

## A SOIL-STEEL BRIDGE UNDER HIGH-SPEED RAILWAYS<sup>1</sup>

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This paper presents some recent research on railway bridge dynamics with application to buried flexible structures. Based on a combination of simulations and full-scale testing, current research indicates that a rather comprehensive numerical model is required to accurately describe the response from passing trains.

Key words: high-speed railway; dynamic soil-structure interaction; ballast acceleration; soil-steel composite bridges

### 1. INTRODUCTION

For railway lines with a design speed over 200 km/h, dynamic analyses of railway bridges are required, in order to assure interoperability with future high-speed trains. A set of dynamic design criteria are stipulated in the Eurocode EN 1990, where the vertical deck acceleration usually is the most important. These criteria are however intended for conventional slab- and beam like structures and are not directly applicable buried flexible steel structures (hereafter denoted SSCB for soil-steel composite bridges). Hence, there are today no regulations or guidelines regarding the dynamic behaviour of SSCB for high-speed railways.

The dynamic response highly depends on the properties of the soil, providing both interaction and load distribution with the corrugated steel profile. Further, wave propagation in the soil may potentially result in excessive vibrations of the track at higher train speeds, especially for smaller height of cover. There is currently no experience of SSCB for high-speed railways and it is therefore difficult to determine under which conditions they may exceed the serviceability criterions in EN 1990.

In this paper, results from full-scale testing is used when calibrating both 2D and 3D FE-models of a SSCB for high-speed trains.

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## 2. FULL-SCALE TESTS

The full-scale testing was performed in May 2010 and was reported in Andersson et al. (2012). The bridge is located in Sweden, about 40 km North of Stockholm, along the express line between Arlanda Airport and Stockholm Central Station. A view of the bridge during passage of a high-speed commuter train is shown in Figure 1. The allowable speed is 170 km/h and is operated by mixed train traffic. In the following simulations, the results are compared to a single passage with commuter train X52 on track U1. Details of the train is shown in Figure 2.



Figure 1. View of the bridge during an X52 commuter train passage

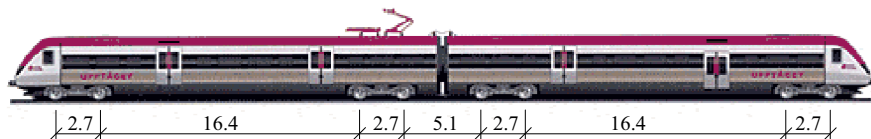


Figure 2. Detail of commuter train X52, axle load 175 kN

The closed elliptical steel culvert has a horizontal diameter of 3.75 m, vertical diameter of 4.15 m and a length of 27.9 m across the two-track railway. The fill height at the crown is 1.7 m.

The instrumentation is presented in Figure 3, consisting of displacement transducers d1 and d2 (LVDT) at the crown, accelerometers a1-a6 mounted both on the steel culvert and in the ballast, and strain gauges e1-e12 on the inner side of the steel culvert.

