Optimization of Steel Arch Rib Lateral Bracing

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ABSTRACT

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This project examines the use of the stress design ratio to optimize the lateral bracing between arch ribs. Literature review on the design of bracing between arch ribs is given. Using the design processes set out, an excel worksheet was modified to account for the brace sizing throughout a long span arch bridge. These values were then checked against the actual design of the Lupu Bridge.

Keywords: stress design ratio, arch rib lateral bracing, Lupu Bridge.
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INTRODUCTION

Arch bridges have been constructed since about 1300 BC, but it wasn’t until the Roman Empire that they truly were utilized in a meaningful way. During that time those bridges were used to build roadways and aqueducts. Due to the nature of the arch bridge those structures were able to span longer distances than previous bridges with limited material use and last a long period of time. In fact many of the bridges built during that time are still in use today.

The arch bridge works by building the structure in such a way that the majority of it is in compression and the tension forces are negligible. This makes concrete and masonry great materials for construction as they are both strong in compression. However, due to the material properties, the span length of an arch was limited to about 40 meters. To overcome this limitation viaducts, or a bridge composed of several small spans, were used.

To span even greater distances in modern construction steel has replaced concrete and a truss style bridge is utilized instead of a solid structure. Using these methods spans up to 552 meters has been achieved. However, by using the trusses between arch ribs, it becomes more susceptible to out-of-plane buckling.

This project examined steel arch bridges for the out-of-plane buckling, and their need for lateral braces between the steel arch ribs. Research on the different loads that affect the lateral bracing was examined and are presented in this project report. An excel worksheet was created to optimize the amount of steel needed for these braces using the stress ratio method. These results were then compared with the actual design of the Lupu Bridge.
2 LITERATURE REVIEW

In preparation of carrying this project several articles and books were read on the design process for the lateral bracing between arch ribs. The following is a brief review of this literature.

2.1 Out-of-Plane Buckling of Solid Rib Arches Braced with Transverse Bars

In this article Tatsuro Sakimoto and Yoshio Namita explored a numerical analysis of the buckling load caused by the out-of-plane buckling in a solid circular arch rib bridge. Using the transfer matrix method general equations the out-of-plane buckling were derived dependent on the flexural rigidity of the arch ribs and the transverse bars, the number of transverse bars, and the end conditions of the arch ribs. The numerical analyses were then compared to results from a model test and were found to be within 90% of the experimental values. In addition to these results it was found that the number of transverse bars wasn’t as important as the location of these braces.

2.2 Ultimate Strength of Steel Arches Under Lateral Loads

In this article Tatsuro Sakimoto and Sadao Komatsu explained the study of lateral braces in increasing the structural stability of an arch bridge to lateral loads. In the traditional design method lateral bracing is based on simple beam theory and treated independently of the design of
the arch ribs. This article presented a method to design these systems together, and simplified numerical method to estimate forces on the lateral bracing. These methods and numerical analysis were designed and valid for a through type two-hinged, circular solid steel arch bridges of uniform depth. Model tests were done and compared to these numerical results. These test found that the proposed numerical analysis provide a conservative estimate of the combined vertical and lateral forces on the bridge. Surprisingly, the model tests also found that the shear force due to wind was almost double of conventional design methods.

2.3 Numerical approach to the lateral buckling of steel tied-arch bridges

In this article H. De Backer Outtier and Ph. Van Boaert discusses how finite element models of several steel tied-arch bridges were created and used to calculate the lateral buckling strength of steel tied-arch bridges. To model a more real world application several out-of-plane imperfections were included in the finite element model of the bridge. Based on the calculated values, the resistance to out-of-plane buckling was calculated. The results of the calculations were compared to the existing buckling curves for straight beams, which are generally used in the European Nations. From this comparison it was found that out-of-plane buckling of arches is less critical than when calculated using buckling curves from a straight beam as per the code in these nations. The article also elaborated in the lack of consensus on how to deal with the lateral buckling of arch bridges.

2.4 Key Technology for Design of Lupu Bridge

Yue Guiping details many of the design aspects of the Lupu Bridge in Shanghai. The design plans of the bridge are laid out and discussed. The key aspects of the design process of the bridge both for construction and operation of the bridge are discussed.
2.5 *Arch Bridges from Steel Designer’s Handbook*

Arthur W. Hedgren, Jr. discusses many of the design aspects for steel arch bridges. Details on what needs to be considered for the design of the deck, arch rib, tie, hangers, deck lateral bracing, and rib lateral bracing are presented. Example designs are also provided using the LRFD design method.

