

## FULL SCALE TESTS AND STRUCTURAL EVALUATION OF SOIL-STEEL FLEXIBLE CULVERTS FOR HIGH-SPEED RAILWAYS

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### Abstract

In this paper, results from full-scale tests on a corrugated soil-steel flexible culvert for railway traffic are presented. The bridge was instrumented with strain gauges, accelerometers and displacement gauges, measuring the response from passing trains. The aim of the measurement campaign was to gain knowledge of the dynamic behaviour caused by train induced vibrations, both of the bridge structure and the overlying railway embankment. From the measured data, the load distribution and soil-stiffness can be estimated. The results also serve as input for calibration of numerical models that are used for predicting the behaviour of high-speed trains.

Key words: Soil-steel flexible culvert, high-speed railways, field measurements, load distribution, ballast acceleration

## 1. INTRODUCTION

Design methods for corrugated soil-steel flexible culverts currently accepted by the Swedish Transport Administration are only valid for normal speed railway lines, up to 200 km/h. As for other bridges, dynamic effects are considered using dynamic amplification factors of the static response. For higher speeds, more advanced dynamic analyses are required, accounting for resonance and the risk of ballast disintegration. There are currently no regulations or guidelines for such analyses of soil-steel flexible culverts. To accurately envisage the behaviour of such bridges for high-speed railway lines, a proper model with a set of both static and dynamic parameters is needed.

Full-scale dynamic tests have been performed on an existing soil-steel flexible culvert for railway traffic. The studied bridge is not located on a high-speed line, as the maximum speed is around 170 km/h. Still, the results serve as initial

input for both static and dynamic properties in the development of suitable structural models. Strains, displacements and accelerations are measured and compared with their theoretically predicted counterparts. In addition, the measured steel strains and displacements give valuable information on the longitudinal and transversal behaviour of the culvert caused by true load distribution from passing trains.

## 2. FULL-SCALE TESTS

### The Bridge

The studied bridge is located in Sweden, about 40 km north of Stockholm. It is a two-track railway bridge designed as a closed elliptical corrugated steel culvert, with a horizontal diameter of 3.75 m and a vertical diameter of 4.15 m. The total length transversal to the tracks is 27.9 m and the fill height at the crown is 1.7 m. The bridge was built in 1995 during an extension of the north junction of the Arlanda Express line, a link between the East Coast railway line and Arlanda Airport. The allowed speed is 170 km/h and the line is used by mixed traffic, although mainly consisting of long-distance commuter trains. A photo of the bridge during passage of an X52 commuter train is shown in Figure 1.



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

### Instrumentation

The bridge was instrumented with a total of 12 strain gauges, 2 displacement transducers and 8 accelerometers. Continuous monitoring was performed be-

tween 9 AM and 16 PM on the 19<sup>th</sup> of May 2010. Data was collected using a HBM MGC-Plus A/D converter with 20 bit resolution. The sample frequency was set to 800 Hz and an analogue Bessel Low-Pass filter with a cut-off frequency of 400 Hz was employed to avoid aliasing.

The instrumentation is presented in Figure 2 to Figure 4 below. Accelerometers are denoted a1-a8, displacement transducers d1-d2, and strain gauges e1-e12. Most gauges were positioned directly under the track U1, Figure 2. To gain further information on the load distribution, however, d1 and e9-e12 was instrumented closer to track N1. Both displacement transducers measured vertical crown deflection. Accelerometers were positioned both on the arch intrados and on the railway embankment, a3-a5 was positioned between the sleepers of track U1, both 20 m in front of and behind the culvert as well as directly over the crown.

