RECENTLY DESIGNED BOW-STRING RAILWAY BRIDGES IN SLOVAKIA

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Abstract: The paper deals with newly designed arch bridges on Slovak railway lines. The three bow-string bridges across rivers have been designed on the basis of Eurocodes. The orthotropic steel deck was applied in all cases. Main characteristics and dimensions of the bridges are given in the paper. The bridges are to be built on the main railway line in Slovakia during its reconstruction for maximum speed 160 km/h. To analyze behavior of the bridges, spatial transformation models were developed using FEM software. Global analysis and verification of arches under buckling is discussed in more detail. The variation of top bracing system and its influence on out-of-plane buckling of the arches is presented. A minimum load amplifier to reach the elastic instability of arches is used to estimate the suitability of the bracings. An aesthetic aspect of various types of top bracings is discussed, as well.
1 INTRODUCTION

The main railway lines in Slovakia, which have been included into European railway corridors, have been under reconstruction and modernization during last decade. In this context, our department has participated in design and processing of project documentation of several railway bridges. Given the nature of the railway line, the majority of these bridges are of small and medium spans [1]. But, there are also several bridges, which are dominant due to their spans or used technology [2]. Among such bridges, the bridges over the Nosicky Water Canal, over the Vah River and over the Bela River could be undoubtedly included. The common feature of these bridges is that a rigid beam reinforced by an arch, so called the bow-string arch, or in central Europe known as Langer’s beam, is applied as their main load-bearing system. Some of the bridges mentioned above have already been presented [2], [3]. In this paper, the main attention is paid to global analysis and verification of the arches under buckling and the associated system of bracing arches.

2 DESCRIPTIONS OF BRIDGES

2.1 Bridge over the Nosicky Canal

The first bridge is situated at km 159.038 of the railway line Bratislava – Zilina and it will cross the Nosicky Water Canal directly behind the railway station Puchov. The bridge is designed as a four-span double-line steel railway bridge with theoretical lengths of single spans 62.4 m + 124.8 m + 124.8 m + 62.4 m (Figure 1).

The steel superstructures of two internal spans 2 and 3 of 124.8 m theoretical length consist of two bow-string girders with the bottom orthotropic bridge deck and the top longitudinal bracing. The plate beams are designed from passable box-section with external dimensions of 1350 × 3100 mm. The circular curved arches with theoretical rise of 22.0 m are centrically connected to the beams above the supports and they are also made of passable box-sections with internal dimensions of 1220 × 1710 mm. The vertical hangers are designed from steel tubes R 273/20 mm, filled with concrete C 20/25 in order to increase dynamic resistance to the transverse vibrations.

The steel superstructures of two side spans 1 and 4 of 62.4 m theoretical length are similar to the previous one. The plate beams also have the passable box-section with external dimensions 1000 × 2410 mm. The beam depth increases above the piers to the value 3070 mm due to an aesthetic transition to the main beams in internal spans 2 and 3. The circular curved arches with theoretical rise of 11.0 m are connected to the beams above the supports with the eccentricity of 460 mm and they are also made of passable box-sections with internal dimensions of 1220 ×
1710 mm. The vertical hangers are designed from steel tubes R 219/17.5 mm, filled with concrete C 20/25. The more detailed description of the whole bridge can be found in [3].

2.2 Bridge over the Vah River in Trencin City

The second bridge was designed as an alternative proposal to a reinforced concrete seven-span continuous bridge in the phase of preliminary design of the bridge. The bridge carries a relocation of the railway line with the design speed of 160 km/h over the Vah River and adjacent inundation area in Trencin City. The bridge is located immediately below the confluence of the Vah River and the Nosicky Water Canal. The new bridge will replace the original railway steel truss bridge, which is located about 100 m downstream. Specified length of bridging as well as an effort to minimize construction works in the bed of the Vah River led to the choice of a four-span bridge structure under each railway line with theoretical lengths of single spans of 84.0 m (Figure 2). The horizontal alignment of bridging is at an angle of 82° to the river bed, which also represents the horizontal slant of the superstructures. The steel superstructures consist of two bow-string girders with the bottom orthotropic bridge deck and the top longitudinal bracing. The plate beams placed at axial spacing of 6.25 m have a constant cross-section of an unsymmetrical welded I shape with total depth of 2.78 m. The parabolic curved arches with theoretical rise of 11.0 m are connected to the beams with the eccentricity of 1000 mm above the supports and they are made of closed box-sections with outer dimensions of 640×760 mm. The vertical hangers are designed from the symmetrical I-shaped welded cross-sections of 500 mm height. The more detailed description of the whole bridge can be found in [2].