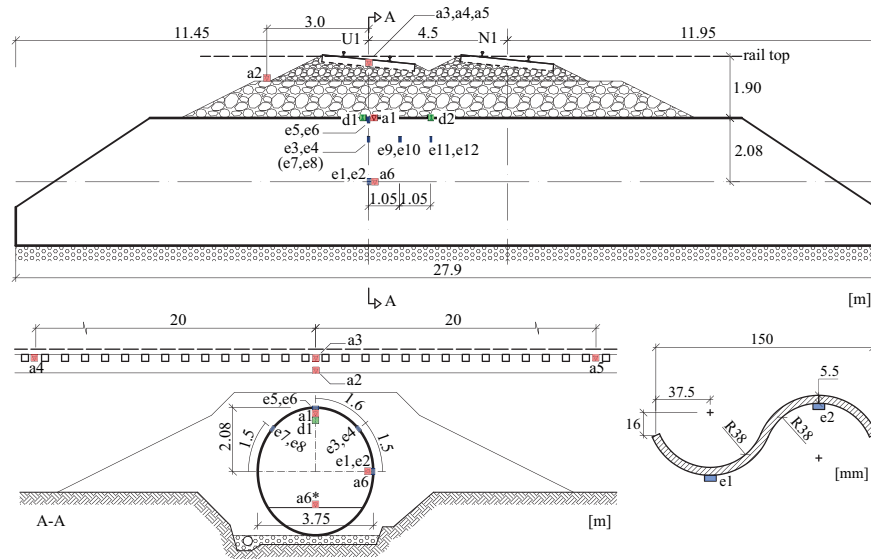


Figure 3. Instrumentation in cross-section, elevation along section A-A and details of the steel profile

### 3. SIMULATIONS

Models to predict the dynamic response of the bridge has previously been developed by Mellat (2012) and Mellat et al. (2014), comprising both 2D and 3D approaches. A parametric study of similar bridges using a 2D-approach was performed by Woll (2014). Additional studies was performed by Aagah & Aryannejad (2014).

A 2D FE-model of the bridge is illustrated in Figure 4, using similar approach as the authors mentioned above. The steel culvert is modelled with Euler-Bernoulli beam elements with cross-sectional properties based on the corrugation in Figure 3. The soil and ballast material as well as the sleepers are modelled with 4-noded plane stress elements. The UIC-60 rail is modelled with Euler-Bernoulli beam elements and connects to each sleeper with rigid links. The beam elements of the culvert is rigidly connected to the adjacent soil. So-called silent boundaries are facilitated, to mitigate reflecting waves in the soil. This is accomplished by assigning higher material damping. The total length of the model is 60 m. The main uncertainties in the 2D model are the E-modulus of the soil and effective width of the soil and the culvert. To better estimate the real load distribution, a 3D-model is created by extending the 2D-model in the transverse direction, see Figure 5. In the 3D-model, the soil and the sleepers are modelled with 8-noded solid elements and the culvert with 4-noded orthotropic shell

elements. In both models, the E-modulus of the soil is assumed constant, i.e. the depth dependent increase in stiffness is neglected.

The X52 train is modelled as vertical point loads on the rail, traversing the bridge in a dynamic analysis with direct time integration. The 3D-model is calibrated against the experimental data by adjusting the E-modulus of the soil. The best fit is found for  $E_{\text{soil}} = 120$  MPa. The same E-modulus is used in the 2D-model, but the effective width is instead calibrated to fit the experiments. Both models are calibrated by manual parameter updating, focusing primary on the vertical crown displacement. In the 2D-model, the effective width is 2.5 m for the sleepers and 3.0 m for the ballast. A constant effective width for the whole soil was used, best fit with the experimental results was found for 16 m width of the soil.