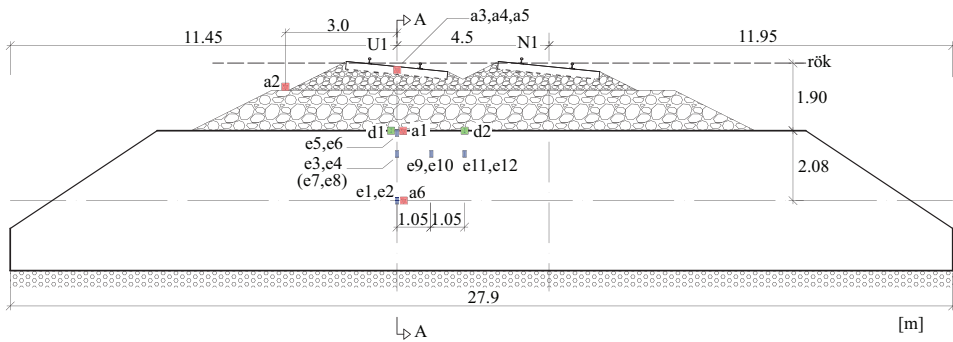


Figure 2. Instrumentation, cross-section at the crown centre line

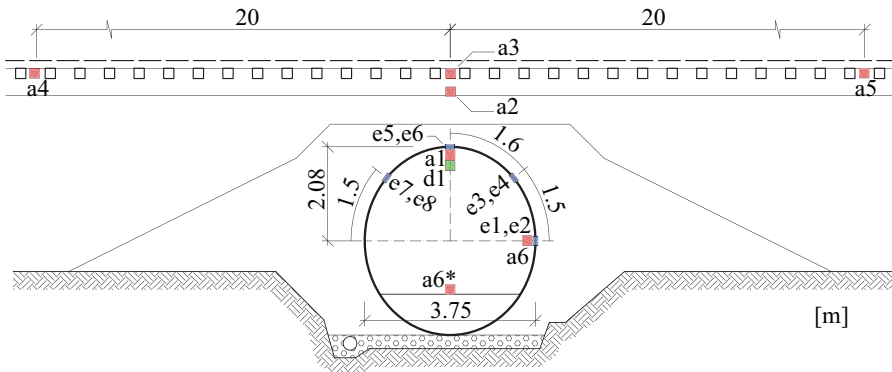


Figure 3. Instrumentation, Section A-A, track U1

After 29 train passages, accelerometer a4-a6 was moved to positions indicated with \*. Accelerometer a3-a5 was used to compare the response on the slee-

pers and the adjacent ballast, Figure 4a, while a6 was used to determine any acceleration of the soil inside the culvert, directly above gauge d1, Figure 3.

All strain gauges measured the longitudinal strain at the arch intrados, at each location both on the crest and on the valley of the corrugation, Figure 4b. This facilitates estimation of the proportion of bending moment and axial force in the arch.

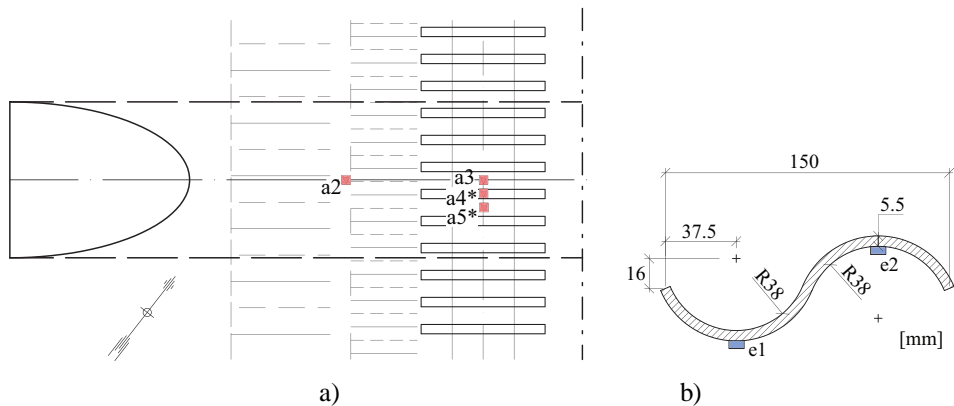


Figure 4. Instrumentation, a) plane view of track U1, b) detail of the corrugated steel sheet

A total of 52 train passages were recorded. The most common train type is the X52 commuter train, partly shown in Figure 1. It consists of only two wagons with a total length of 54 m and axle load of 185 kN/axle, the axle distances are illustrated in Figure 5. Other recorded trains were the X40 commuter train (136 m and 175 kN/axle), the X2 commuter train (140 m and 185 kN/axle) and long-distance trains consisting of Rc6 locomotives at both the front and the end with 9 or 10 intermediate wagons. In addition, one freight train consisting of an Rc4 locomotive and 17 wagons was recorded.