![Figure 2: Longitudinal view of the bridge over the Vah River](image)

2.2 Bridge over the Bela River in Liptovsky Hradok City

The third bridge is situated directly behind the railway station Liptovsky Hradok of the railway line Kosice – Zilina and it will cross the Bela River, right-side inflow of the Vah River. The bridge is designed as a double-line steel railway bridge with theoretical span length of 66.0 m (Figure 3). The steel superstructure consists again of the bow-string girders with the bottom orthotropic bridge deck and the top longitudinal bracing. The plate beams are designed from passable box-section with external dimensions of 1000 × 2030 mm. The circular curved arches with theoretical rise of 12.0 m are connected to the beams above the supports with the eccentricity of 400 mm. The cross-sections of arches are also made of passable box-sections with internal dimensions of 1000 × 1134 mm. The vertical hangers are positioned in the tenths of span and they are made of circle cross-section of diameter Ø 110 mm. The bridge cross-section is closed by top longitudinal bracing.
The deck plate 16 mm thick is shaped into the profile of ballast bed channel with vertical webs. Flat longitudinal stiffeners located at distances of 460 mm are designed from plates $25 \times 250$ mm. Transverse stiffeners are arranged at distances of 2200 mm and they have a variable cross-section of an inverted T shape, consisting of the web plates $14 \times 1216$ mm at the lowest point of the bridge deck and the bottom flange of $30 \times 320$ mm. The end transverse stiffeners are strengthened because of the potential embedding presses for lifting up the superstructure to replace structural bearings, if necessary.

3 INFLUENCE OF BRACING SYSTEM ON ARCH STABILITY

Firstly, it is usually necessary to optimize the main dimensions, especially to find the optimal ratio of arch rise to its length. Then, the configuration of top longitudinal bracings is varied to find bracing systems, which primarily fulfils stiffening function and feasibility. If any, aesthetics demands are taken into account as well. After the preliminary design and optimization process, more complex static analysis, stability analysis and dynamic analysis should be performed.

3.1 Variation of top bracings

In order to analyze the influence of top longitudinal bracing on stability of the arches, a simple parametric study was performed. Different types of the bracing system were
incorporated into computational models of all four aforementioned railway bridge superstructures. The first comparative model was considered without any top bracing system (I). Then, two basic types of top bracing systems were taken into account: the frame system (II) with varying number of cross-beams and the truss system (III) with various arrangements of diagonals. All the considered bracing systems are summarized in Figure 4. In all cases, the bracing members were designed of circular hollow sections. In case of truss bracing system the slenderness of members did not exceed the value of 150.

3.2 Stability analysis

Using of software based on Finite Elements Method (FEM) allows for spatial behavior of bridges. Considering the size of the structures, the beam finite elements were used for modeling main girders, arches, hangers and top bracings. Steel plates of the deck were meshed by shell finite elements. The plates were stiffened by the ribs in the longitudinal and transversal direction, approximating the longitudinal and transversal stiffeners of the decks. Considering the actual structural details, the arch-to-beam joints were considered as rigid. The connections of hangers were approximated by the hinge joints, except for hangers of welded I-cross-section of a single track railway bridge of 84.0 m long span, where the joint was modeled as rigid in out-of-plane bending.

Implementation of both the imperfections and the second order effects into global analysis was found as a very time-consuming procedure because of full geometry models. Therefore, it was decided to take into account the imperfections of the arch in the global analysis and to allow for the second order effects during verification process. Buckling effects can be then included by introduction of equivalent buckling lengths, corresponding to a global eigenmode of the loss of structural stability, especially of the arches. In that case, the first order analysis may be used to find the factor \( \alpha_{cr} \), by which the design load should be increased to cause the elastic instability in a global mode.

