Problems and innovative solutions considering “open type” bridge deck structures in old steel riveted railway bridges

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Abstract There are 331 steel bridges with total length 11201,70m in exploitation in the railway infrastructure of the Republic of Bulgaria. Significant part of those bridges is with “open type” deck including longitudinal and cross beams. The stress state of a standard bridge deck of an old steel riveted truss bridge along Sofia - Varna railway line at km 17+536 (representing wide class of bridges in exploitation) is analysed in this paper in accordance with the prescriptions of the Eurocode design standard. Results, obtained from different FEM (with Frame and Shell elements) indicate that the load bearing capacity of longitudinal and cross beams is significantly insufficient, which is due to increased loads and some specificity of the construction details. On the other hand there is an overage of load bearing capacity for the elements of the main truss and the effect of an innovative concept for replacement of the existing “open type” deck with new reinforced concrete plate is investigated and analyzed.

1. Introduction.
There are 331 steel bridges with total length 11201,70m in exploitation in the railway infrastructure of the Republic of Bulgaria. Significant part of those bridges are riveted with “open type” deck. Old steel riveted bridges were designed in accordance with the Admissible Stresses Method which is much conservative in comparison with the Limit States Method in the present design standards. Thereby there is a reserve in the load bearing capacity of the elements which can compensate the increased live loads [3], [4]. Series of investigations shows that the remaining fatigue life of the main girders of part of these bridges can be prolonged by adequate rehabilitation and strengthening which leads to the possibility of rational time distribution of the restricted funds for maintenance by the responsible institutions – NRIC (National Railway Infrastructure Company) [1], [2], [4]. At the same time because of the details specificity and the increased values of the horizontal live loads serious problems with the load bearing capacity of the elements of the “open type” decks of the old riveted railway bridges are registered [3].

At the Figure 1 are presented cross sections of a typical old riveted truss railway bridge. The bracing system is connected only with the 2L profiles, forming the upper chord of the longitudinal beam, the out of plain inertia moments are very low. This leads to high stresses and insufficient load
bearing capacity for the noising force effects. As it is described in [3] in spite of the conservative design standards and material reserves a serious deficiency of load bearing capacity of the deck members, dangerous for the exploitation of the bridge, is registered.

![Figure 1. Bridge deck cross section (left) and part of deck plan (right – original drawing) of an old steel riveted bridge with truss main girder along Sofia-Varna railway line at km 17+536 and 31m span length.](image)

2. Numerical example.
In order to analyze the quantitative aspect of the problems with “open type” bridge decks described in point 1 a two different FEM of the old riveted bridge from Figure 1 in SAP2000 - M1 and M2 are developed – Figure 2. The elements of the main truss girder in both variants are modeled as beam (Frame) finite elements with rigid connection in joints. In model M1 longitudinal girders are modeled as beam (Frame) finite elements (FE), simply supported in vertical direction on the cross girders because the structural connection is realized only between webs. In model M2 the elements of the bridge deck are modeled as Shell FE considering the correct geometry and connection between webs which leads to semi-rigid connection between longitudinal girders and cross girders. In model M2 the connection between the cross girders and the verticals of the main riveted truss considers the real zone of connection by adjusting the mesh of finite elements of the Shell web of cross beams and Frame elements of the vertical of main truss girder – Figure 3.

![Figure 2. FEM models - M1(Frame) and M2(Shell).](image)
Both models M1 and M2 are presented on Figure 4 and Figure 5. The bracing system for the noising force is connected only with the upper 2L profiles, forming the midsection of longitudinal girder and cross girders obtained from the analyzed superstructure; railway track, sidewalks and parapets; classified vertical loads; and horizontal (noising force \( L_n \), braking inertia force \( Q_{bh} \), and acceleration inertia force \( Q_{bb} \)) live loads. The characteristic maximum values of edge stresses in midsection of longitudinal girder and cross girders obtained from the analyzed loads and effects in both models M1 and M2 are presented on Figure 4 and Figure 5. It must be mentioned that higher values for edge stresses in longitudinal girder are obtained from noising force. It is expected because the bracing system for the noising force is connected only with the upper 2L profiles, forming the upper chord of the longitudinal girder and considering the negligible torsion stiffness the noising force is transferred by the out of plane bending of those 2L profiles.