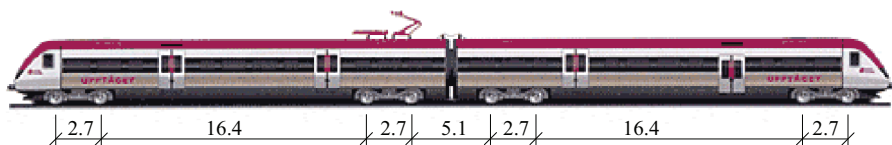


Figure 5. X52 commuter train with 2 wagons

## Vertical crown displacements

The vertical displacement of the arch crown during passage of an X52 commuter train is presented in Figure 6. Each axle is clearly visible and the response is dominated by each bogie, suggesting an influence length of slightly more than

5 m. The speed is estimated to be 175 km/h based on the time between the first and the last axle and the known axle distances of Figure 5. The maximum vertical displacement of the arch is merely 0.4 mm, decreasing to 0.2 mm at 2.1 m from the centre line (gauge d2). We assumed that both passages induced similar response of the bridge, using symmetry results in a displacement field according to Figure 7.

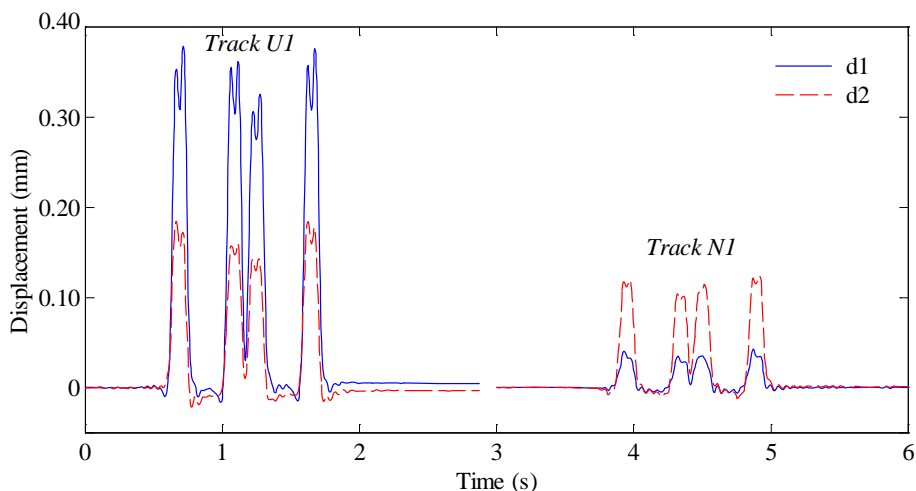


Figure 6. Vertical crown displacement, X52 commuter train passing on track U1 and later on track N1, estimated speed 175 km/h

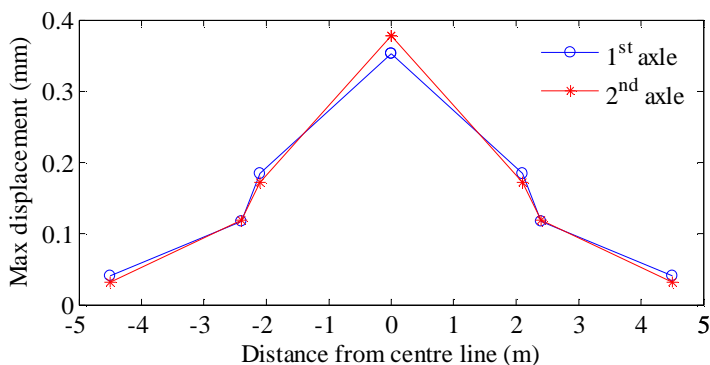


Figure 7. Max vertical crown displacement as a function of the distance from the track centre line, based on data from Figure 6.

The largest displacements for all train passages are presented in Figure 8, as relations between d1 and d2 and separated for different train types. The variation in displacement is generally low, most likely to similar loading and speed. For the long-distance commuter trains, the largest displacement is caused by Rc6

locomotives, with an axle load of 195 kN. The variation in train speed for the same type of trains is generally low, e.g.  $175 \pm 2$  km/h for the X52 train passages and  $135 \pm 5$  km/h for the commuter train passages. It is therefore difficult to draw any conclusions regarding the influence of train speed.

