left to cure before it was loaded which slows down construction. Another problem with using in-situ concrete is that construction is dependent on climate conditions. These can affect the rate of curing and even the strength of the concrete. Despite the disadvantages to the construction method, the Sabo Bridge was actually completed two months ahead of schedule. This would have saved the project a lot of money.
8 Durability

The durability of the Sabo Bridge relies on the steel and concrete components of the bridge and how they react with time and usage. With the concrete in the bridge deck it is essential that the correct mix of constituents are used, that it is compacted correctly and that it is allowed to cure. To ensure this occurred the concrete mixes were tested for strength and also the pouring of concrete occurred on a day with no rain. It is also important to ensure there is sufficient cover to all reinforced bars. These measures should ensure that deterioration due to chloride attacks and carbonation are less likely to occur.

The steel components of the rest of the bridge such as the cables, pylon and abutments also need to be durable. To protect the cable-stays an anticorrosive product with a good bond and service life was applied. For cables attached to the bridge deck, the first 2m length was covered by a steel tube which protects them from fire, explosion and also vandalism. To protect the pylons stainless steel screens are used. To protect the abutments they are coated in anti-corrosive paint.

Inspections of the bridge should be made and documented regularly by experts. Inspection of the deck and piers is simple due to the fact that the structure is on show. This means any cracks or corrosion can be seen with the naked eye. The pylon can also be checked by dismantling the protective screens.

9 Vandalism

Vandalism and crime on the bridge are concerning issues for the Sabo Bridge. The Midtown Greenway pedestrian route has attracted crime and there are even reports of armed robbery. There is also a fair amount of graffiti along the route. It was therefore important to prevent these incidents occurring on the Sabo Bridge.

To accomplish this, a comprehensive security system was established around the bridge. A collection of security cameras were installed to catch vandals on film. These cameras were made visually obvious with the aim of preventing vandals from offending in the first place. Also lighting on the bridge and the approach spans was made a high priority. The bridge is clearly illuminated at night which makes the route appealing to its users and at the same time deters vandals. Emergency phones were also installed on the bridge for the use of its users in times of need.

10 Future Changes

Future changes to the Sabo Bridge are likely to be minimal. This is due to the fact that the bridge is purely designed for pedestrian use. Therefore any significant changes would require a major redesign of the bridge both aesthetically and structurally. For example if the bridge were to become used for vehicular traffic several changes would need to be made. Firstly the width of the deck would need to be widened as currently the bridge could only fit 1 lane of traffic. This would increase torsional loading of the deck. Also the strength of structural elements would need to be tested under vehicular live loads. This may require substantial changes to the deck cross-section and pylon. This would change the bridge’s slender aesthetics and would all be very expensive.

Possible changes to the bridge would involve changes to components with superimposed dead loads. These include installation of benches, new parapets, lighting, security devices (e.g. cameras) and new deck surfacing. These minor changes may slightly affect aesthetics of the bridge; however the structural integrity would not be compromised. This is due to the load factors imposed during design which anticipate minor changes to the bridge.

11 Conclusion

This paper has achieved the aim of critically analysing the Martin Sabo Bridge. The use of a cable stay design allows the structural components of the bridge to become key aesthetic features. The assumptions of loadings upon the bridge and the subsequent strength checks showed that the structural elements of the bridge do not need to be large or deep. This allowed bridge components, such as the pylon and the deck, to take on slender forms. The resulting aesthetic effect is that of a sleek dramatic crossing which has become a landmark for the Midtown Greenway route.

12 Acknowledgments

Acknowledgements go to M. Evernden for lectures on Bridge Engineering, to T. Ibell for his guideline book on Bridge Engineering, and to PDM Bridge Ltd. for their help with researching information on the Sabo Bridge.

13 References