The critical buckling length \( L_{cr,z} \) of arch member for buckling about \( z \) axis can be then obtained from the well-known equation:

\[
L_{cr,z} = \pi \sqrt{\frac{E \cdot I_z}{\alpha_{cr} \cdot N}} = k_z \cdot L_{arch}
\]  

(1)

The unknown parameters in the equation (1) not mentioned hereinbefore are: \( E \) - the steel modulus of elasticity; \( I_z \) - the second moment of area about \( z \) axis of the arch cross-section; \( N \) - the value of normal force in the arch element, for which the stability is analyzed; \( k_z \) - the coefficient of buckling length for buckling about \( z \) axis; \( L_{arch} \) - the theoretical length of arch axis.

Similarly, the influence of deformed geometry, so called "the second order effect" can be expressed by means of multiplying horizontal bending moments in arches determined using the first order theory by the factor \( k_{II} \), according to equation:

\[
k_{II} = \frac{1}{1 - 1/\alpha_{cr}} = \frac{\alpha_{cr}}{\alpha_{cr} - 1}
\]  

(2)

The equation (2) is authorized by Eurocode 3 [4] only for values of amplifier \( \alpha_{cr} \) higher than 3.0. In-plane buckling of the arches is controlled by tension hangers. This was also confirmed by stability analyses. Moreover, second order effects in vertical direction could be regarded
as inessential, because the amplifier to reach the elastic in-plane instability \( \alpha_{cr} \) was far above the value of 10.0 in all four cases of arch structures.

### 3.3 Results comparison

The results of stability analyses of four superstructures stiffened by all aforementioned types of bracing systems are presented and compared in Table 1. Denotation of the particular bracing systems is shown in Figure 4. The arch stability is significantly affected mainly by the span-to-width ratio of the bridge superstructure. Based on results from analyzed arches, the ratio of about 5.0 provides the value of amplifier \( \alpha_{cr} \) greater than 3.0 even on using non bracing system. As could be expected, the frame bracing system is generally less effective than the truss one. In the case of narrow bridges with span-to-width ratio greater than 10.0, the frame system appears to be ineffective (\( \alpha_{cr} < 3.0 \)).

The comparison in Table 1 outlined dominant role of rigid model of arch-to-girder connection. Actually, the out-of-plane buckling lengths of non braced arches could be considered under the one third of the theoretical arch length for all analyzed arches.

When using the truss bracing, the rhombic system seems to be the most effective. On the contrary, the K-truss system is the least effective, although only small differences were observed. Regardless of the span and the truss type, all coefficients \( k_z \) of critical buckling lengths of arches for buckling about \( z \) axis were within the interval from 0.167 to 0.205.

Nevertheless, in accordance with [3], the second order effects should be taken into account in the global analysis when amplifier \( \alpha_{cr} \) is less than 10.0.

<table>
<thead>
<tr>
<th>Span of the bridge [m]</th>
<th>62.4</th>
<th>66.0</th>
<th>84.0</th>
<th>124.8</th>
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<td>Number of railway lines</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Span-to-width ratio</td>
<td>5.07</td>
<td>5.28</td>
<td>13.44</td>
<td>9.90</td>
</tr>
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<td>box girder</td>
<td>box girder</td>
<td>plate girder</td>
<td>box girder</td>
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<tr>
<td>Arch-to-girder connection</td>
<td>rigid</td>
<td>rigid</td>
<td>rigid</td>
<td>rigid</td>
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<table>
<thead>
<tr>
<th>Top bracings of the arches</th>
<th>( \alpha_{cr} )</th>
<th>( k_z )</th>
<th>( \alpha_{cr} )</th>
<th>( k_z )</th>
<th>( \alpha_{cr} )</th>
<th>( k_z )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without bracing</td>
<td>( \alpha_{cr} )</td>
<td>( k_z )</td>
<td>( \alpha_{cr} )</td>
<td>( k_z )</td>
<td>( \alpha_{cr} )</td>
<td>( k_z )</td>
</tr>
<tr>
<td>One in the middle</td>
<td>I</td>
<td>4.57</td>
<td>0.31</td>
<td>3.83</td>
<td>0.31</td>
<td>1.53</td>
</tr>
<tr>
<td>In every 2nd hanger</td>
<td>IIa</td>
<td>5.06</td>
<td>0.30</td>
<td>4.25</td>
<td>0.30</td>
<td>1.90</td>
</tr>
<tr>
<td>In every hanger</td>
<td>IIb</td>
<td>5.25</td>
<td>0.29</td>
<td>4.43</td>
<td>0.29</td>
<td>2.36</td>
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<td>Frame bracings</td>
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<tr>
<td>In every 2nd hanger</td>
<td>IIc</td>
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<td>0.28</td>
<td>4.48</td>
<td>0.29</td>
<td>2.75</td>
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<td>K - truss</td>
<td>IIIa</td>
<td>12.61</td>
<td>0.19</td>
<td>8.82</td>
<td>0.20</td>
<td>3.05</td>
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<td>9.18</td>
<td>0.20</td>
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<td>Rhombic truss</td>
<td>IIIc</td>
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* adopted solution, when stiffening diagonals are added on both sides of arches to the portal bracing

