ANALYSIS AND COMPARISON OF EUROCODE AND SNIP TRAFFIC LOAD MODELS FOR RAILWAY BRIDGES

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Abstract. The traffic loads and consequently the load effects in the bridge structures determined according to the Eurocode (EN) have been found to be substantially different from the values obtained by the SNiP code model used in Lithuania. For highway bridges the SNiP loads are lighter (sometimes 1.5-2.0 times), for railway loads they are considerably heavier. It was decided to investigate whether railway bridges designed to EN and SNiP load models have sufficient capacity and how to achieve good agreement between the load effects determined according to both codes. In this article the results of comparison for simply-supported bridge decks with span lengths ranging from 2 to 80 m. are presented. Rail traffic load models from EN and SNiP are analysed. The objective of the study was to modify the Eurocode rail traffic load factors to fit them to the SNiP loading provisions and to determine the consequences for bridge superstructures, if the bridges were designed according to the code SNiP and assessed according to the Eurocode. It is shown that the actions imposed on bridges of 10-40 m length of spans (that is common in Lithuanian practice) due to rail traffic according to the EN code are 20-40% lower than those determined according to the SNiP. The EN code gives even five traffic load models in comparison with only one load model in the SNiP. However the effects of load model SW/2 are sometimes lower than those calculated by LM71. The recommendations are presented how to adjust the loading models for both codes in order to insert the specifications of loads on bridges in new national Annex actually in preparation for Lithuanian standard LST EN 1990/A1:2006 lt.

Keywords: railway bridges, traffic load models, Eurocode, SNiP, flexural moments, shear forces, recommendations.

1. Introduction

The railway transport forms an important part of a whole transportation system of each country. Lithuania is being crossed by Trans-European Rail Corridors connecting Baltic countries via Warsaw to the rest of EU as well as Lithuania with Byelorussia, Ukraine and Russia becoming a part of major Trans-Asian corridors (Vasiliauskas and Barysien 2008). The main problem facing the railway bridges is that national and EU territory railway networks operate to different track and load standards.

Actual traffic loads on bridges are quite complex because of the randomness of traffic process. Normally, they are represented in the codes by simple load models, reflecting the worst loading that can be caused on the bridge by actual and future traffic. Bridge loadings described in codes of practice vary from country to country (e.g., O’Connor and Shaw 2000). It is evident that there has been a trend towards the increase in design loads and loading configurations over the years in most countries. Problems occur when different codes or standards are combined together.

In Lithuania many existing bridges were designed using codes SNiP (CнiП). Design of road bridges according to Eurocodes started from 2000. Lithuanian standards LST EN 1991-2:2005 lt and LST EN 1990/A1:2006 lt were issued which give rules and methods for establishing combinations of actions for road and railway bridges. Preliminary analysis and comparison of traffic load effects determined according to SNiP and first pre-standards ENpr at that time showed that the Eurocode loads for existing highway bridges are heavier sometimes up to 1,5-2 times (Notkus 1998, 2005) and for railway bridges are considerably lighter (Notkus 1998, 2003; Cypinas 2003, Cypinas et al. 2005). This means, for example, that old highway bridges need repair and strengthening that is not always reasonable. The same situation is observed in other countries.
Comparison of national rail traffic loads which have been found to be significantly different in the different codes (Gu et al. 2006; Heiden et al. 2003; O’Connor and Shaw 2000) to the EN provisions is of vital importance. Some publications on this subject can be found (e.g., James 2003; Sieffert et al. 2006). Modernization of European rail network with the aim to increase axle loads up to 33 tones for freight traffic and the maximum speeds up to 350 km/hour for passenger traffic (Cho et al. 2010) is leading to the need to re-evaluate loading conditions for high-speed railway bridges. Note that there are significant cost benefits in the use of heavier and rapid trains.

In this paper some comparisons of the SNiP code and the Eurocode (EN) railway bridge loadings are presented in order to insert the specifications of loads on our bridges in new national Annex actually in preparation for Lithuanian standard LST EN 1990/A1:2006 lt.

2. Presentation of EN and SNIP traffic loads

In EN 1991-2 (EN 2003) five models of railway static vertical loading are given:
- Load Model 71 (and Load Model SW/0 for continuous bridges) to represent normal rail traffic on mainline railways;
- Load Model SW/2 to represent heavy rail traffic;
- Load Model HSLM to represent the loading from passenger trains at speeds exceeding 200 km/h;
- Load Model “unloaded train” to represent the effect of an unloaded train.

Load Model 71 represents the static effect of vertical loading due to normal rail traffic. The characteristic values given in Fig 1 shall be multiplied by a factor \( \alpha \), on lines carrying rail traffic which is heavier or lighter than normal rail traffic. The factor \( \alpha \) may be specified in the National Annex and shall be one of the following:

\[
\alpha = \{0.75; 0.83; 0.91; 1.00; 1.10; 1.21; 1.33; 1.46\}.
\]

Load Model SW/2 represents the static effect of vertical loading due to heavy rail traffic (Fig 2 and Table 1).