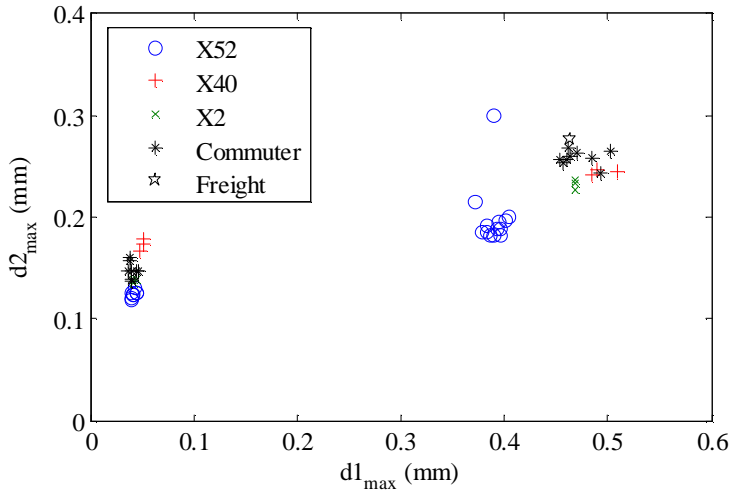


Figure 8. Max vertical crown displacement  $d1$  and  $d2$  for all train passages

## Accelerations

When analysing railway bridges for high-speed trains, a set of dynamic design criteria are to be fulfilled. The criteria mainly concern the serviceability state, e.g. accelerations levels, displacements, end-rotations etc. Most often the acceleration criterion is decisive. According to EN 1990 (CEN, 2002), it is stated that the vertical bridge deck acceleration shall be limited to  $3.5 \text{ m/s}^2$  within the frequency range no higher than 30 Hz,  $1.5f_1$  or  $f_3$ , where  $f_1$  and  $f_3$  are the first and third modes of vibration. The reason for the acceleration criteria is mainly to avoid ballast instability with resulting misalignment of the track. The design value has been investigated by (ERRI, 2000) by means of an experimental shake-down tests. They concluded that ballast instability generally occurred for a vertical deck acceleration of  $0.8g$  and the design criterion was determined simply as a safety factor 2 of that value. Most tests were performed within a 20 Hz frequency range.

Most criteria concerning dynamics of high-speed trains in EN 1990 are based on structural members acting as vibrating beams. Since the static behaviour of corrugated steel culverts highly depends on the soil-structure interaction, similar importance in dynamics is also expected. The problem at hand is likely more related to soil dynamics rather than structural dynamics. As no specific regulations are available, the initial assumption is to adopt the same criteria

as for regular structural members. In Figure 9, a comparison of the culvert crown acceleration  $a_1$  is compared to the ballast acceleration  $a_3$  between the sleepers. Within the 30 Hz band, the peak acceleration is  $0.8 \text{ m/s}^2$  in the steel and  $1.3 \text{ m/s}^2$  in the ballast. Comparing the frequency content during the forced vibration, Figure 9b, similarities are found up to 150 Hz. The decreased energy level in  $a_1$  compared to  $a_3$  beyond 150 Hz may be due to the wave speed of the backfill and the fill height at the crown. Assuming a backfill density of  $2000 \text{ kg/m}^3$  results in a p-wave speed  $c_p = 260 \text{ m/s}$ . Further assuming the wavelength to be limited by the crown fill height,  $\lambda = 1.9 \text{ m}$  and the maximum frequency  $f = 150 \text{ Hz}$  results in an estimated Young's modulus  $E_{\text{fill}} = 120 \text{ MPa}$ . This is still to be regarded as a hypothesis.