2.6 *Arches from Guide to Stability Design Criteria for Metal Structures*

Ronald D. Ziemian presents the considerations for the structural design of arch for both in-plane and out-of-plane stability. For both types of stability design standards, structural analysis, and numerical methods are discussed. The differences that need to be considered for different type of arches are discussed and general design methods to account for these differences are given.

2.7 *Arch Bridges*

In this article Douglas A. Nettleton emphasizes aspects of arch bridge design such as wind stress analysis and deflection, stress amplification due to deflection, consideration of rib shortening moments, plate stiffening, and calculations for preliminary design. The article discusses the three major types of arches in use today, mainly steel, concrete, and covered arches. In the steel arch bridges a discussion is given on how to calculate the loads from winds as well as the preliminary design methods. Examples of the design methods are then given.
3 FORCES ACTING ON BRIDGE

In the design process it is first necessary to identify and estimate the forces acting on the structure. When designing lateral bracing in bridges two types of forces usually control the design: wind and seismic forces. However, these two forces are assumed to act separately. In general design practice this means that one type would be assumed to control and after designing against that type the other force would be checked. For this project only the wind force was considered and no check was done in regards to the seismic design. This decision was made as wind loads are generally considered to control in long span bridges (Barker, 2007).

In conjunction with the wind or seismic force it was also necessary to consider a nominal force that accounts for the buckling of the arch ribs, as well as the gravity force of the braces themselves.

3.1 Wind Loads

To estimate the wind loads the procedures in ASCE 7 were followed as stated in section 6.5.3. Usually a wind speed is estimated based on geographic location; for this project an assumed velocity of 178 mph was used. All other factors were assumed to be 1. Then a wind pressure, q, was found using the equation:

\[
q = 0.00256 \times k_z \times k_{zt} \times k_d \times V^2 \times I
\] 3.1
Where $V$ is the wind speed, $I$ is the importance factor, $k_z$ is the height factor, $k_{zt}$ is the topographic factor, and $k_d$ is the directionality factor.

Using this wind pressure acting on the rib was found at every brace point by multiplying this pressure by the depth of the arch rib and the average distance between brace points. A wind reduction coefficient was considered to account for truss style ribs compared to solid ribs.

To find how the wind forces act on the braces first the shear force on the arch rib is found by modeling the arch as a simply supported beam as shown in Figure 1. Where $V_1, V_2,$ etc are the wind forces at the brace points. The shear force at any point along the arch rib is then the summation of all wind forces to the half-span point minus any of the wind forces up to that point.

![Figure 1 Shear Force Acting on Arch Rib](image)

### 3.2 Buckling Load

The buckling load is an assumed nominal force that is estimated to deal with the arch ribs buckling. When an arch rib begins to buckle part of the vertical force from the rib, hangers, struts, and other components of the bridge then have a lateral component due to the deflection.
The best method to estimate this force is still under debate and dependent on the type of arch bridge (De Backer 2007).

Many methods have been developed to estimate the buckling load on the bracing between the arch ribs. These methods depend on the shape of the arch, parabolic or circular, the rib depth, the incline of the arch ribs, the type of arch, and the foundation conditions. Though this project mainly looked at the Lupu Bridge in comparison where one of the numeric methods could be utilized, the goal was to make the equations used be universal enough to be used on several bridges. Therefore, this project utilized the 2% rule, which states: “Bracing systems shall be proportioned to have strength perpendicular to the longitudinal axis of the braced member in the plane of buckling equal to 0.02 times the factored compressive force at each brace point in the member being braced, unless a detailed analysis is carried out…” (AASHTO 2012). Though this method can be universally applied to many bridges its major drawback is that it is considered highly conservative, and is usually not used in long span bridges as a more accurate load force can be found. The buckling force was then found using:

\[
F_{buckling} = 0.02 \times F_{comp} \tag{3.2}
\]

Where \(F_{buckling}\) is the buckling force and \(F_{comp}\) is the compressive force in the arch rib.