![Fig 1. Load Model 71 and characteristic values for vertical loads](image1)

![Fig 2. Load Models SW/0 and SW/2](image2)

### Table 1. Characteristic values for load Models SW/0 and SW/2

<table>
<thead>
<tr>
<th>Load model</th>
<th>( q_{vk}, \text{kN/m} )</th>
<th>( a, \text{m} )</th>
<th>( c, \text{m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW/0</td>
<td>133</td>
<td>15.0</td>
<td>5.3</td>
</tr>
<tr>
<td>SW/2</td>
<td>150</td>
<td>25</td>
<td>7.0</td>
</tr>
</tbody>
</table>

The effect of lateral displacement of vertical loads should be taken into consideration. The eccentricity of vertical loads for LM71 and SW/0 \( e \leq r/18 \) (\( r \) is distance between wheel loads), the ratio of wheel loads on all axis on one track 1.25:1.0 or 0.5556:0.4444 (section 6 of EN 1991-2 or LST EN 1991-2:2005 lt). Partial load factor \( \gamma_f \) is 1.45 for LM71, and 1.2 for SW/2 (Table A2.4(b) of EN 1990:2002/A1:2005).

The dynamic amplification factor \( \Phi_L \) which enhances the static load effects under load models LM71, SW/0 and SW/2 for track with standard maintenance is determined as follows:

\[
\Phi_L \leq \frac{2.16}{\sqrt{L_\phi}} - 0.2 + 0.73, \quad 1.0 \leq \Phi_L \leq 2.0, \tag{1}
\]

where \( L_\phi \) is the length specified for particular elements. For example, for simple supported main girders, \( L_\phi \) is the span length.

A static analysis of the bridge structures is carried out with the load models described above using the dynamic factor \( \Phi_3 \), partial load factor \( \gamma_f \), and \( \alpha \geq 1 \) (if required).

The rail traffic load of the SNiP is based on the load model CK14. The load is given as a uniformly distributed line load, with different values depending on the loading length and shape of influence line (different for bending moments and shears forces). The CK14 line loads are presented in SNiP (CHnP, 1984) (see Table 2). Partial load factor is taken as 1.3 for loading length 0, 1.15 for loading length 50 m. and 1.1 for loading length 150 m. Intermediate values are determined by interpolation. Wheel loads are uniformly distributed on the track rails (0.5;0.5).

### Table 2. Characteristic values for load model CK14 (Extract from Table 1 Annex 5 of CHnP 2.05.03-84)

<table>
<thead>
<tr>
<th>Length, m</th>
<th>( kN/m ) for V</th>
<th>( kN/m ) for M</th>
<th>Length, m</th>
<th>( kN/m ) for V</th>
<th>( kN/m ) for M</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>428</td>
<td>374</td>
<td>20</td>
<td>207</td>
<td>181</td>
</tr>
<tr>
<td>4</td>
<td>304</td>
<td>266</td>
<td>40</td>
<td>168</td>
<td>147</td>
</tr>
<tr>
<td>6</td>
<td>273</td>
<td>239</td>
<td>60</td>
<td>151</td>
<td>137</td>
</tr>
<tr>
<td>10</td>
<td>245</td>
<td>214</td>
<td>80</td>
<td>144</td>
<td>137</td>
</tr>
<tr>
<td>15</td>
<td>223</td>
<td>195</td>
<td>100</td>
<td>140</td>
<td>137</td>
</tr>
</tbody>
</table>
The dynamic factors of SNiP for steel deck is
\[
\begin{cases}
(1 + \mu) = 1 + \frac{18}{30 + \lambda}; \\
(1 + \mu) \geq 1.15.
\end{cases}
\tag{2}
\]
for concrete deck is
\[
\begin{cases}
(1 + \mu) = 1 + \frac{10}{20 + \lambda}; \\
(1 + \mu) \geq 1.15.
\end{cases}
\tag{3}
\]
where \(\lambda\) is the length of loading.

3. Analysis and comparison of traffics loads

The load models of EN ir SNiP have been compared according to maximum characteristic and design bending moments and shear forces on simply-supported bridge decks with span lengths ranging from 2 to 80 m. carrying single lane of rail traffic. For spans up to 50 m the girder decks are considered, for longer spans the truss decks are analysed. Bridge decks are composed of two girders or two trusses carrying only one track. Decks of steel bridges are located between the main trusses near the bottom chords. Comparison was performed including partial load factors, impact factors, and eccentricities when designing new bridges as is presented in section 2. The comparison is shown in Figs. 3, 4, 5, and 6. In these figures bending moments and shear forces are presented as the ratios between the bending moments (shear forces) due to EN and SNiP code live load models and to the load model CK14 for concrete beams \((M_{CK,con} \text{ or } V_{CK,con})\). In figures they are noted as the ratios of characteristic \(M_{L1}/M_{CK,con} \text{ and } V_{L1}/V_{CK,con}\) and design \(M_{d,L}/M_{d,CK,con} \text{ and } V_{d,L}/V_{d,CK,con}\) values.