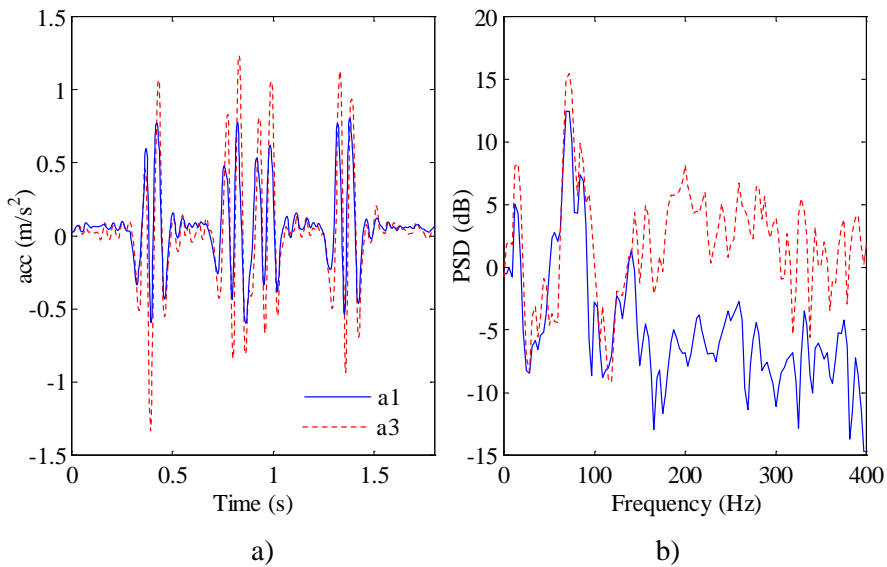


Figure 9. Vertical acceleration during passage of an X52 train, a) time history subjected to a 30 Hz low-pass filter, b) frequency content

From the shake-down tests in (ERRI, 2000) a transfer function between the deck- and ballast was described using the RMS of the measured signals. A ratio  $\text{RMS}_{\text{deck}}/\text{RMS}_{\text{ballast}} = 0.5$  was generally obtained. From the unfiltered accelerations of Figure 9a, a ratio  $\text{RMS}_{a_1}/\text{RMS}_{a_3} = 0.86/1.57 = 0.55 \text{ m/s}^2$  was obtained. In Figure 10, the peak of a running RMS with different time increments is presented. The high-frequency content is attenuated using a longer time increment and appears to stabilize for  $dt > 0.3 \text{ s}$ .

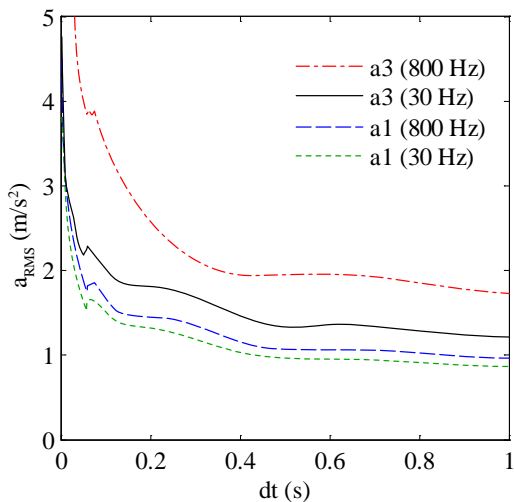


Figure 10. Time history of vertical accelerations during passage of an X52 train

The variation of peak acceleration within the 30 Hz bandwidth is presented in Figure 11 for all recorded train passages. Accelerometer a2 is mounted in the ballast at the crown, 3.0 m besides the track centre line, Figure 2. The peak acceleration scatters significantly, often showing semi-transient time histories. Accelerations less than  $0.5 \text{ m/s}^2$  are most often due to trains passing on the opposite track N1. Comparing a1 and a2, all but the commuter trains show limited scatter. Comparing a1 and a3 however, severe scatter is obtained for all passages.