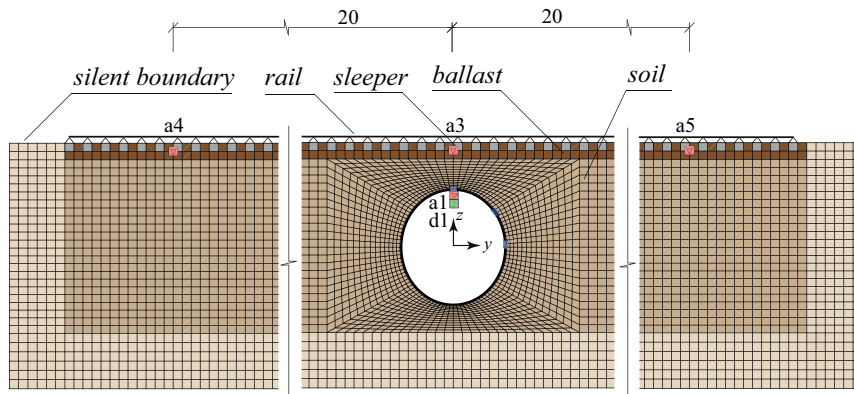


Figure 4. Detail of the 2D FE-model

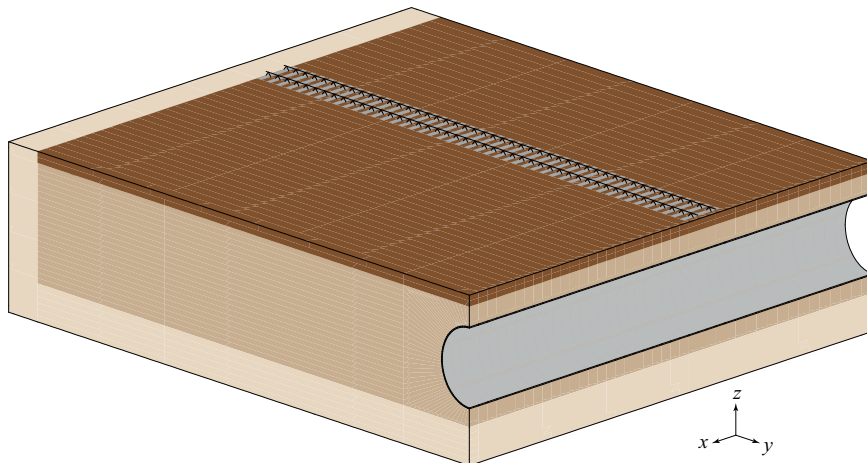


Figure 5. Slice of the 3D FE-model (half of the complete model)

The vertical crown displacement  $d1$  of the culvert under track U1 is shown in Figure 6. A fairly good match is found using both the 2D and 3D models, given the level of approximations regarding the soil.

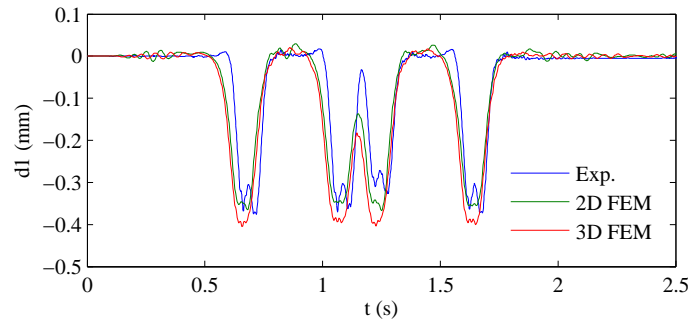


Figure 6. Vertical crown displacement  $d1$ , X52-train at 170 km/h

In dynamic analysis of bridges with ballasted tracks for high-speed railways, the vertical deck acceleration is usually limited to  $3.5 \text{ m/s}^2$ . This is an indirect measure of the risk of ballast instability, from lab experiments found to occur at about  $7 \text{ m/s}^2$ . For SSCB, it is assumed better to study the acceleration in the track rather than the steel culvert. The acceleration at point  $a3$  located on a sleeper at bridge midspan is presented in Figure 7. A good match is found between the experiments and the models. It should be noted however, that the response from the short train at the current speed is mainly governed by transient response of each axle and is not at resonance.

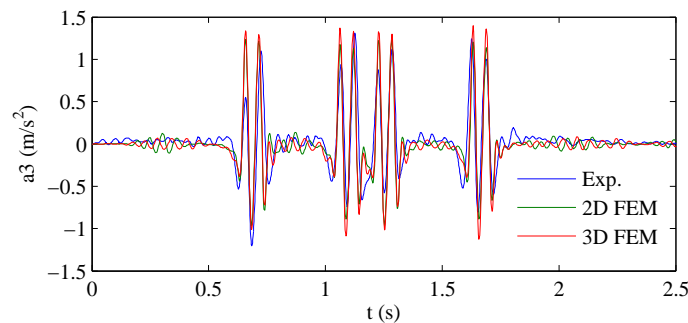


Figure 7. Vertical sleeper acceleration  $a3$ , X52-train at 170 km/h

The radial stress in the steel culvert is measured at both the bottom of the corrugation (odd numbers  $e1$  to  $e11$ ) and the top of the corrugations (even numbers  $e2$  to  $e12$ ). The results along the centre line of track U1 is shown in Figure 8 to Figure 10. Negative stress corresponds to compression. The stresses are over-estimated at the abutment using both the 2D and 3D models. A somewhat better

match is found at the haunch, especially at point e3. At the crown, the stress at point e5 is influenced by bending moment which is rather well described with the 3D model but not the 2D model.