Table 1: Calculated values of the factors \( \alpha_{cr} \) and \( k_z \) for various bracing systems

Calculated values of the amplifier \( \alpha_{cr} \) from Table 1 are graphically compared in Figure 5. It is clearly seen that, in the case of shorter spans, the use of truss bracing would significantly stabilize arches. If the span becomes longer, the difference between frame and truss bracing is less evident. Especially, in case of the bridge with span of 84.0 m, the truss bracing can be thought as useless provision. Basically, this superstructure represents a narrow bridge and, moreover, with plate girder of I-shaped cross-section.
3.4 Discussion of aesthetic look of bracings

Top bracing of bow-string arch bridges is placed on visible part of the bridge and usually plays a dominant role in appearance of the bridge. In Figure 4, various arrangements of bracing in the case on the longest span superstructure (124.8 m) are compared.

Figure 5: Comparison of minimum load amplifiers $\alpha_{cr}$ to reach the elastic instability

Figure 6: Aesthetic look of bracings; denotation - see Figure 4 and Table 1
Based on human nature, many opinions on the best solution would be certainly found. As could be expected, the truss bracing seems to be less attractive. If the frame bracing is applied, it can be stated that the less cross members are installed the more beautiful appearance is achieved. The mission is if the question: "Which is the nicest bracing?" is the most accurate in the case of the railway bridge not placed just in the middle of a big city. If the ability to stabilize arches is taken into account, the question can be probably turned into: "Which bracing with high level of stiffness has still acceptable look?"

Finally, the concept of frame bracing in every second hanger (variant IIb) was adopted for the bridges shown in Figures 1 and 3, where the rigid joint of box girder to box arch play very important role in the arch stability. Steel tubes of R 610/16 mm are designed for the frame bracing of 66.0 m long superstructure of the bridge over the Bela River (Figure 3). Similarly, steel tubes of R 508/20 mm, will be installed as the frame bracing of 62.4 m long superstructures of the bridge over the Nosicky Canal in Puchov City (Figure 1). The cross-sections of the two 124.8 m long middle spans are closed by upper steel tubes of R 914/25 mm. Moreover, to increase the stability and to push up amplifier \( \alpha_{cr} \) over the value 3.0, both portal cross-braces are reinforced with a pair of diagonals from tubes of R 610/20 (variant IIb*). In the case of superstructures given in Figure 2, the K-truss bracing would be probably installed. Anyway, in the case of this bridge a reinforced concrete seven-span continuous bridge won the competition of preliminary designs.

4 CONCLUSIONS

Three newly designed bow-string arch bridges were briefly introduced and stability of the arches was discussed in the paper. Comparison of the results obtained from the stability analysis of four alternatives of superstructures shows that additional to the system of bracing, some other parameters are important, as well. The stiffness of joint between the arch and main girder seems to be also essential. A stiffer box-to-box connection of arch-to-girder joint can notable increase the arch stability in comparison with the arch, which is fixed to a common plate I-girder. The span-to-width ratio has significant influence, as well.

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REFERENCES