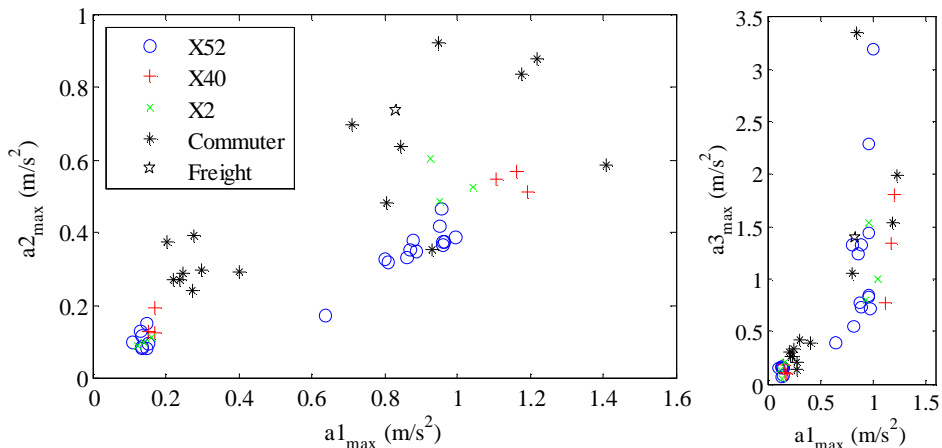


Figure 11. Max vertical acceleration during train passages, all signals have been subjected to a 30 Hz low-pass filter.



## Stresses in the corrugated steel sheet

A total of 12 strain gauges were installed at the steel sheet intrados, located in pairs in a total of six sections, Figure 2 and Figure 3. All presented stresses were obtained by assuming  $E_s = 210$  GPa. The portion of axial stress and bending stress is estimated using Equation (1). The bottom and top stresses are then again obtained using Equation (2).

$$\sigma_N = (\sigma_{\text{bot}} + \sigma_{\text{top}})/2, \quad \sigma_M = (\sigma_{\text{bot}} - \sigma_{\text{top}})/2 \quad (1)$$

$$\sigma_{\text{bot}} = \sigma_N + \sigma_M, \quad \sigma_{\text{top}} = \sigma_N - \sigma_M \quad (2)$$

For the same X52 passage as previously presented, time history of stresses in the crown is presented in Figure 12. The largest axial stress is governed by each bogie whereas the largest bending stress is governed by each axle. Hence, the peak axial stress and bending stress do not coincide.

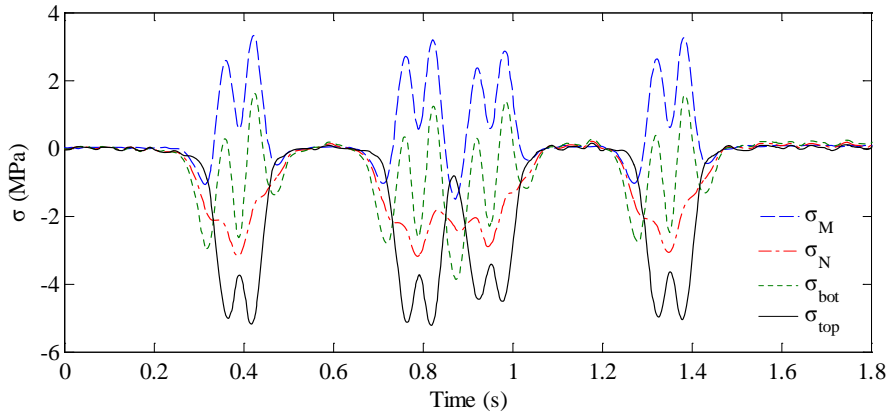


Figure 12. Time history of recorded stress at section 5-6 during passage of one X52 train

The peak stresses in all studied sections are presented in Figure 13 for the same train passage. The results show the proportions of axial- and bending stresses at the time of peak compressive (negative) and peak tensile (positive) stress. Tensile stresses are mainly present at the crown due to the bending moment. The proportions of axial stress and bending stress at the crown are presented in Figure 13b for all recorded train passages. The proportions of bending stress and axial stress scatter significantly. However, a near linear proportion is perceived for similar train types and summing up the contributions results in significantly less scatter regarding the total stress. In conclusion, all train passages caused very small stress levels.