![Figure 3](image.png)

**Figure 3.** Connection between cross beams and the vertical of the main truss in the model M2.

Most unfavorable load combinations of loads and effects in accordance with [5] and reduced in accordance with [6] for bridges in exploitation, used in Germany – self weight \( G_s \) of the steel bridge superstructure; railway track, sidewalks and parapets; classified vertical (LM71 and SW/2, in accordance with BDS EN 1991-2) and horizontal (noising force \( L_n \), braking inertia force \( Q_{bh} \), and acceleration inertia force \( Q_{bb} \)) live loads. The characteristic maximum values of edge stresses in longitudinal girder are obtained from noising force. It is expected because the bracing system for the noising force is connected only with the upper 2L profiles, forming the upper chord of the longitudinal girder and considering the negligible torsion stiffness the noising force is transferred by the out of plane bending of those 2L profiles.

**Figure 4.** Stresses from characteristic values of the analysed loads and effects for longitudinal girder from models M1 and M2.

**Figure 5.** Normal stresses from characteristic values of the considered loads and effects presented for cross beams for model M1 and M2.
The percent difference between the characteristic normal edge stresses in longitudinal and crossbeam are presented obtained from the considered loads for model M1 (Frame) and model M2 (Shell) are presented in Table 1. The differences for noising force are negligible, but from the classified vertical load LM71 are significant. The reason probably is that the connection between crossbeam and the vertical of the main truss in model M1 is realized in a particular node and in model M2 is realized in a zone with height equal of the height of the crossbeam – Figure 3. In the longitudinal beam as it is expected for the midsection m the normal stresses are higher in comparison with these obtained from M2, because the elastic stiffness of the connection between the webs of longitudinal beam and crossbeam is not considered in model M1. The normal stresses in the support section of the longitudinal beam are caused by the interaction with the main truss girder. It is seen from Table 1 that because of the specific connection detail between the upper bracing system and the longitudinal beam the normal stresses caused by noising force are much higher than those obtained from the classified vertical loads UIC71 as it was mentioned above.

**Table 1.** Maximal edge normal stresses for various effects

| Longitudinal Beam | Transverse Beam |
|-------------------|-----------------|
| [N/mm²]           | [N/mm²]         |
| m, LM71           | m, LM71         |
| 113               | 135             |
| 93                | 75              |
| 22.01             | 79              |
| m, LM71           | m, LM71         |
| 50.60             | 44.01           |
| 51.51             | 146.85          |
| 1.80              | 2.34            |
| m, Lb, s          | m, Lb, s        |
| 296.85            | 297.00          |
| 0.05              |                  |

In Table 2 are presented the most unfavorable values of the utilization factor \( \eta = \sigma_{Ed} / f_{yd} \) \( \sigma_{Ed} \) - design value of the edge normal stress from the most unfavorable load combination, calculated in models M1 and M2, \( f_{yd} \) - design value of the yielding stress) for the edges of the cross sections of the longitudinal beam midsection (m) and support (s) and cross beam midsection (m) in accordance with loads defined in [5] and the decreased loads in [6] (by means of decreasing \( \gamma_0 \) and \( \alpha \)).

**Table 2.** Utilisation factor \( \eta \) for members of the bridge deck

| Model M1: FRAME - \( \eta_1 \) | Model M2: SHELL - \( \eta_1 \) |
|-------------------------------|-------------------------------|
| \( y=1.45; \alpha=1.21 \)     | \( y=1.45; \alpha=1.21 \)     |
| \( y=1.3; \alpha=1.00 \)      | \( y=1.3; \alpha=1.00 \)      |
| **LONGITUDINAL BEAM - m**     | **LONGITUDINAL BEAM - m**     |
| 4.03                          | 3.73                          |
| 3.12                          | 3.40                          |
| **LONGITUDINAL BEAM - s**     |                                |
| 1.67                          | 1.74                          |
| 2.84                          | 2.93                          |
| **TRANSVERSE BEAM - m**       |                                |
| 2.48                          | 2.26                          |
| 1.97                          | 2.17                          |

It is seen from Table 2 that in accordance with [5] the load bearing capacity of the longitudinal beam is \( \sim 3.0÷4.0 \) times lower than the standardized design minimum and for the cross beam it is \( \sim 2.0÷2.5 \) times lower. In the case of rehabilitation and strengthening of old riveted railway bridges in relatively good condition of the main girder and considering such a significant load bearing capacity deficiency for elements of the deck solutions for replacement of the “open type” old deck must be analyzed.