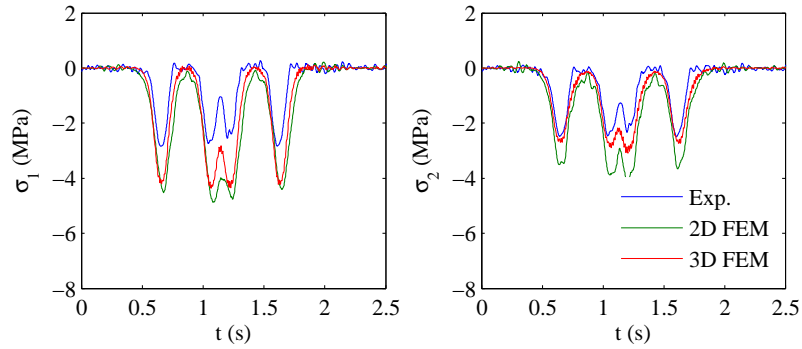


Figure 8. Stresses at the abutment, X52-train at 170 km/h

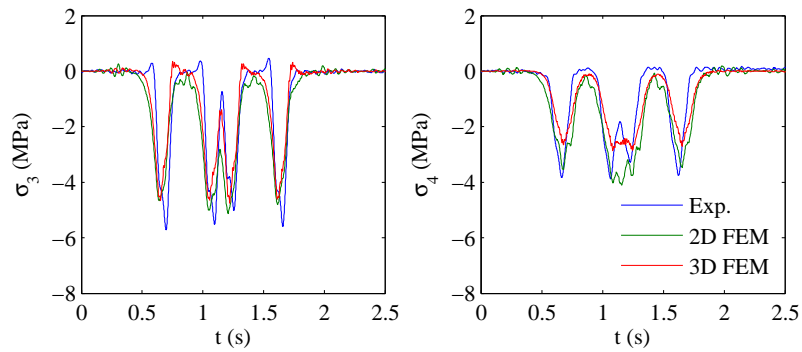


Figure 9. Stresses at the haunch, X52-train at 170 km/h

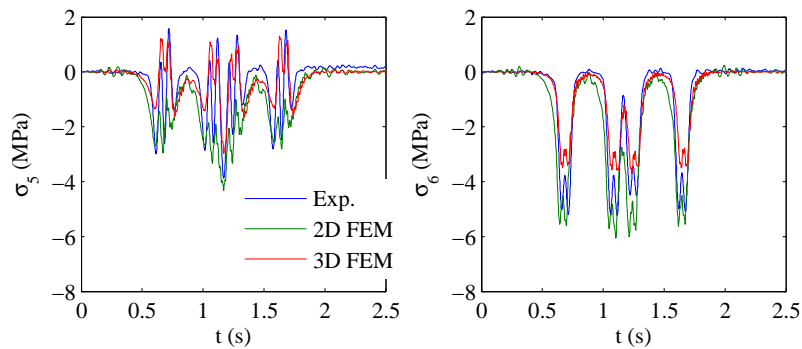


Figure 10. Stresses at the crown, X52-train at 170 km/h

The influence of load distribution in the transverse direction can be studied using sensor e9 to e12. A comparison between the experiments and the 3D-model is presented in Figure 11. A good match is found for most of the positions, indicating that the 3D model is able to accurately represent the stresses in the steel culvert.