### 3.3 Gravity Load

The gravity load is to account for the self-weight of the brace. As the brace is secured between 2 arch ribs it is modeled as a fixed-fixed beam. The equation from the moment caused by gravity, \(M_{grav}\), is then:

\[
M_{grav} = \frac{Ay^2}{12} \tag{3.3}
\]

Where \(A\) is the area of the brace, \(\gamma\) is the density of steel, and \(d\) is the length of the brace.
4 BRACING TYPES AND ANALYSIS

Though there are many types of braces that can be utilized, this project concentrated on box girder braces. This was done as box girder braces are one of the most common brace types used in arch bridges and previous assumptions that exist in the excel worksheet could continue to be used. The brace is modeled as shown in Figure 2, where all the area of the brace is assumed to be in the four corners of the brace that are connected by webs of negligible size.

These braces are then considered to be laid out in two different patterns. The first pattern is vierendeel, or transverse bars, as shown in Figure 3. The second pattern is a diagonal truss that is also shown in Figure 3.

The depth of the brace is assumed to be a function of the arch rib depth. A reduction factor is multiplied by the arch rib depth to calculate the brace depth.
4.1 Wind Force Acting on the Brace

Knowing the shear on the arch rib it is possible to solve for the force acting on the brace between arch ribs. From mechanics the force is:

$$F_{\text{wind}} = \tau s b \quad \text{(4.1)}$$

Where $\tau$ is the shear stress, $s$ is the spacing between brace points, and $b$ is the width of the brace. $\tau$ can be found using the shear formula:

$$\tau = \frac{VQ}{lb} \quad \text{(4.2)}$$

Where $V$ is the shear at any point, $Q$ is the first moment of a plane area, $I$ is the moment of inertia, and $b$ is the width of the brace. The moment of inertia is solved using:

$$I = \frac{Ah^2}{2} \quad \text{(4.3)}$$

Where $A$ is the area of the brace, and $h$ is the distance between the two arch ribs. The first moment of a plane area is:

$$Q = \frac{Ah}{2} \quad \text{(4.4)}$$

Using these other values the shear stress can be written:

$$\tau = \frac{2VAh}{2Ah^2b} = \frac{V}{hb} \quad \text{(4.5)}$$

The force in the brace from the wind can then be written as:

$$F_{\text{wind}} = \tau s b = \frac{V}{hb} s b = \frac{Vs}{h} \quad \text{(4.6)}$$
This force is then used in determining the stress ratio but is used differently depending on the bracing pattern.

4.2 Vierendeel Bracing

The vierendeel bracing is treated like finding the stress in the web of an I-beam. The brace is modeled as shown in Figure 4. As the force from the wind acts on both ends of the brace in opposite directions a couple is created. To resist this couple a moment at the end of the brace is created and is calculated using:

\[ M_{\text{wind}} = \frac{h}{2} \cdot \frac{v_s}{h} = \frac{v_s}{2} \]  \hspace{1cm} 4.7

Where \( V \) is the shear force at that brace point, and \( s \) is the spacing between braces.

To optimize the brace, the stress ratio method is used. The stress ratio is solved using the following:

\[ \sigma_{\text{ratio}} = \frac{F_{\text{buckling}}}{A} \cdot \frac{M_{\text{wind}} + M_{\text{grav}}}{\frac{At^2}{2}} \]  \hspace{1cm} 4.8

Where \( A \) is the area of the brace, \( t \) is the depth of the brace, and \( \sigma_{\text{allow}} \) is the allowable stress.

The stress ratio is then multiplied by the original area to find the new area of the brace. This is continued until the stress ratio is equal to 1.

Figure 4 Vierendeel Model
4.3 **Diagonal Bracing**

The diagonal bracing is modeled as shown in Figure 5. The force in each member from the wind is then:

\[ F_{wind} = \frac{Vs}{d} \]  \hspace{1cm} 4.9

Where \( V \) is the shear at that brace point, \( s \) is the spacing between braces, and \( d \) is the length of the brace.

To optimize the brace, the stress ratio method is used. The stress ratio is solved using the following:

\[ \sigma_{ratio} = \frac{F_{buckling} + F_{wind} + M_{grav}}{A} \frac{M_{grav}}{t} \frac{1}{\sigma_{allow}} \]  \hspace{1cm} 4.10

Where \( A \) is the area of the brace, \( t \) is the depth of the brace, and \( \sigma_{allow} \) is the allowable stress.

The stress ratio is then multiplied by the original area to find the new area of the brace. This is continued until the stress ratio is equal to 1.