As can be seen from presented figures, discrepancy of moments and shear forces determined according to the EN and the SNiP load models is significant.

![Fig 3. Comparison of deck characteristic flexural moments for different code traffic load models and deck span lengths](image)

![Fig 4. Comparison of deck design flexural moments for different code traffic load models and deck span lengths](image)

![Fig 5. Comparison of deck characteristic shear forces for different code traffic load models and deck span lengths](image)
Fig 6. Comparison of deck design shear forces for different code traffic load models and deck span lengths

Eurocode load model SW/2 seeks to model heavy rail traffic. As follows from figures, for deck spans 10-40 m, load model SW/2 only slightly influences load effects with regard to load model LM71: characteristic moment values are slightly higher, design moment values are even lower (approximately 15%), characteristic and design values of shear forces are approximately equal. For the bridge decks up to 10 m, the load effects are from 1.5 to 3 times lower than those calculated by SNIP loading. Hence, in most cases, the SW/2 heavy load model may not provide the required load carrying capacity of bridges for current and future rail traffic in Lithuania. As a result, load and speed restrictions may be imposed to prevent overloading of railway bridges. Inevitable some modifications on application of load model SW/2 for national bridges are needed.

4. Recommendations

The National Standards implementing Eurocodes may be followed by the National Annexes. They may include, for example, alternative load models, values of $\alpha$ and dynamic factors (see section 2). In an attempt to harmonise EN and SNIP traffic loads three alternatives were analysed.

Case 1. Adjusted values of the LM71 load model can be obtained using Eurocode factor $\alpha$, for example 1.1 or 1.3. However, constant value of $\alpha$ may lead to a large error because calculated action effects according to both codes were substantially different for different lengths of spans. Constant value of factor $\alpha$ may be used only for a given span length.

Case 2. Different values of $\alpha$ based on the length of span or the length of influence line could be used. However, the span lengths should be divided into intervals. The observed range of factor $\alpha$ for flexural moments was 0.9 to 1.1 for span lengths 1-20 m, 1.0 for spans 20-50 m, 1.1 to 1.33 for spans 50-70 m, and 1.33 for longer spans (see Fig 7). Unfortunately, these values do not fit for shearing forces; for span lengths 10-50 m, the discrepancy is of the order of 25-20% (Fig 8). On the other hand, in this case the SW/2 load model becomes not useful.

Case 3. The heavy rail traffic SW/2 model loads as well as those of LM71 were multiplied by the same partial load factor $\gamma_f = 1.45$ with the same lateral eccentricity of vertical loads $e = r/18$ (the ratio of wheel loads 0.5556:0.4444).
The comparison of load effects is shown in Fig 9 and Fig 10. As can be seen there is relatively a good agreement between the bending moments and shear forces imposed by the SW/2 design load and the design load used in Lithuania. The difference is in the range of 5% for design bending moments and 5 to 15% (for spans 7-25 m.) for design shear forces. Characteristic values of internal forces induced by SW/2 and presented in Fig 3 and Fig 5 would be increased of the order of 10% (influence of eccentricity). Hence, the proposed load adjustment is simple, based on consideration of national traffic regulations.

Conclusions

1. For concrete rail bridge spans ranging from 10 to 40 m., that is typical in Lithuania, bending moments and shear forces induced by the Eurocode LM71 and SW/2 load models are up to 40% lower than those determined according to SNiP.

2. The SNiP traffic load produces the highest values of bending moments and shear forces indicating that Lithuanian bridges designed for live load C14 have sufficient capacity to carry the Eurocode rail traffic loads.

3. Heavy rail traffic load model SW/2 for deck spans of 1-50 m only slightly influences load effects with regard to load model LM71.

4. For design of new railway bridges on Lithuanian railways with different from European track standards it is recommended to adjust the heavy rail traffic SW/2 load model for national application. New national Annex for Lithuanian standard LST EN 1990/A1:2006 It actually is in preparation. The rail traffic SW/2 model loads as well as those of LM71 should be multiplied by the same partial load factor $\gamma = 1.45$ with the same lateral eccentricity of vertical loads $e = r/18$ (the ratio of wheel loads $0.5556:0.4444$). Two design situations with the load models LM71 and SW/2 should be analysed and the most unfavorable case identified that will be used for the bridge design.

References


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