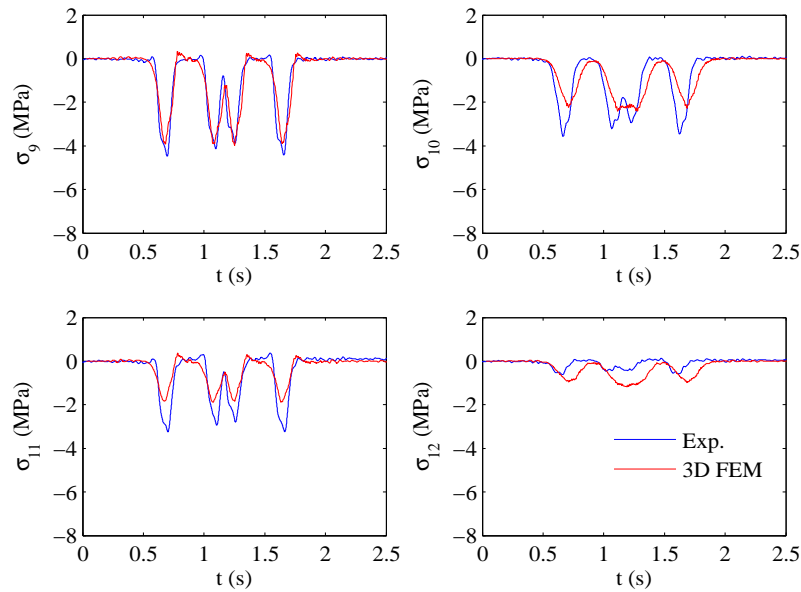


Figure 11. Stresses at point 9-12, X52-train at 170 km/h

As a theoretical study, the peak acceleration is calculated using the 2D-model and the train load model HSLM-A according to EN 1991-2. The results are shown in Figure 12. No clear resonance is obtained, but rather a slight increase in response with increased speed. The response is significantly higher at point a3 on the sleeper compared to a1 at the crown of the culvert. Point a4 and a5 located 20 m before and after the bridge, shows only marginally lower values compared to a3.

#### 4. CONCLUSIONS

Based on the results in this paper, the following is concluded for the studies bridge:

- Based on a 3D FE-model, an E-modulus of 120 MPa for the soil is found to give the best match compared to experiments of a passing train;

- Using the same E-modulus in a 2D FE-model, an effective width of 16 m is required, which is significantly larger than predicted by a 2:1 load distribution approach;
- The stresses in the steel corrugation is generally low and mainly in compression. A reasonably acceptable match is found using both the 2D and 3D models, although both over- and underestimations are obtained;
- Good match in track acceleration is found, although mainly covered by impact loading. Simulations of high-speed trains does not reveal any resonance-like behavior.

Future research regarding the dynamic response of SSCB should focus on a combination of experimental testing, preferably using controlled excitation, in combination with model updating. Other bridge configurations, e.g. with longer spans and shallow fill depth should be studied.

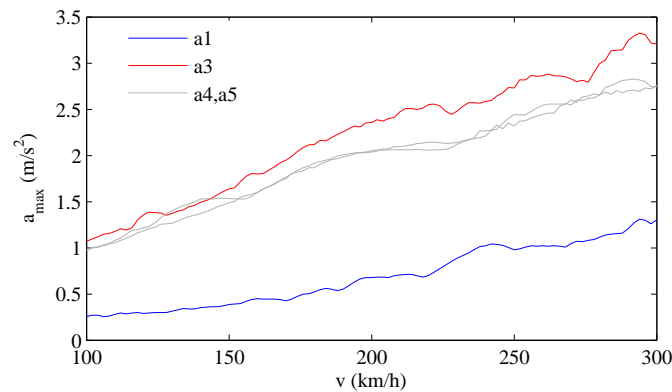


Figure 12. Envelope of peak acceleration from train model HSLM-A, 2D-model

## LITERATURE

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