3. **Variant for replacement of the old steel deck of “open type” with reinforced concrete, connected for composite action with elements of the bridge.**

The developed detailed design [3] shows that significant part of the members of the old steel riveted bridge described in point 2 has sufficient load bearing capacity in respect to defined in [5] loads and effects even in the case of the replacement of the existing steel riveted deck with reinforced concrete plate (with thickness 200÷230mm and concrete C45/55), connected for a composite action with the upper chord of the old riveted truss bridge – Figure 6. It must be mentioned that the increasing of the
self-weight is significant. The self-weight of the old bridge in present condition is 24,24kN/m (including elements of the structure and railway track with timber sleepers) and after replacement of the deck with reinforce concrete plate and the self-weight becomes 64,4 kN/m. If UHPFRC (Ultra High Performance Fiber Reinforced Concrete) is used for the new deck plate the unfavorable effect of self-weight increasing can be reduced [3]. Variant for 150mm thick prestressed in transverse direction UHPFRC (with characteristic compression strength $150 \text{N/mm}^2$ and modulus of elasticity $50000 \text{N/mm}^2$ [7]) deck plate is developed. The weight of this 150mm UHPFRC deck plate is about 50% lower than a 200-230mm thick RC plate from C45/55 (29,7kN/m for the standard RC and 19,7kN/m for the UHPFRC plate), which is favorable in respect to the elements of the main truss girder, bearings and the substructure of the bridge. The self-weight of the bridge structure after replacement the deck with 200-230mm thick RC - C45/55 is 64,4kN/m and for transversely prestressed UHPFRC plate is 54,3kN/m (about 18% lower self-weight of the superstructure).

![Figure 6. Cross section of the bridge in variant with reinforced concrete plate C45/55(left) and prestressed in transverse direction plate with 150mm height of UHPFRC with $f_{ck}=150\text{N/mm}^2$ [7] (right) connected for composite action with the upper chord of the truss.]

4. Conclusion

In series of cases the old steel railway bridges can be rehabilitated to correspond the present exploitation and design standards and this is a solution more rational than replacement. In this case the mentioned before serious problems with the deck structure must be solved.

The simplified static models, used for old riveted bridge analysis, are in most cases with sufficient accuracy for the main truss girder elements can lead to significant inaccuracy for the elements of the deck – longitudinal beams and crossbeams. The bridge deck of “open type” of an old riveted truss bridge along the railway line Sofia – Varna at km 17+536 is analyzed. The connection between the riveted members of the deck is complex and cannot be correctly modeled in 3D FEM with beam (frame) elements (in both variants – fixed and free connection) – M1. For comparison a 3D FEM in which the members of the deck are modeled with Shell elements and members of the main truss are modeled with Frame elements is developed – M2. Serious differences between the results for models M1 and M2 is registered for the elements of the deck. The reason is that in the model with Shell elements M2 the connection between cross beam and vertical of the main truss and the connection of longitudinal beam and cross beam are more correctly approximated in comparison with model M1. In spite of this both models shows that the load bearing capacity of the old riveted deck elements is insufficient in accordance with the present European design standard [5] and even in comparison with the reduced demands of [6], used for assessment of old bridges in Germany.

In series of cases (represented by the analyzed in this paper old riveted railway bridge) considering the specificity of the details and the increased values of the defined in Eurocodes horizontal longitudinal and transverse live loads a serious deficiency of the load bearing capacity of the deck...
elements is evaluated (in the particular case 3,0÷4,0 times for longitudinal beams and 2,0÷2,5 times for cross beams). In spite of the conservativeness of the defined in Eurocodes horizontal live loads in the relatively rare case of heavy trains passing with high speed local deck destruction or plastic deformations can occur with all related unfavorable consequences.

In [3] and in the present paper the variant of an old riveted deck of “open type” replacement with reinforced concrete plate connected for composite action with elements of the old bridge is analyzed. In spite of the serious self weight increase concerning the composite action the load bearing capacity of the main girder can be satisfied with relatively low strengthening needs. Using UHPFRC (with characteristics in accordance with [7]) for the new transversely prestressed plate the increase in the self weight is 18% lower. This solution is optimal with respect to the bearings and the substructure.

The problem with the load bearing capacity of riveted “open type” bridge decks is serious and must be analyzed for wider range of old riveted bridges in order to minimize the risk of potential failures by means of in time adequate rehabilitation and strengthening.

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