![Figure 5 Diagonal Bracing Model](image-url)
5 **COMPARISON WITH LU PU BRIDGE**

The Lupu Bridge is a steel arch bridge built in Shanghai, China. The main span of the bridge is 550 meters long and was the part of the bridge that was investigated. Using the modified spreadsheet values were compared to the actual design of the Lupu Bridge. Comparisons to several parts of the bridge are made to give an overall accuracy.

5.1 **Design Values**

All values that were used in the spreadsheet tried to reflect the actual values that were used in the design. Where actual values could not be found, best estimates were used. As the brace depth is roughly half of the depth of the arch rib, the brace/rib ratio was assumed to be 0.5. The other values used as well as the design layout are given in Appendix A.

5.2 **Spreadsheet Values to Actual Values**

The actual area of the arch rib, the force in each hanger, the force in the ties, and the max and minimum brace size areas were found and compared to the spreadsheets values (Guiping, 2008). A summary of the comparisons is shown in Table 1. Screenshots of the whole spreadsheet are given in Appendix B.
Table 1 Comparison of Actual Values to Spreadsheet Values

<table>
<thead>
<tr>
<th></th>
<th>Actual Value</th>
<th>Spreadsheet Value</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Arch Rib Area (m^2)</td>
<td>2.2</td>
<td>1.97</td>
<td>10.5</td>
</tr>
<tr>
<td>Force in hanger (KN)</td>
<td>1245</td>
<td>1184.33</td>
<td>4.87</td>
</tr>
<tr>
<td>Force in tie (KN)</td>
<td>199280.32</td>
<td>203701.36</td>
<td>2.22</td>
</tr>
<tr>
<td>Max Brace Area (m^2)</td>
<td>0.24</td>
<td>0.203</td>
<td>15.4</td>
</tr>
<tr>
<td>Min Brace Area (m^2)</td>
<td>0.18</td>
<td>0.019</td>
<td>89.4</td>
</tr>
</tbody>
</table>

For the most part the values obtained from the spreadsheet area are fairly close to the actual values. However, there are some parts of the bridge that aren’t accounted for that might explain the error. On the top of the arch ribs lies a viewing platform, as well as the lighting system over the bridge. The weight from these would increase the overall dead weight as well as live load put on to the arch ribs as well as the braces. As a check when a higher load is placed on the ribs the error decreases. The only area which is not impacted in any meaningful way is the minimum brace area. The high error here is most likely due to a minimum amount of steel that is required in the braces by code. No code reference to the minimum required area for braces was found and was therefore not included in the spreadsheet.

5.3 Conclusions

The modifications to the spreadsheet were successful in finding roughly the area of the maximum brace size. It was also successful in being able to be implemented to several types of bridges that will be investigated using the spreadsheet. However, it was unsuccessful in determining the minimum brace size. Further research on the minimum area of steel required would need to be investigated for the area of the bridge, as well as any other source of error.
REFERENCES


## APPENDIX A. DESIGN VALUES AND LAYOUT

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-Strength Wires</td>
<td>allowable stress (KPa)</td>
<td>1180000</td>
</tr>
<tr>
<td></td>
<td>density (KN/m^3)</td>
<td>77</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>allowable stress (KPa)</td>
<td>247000</td>
</tr>
<tr>
<td></td>
<td>young's modulus (Kpa)</td>
<td>200000000</td>
</tr>
<tr>
<td></td>
<td>density (KN/m^3)</td>
<td>77</td>
</tr>
<tr>
<td>Load Data</td>
<td>wind pressure (KPa)</td>
<td>3.88</td>
</tr>
<tr>
<td></td>
<td>wind pressure reduction coeff</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>lane load (KN/m)</td>
<td>19.5</td>
</tr>
<tr>
<td></td>
<td>number of lanes</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>deck dead load (KPa)</td>
<td>3.59</td>
</tr>
<tr>
<td></td>
<td>brace weight (Kpa)</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>brace depth / rib depth</td>
<td>0.5</td>
</tr>
</tbody>
</table>

There are 30 pairs of hangers spaced at 13.5 m intervals from plans assuming 2 struts on each side at 36.5 meters spacing.

39.5 wide and 3m high bridge deck road surface is 28.7 m wide.

The overall height of the web of arch rib has a transition from 9m at the skewback to 6m at the arch crown.
APPENDIX B. SCREEN SHOTS OF SPREADSHEET

The actual spreadsheet is held by Dr. Balling, and can be accessed by request to him.