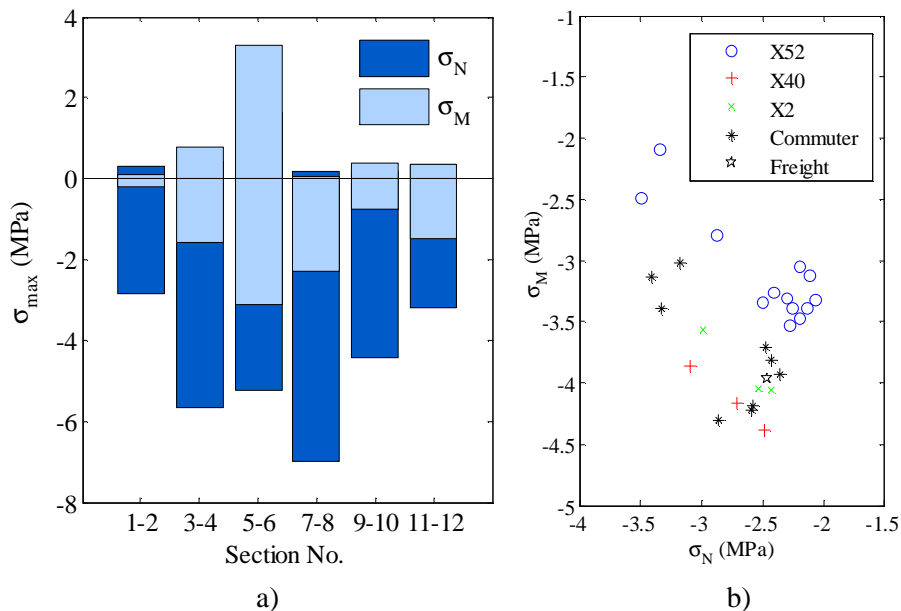


Figure 13. Measured train induced stresses, a) proportions of bending and axial stress at peak stress for one X52 passage, b) corresponding proportions at section 5-6 for all recorded train passages

### 3. CONCLUSIONS

In the present paper, the instrumentation and data analysis of a soil-steel flexible culvert for railway traffic were presented. The displacements, accelerations and longitudinal strains were measured during train passages. Most passages consisted of light-weight commuter trains running at a speed of either 175 km/h or 135 km/h. Since each train type passed the bridge at about the same speed, no conclusions could be drawn regarding the influence of the speed.

The vertical crown displacement was found to be less than 0.5 mm and with a small scatter for each train type. Using the results from two displacement gauges and results from passing trains on each track, the displacement field was estimated and found to be rather wide.

The acceleration was measured both on the steel sheet and on the overlying ballast. A ratio of the deck/ballast acceleration was found in the range of 0.5 based on the RMS of the train passage. The acceleration generally showed a great scatter, even for similar train passages. The peak deck acceleration varied between 0.8 – 1.4  $m/s^2$ , the corresponding acceleration of the ballast varied between 0.5 – 3.5  $m/s^2$ .

Longitudinal strains were measured at six different positions at the arch intrados. At each section, the top- and bottom of the corrugation were measured, facilitating the estimation of axial- and bending stresses. The peak stress levels

were less than 10 MPa, due to the short span culvert with rather high crown fill height. The time of peak axial stress was found to be governed by each bogie load whereas the peak bending stress was governed by each axle load. Due to this, the time of peak axial stress did not coincide with the peak bending stress. The proportion of axial- and bending stress scattered significantly even for similar train passages. The scatter was, however, much smaller when comparing the total stress.

Additional knowledge can be obtained by using the measured data for model calibration, e.g. using the finite element method.

#### 4. ACKNOWLEDGEMENTS

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#### REFERENCES

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#### Streszczenie

Niniejsza praca przedstawia rezultaty testów naturalnej wielkości podatnej konstrukcji gruntowo-stalowej służącej jako wiadukt kolejowy. W badanym obiekcie zainstalowano tensometry, przyspieszeniomierze i mierniki przemieszczeń, mierzące reakcję konstrukcji na przejazdy pociągów. Celem serii pomiarów było uzyskanie wiedzy na temat dynamicznego zachowania konstrukcji wywołanego drganiami konstrukcji wiaduktu oraz sąsiadującego nasypu kolejowego, wzbudzonymi przez pociągi. Dane pomiarowe pozwalają na oszacowanie rozkładu obciążeń oraz sztywności gruntu. Rezultaty służą również jako podstawa do kalibracji modeli numerycznych stosowanych do przewidywania zachowania pociągów wysokiej szybkości.

Słowa kluczowe: podatny przepust gruntowo-stalowy, szybkie koleje, pomiary terenowe, rozkład obciążeń, przyspieszenie balastu

