# Experimental Testing of a Railway Bridge with Near Viscous Dampers

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**Abstract.** This paper present experimental testing of an in-service railway bridge where viscous dampers were installed near the roller support, with the objective to mitigate vibrations from passing trains. Forced vibration tests were performed before and after installing the dampers, which showed a significant increase in resulting modal damping. A long-term monitoring system was installed and more than 4000 train passages has been recorded to date. The system records both the response of the bridge and the resulting force in the dampers. Based on a Bridge Weigh-in-Motion system (BWIM), the train speed and train types are identified. The estimated train speed varies from 40 to 160 km/h for passenger trains, which enables to study the dynamic performance of the bridge under different loading frequencies. The variation in modal properties, e.g. natural frequency and damping is studied as function of the vibration amplitude. The bridge is located in the North of Sweden where the temperature is expected to range from +30°C in the summer to -30°C in the winter. Experimental data will be collected to further study any seasonal changes on the dynamic properties of the bridge, e.g. due to longitudinal stiffness of the continuous track system.

**Keywords:** Railway bridge; experimental testing; damping; viscous damper; seasonal effects.

# 1 Introduction

Railway bridges are sometimes susceptible to dynamic excitation from passing trains, especially if the load frequency coincides with any of the natural frequencies of the bridge. For long and slender bridges on high-speed lines, this may occur within the range of the operational speed. In combination with low inherent damping of the bridge, excessive vibrations during train passage may pose both safety risks and riding comfort issues. One approach to mitigate the response of railway bridges and to ensure resilience is to install external damping devices. This paper presents a case study of an existing railway bridge in Sweden, where experimental testing has been performed before and after installing external viscous dampers. Theoretical models of the bridge has been developed by [1] and [2], design of the dampers and the first part of the experimental testing is reported in [4] and part of the long-term monitoring is reported in [5].

# 2 The case study bridge and the dampers

The case-study bridge is located near Örnsköldsvik, about 540 km North of Stockholm, Sweden. The bridge is a simply supported, single track, steel-concrete composite bridge with a span of 48 m. The elevation of the bridge is shown in Fig. 1. The left support has roller bearings that allows longitudinal movement of the bridge and the right support has fixed bearing. A cross-section of the bridge is shown in Fig. 2 along with a detail of the roller bearing. The main load carrying elements are the two steel I-girders, with a height of 2.5 m and flanges of 1380×50 mm. The flexural rigidity of the composite section is estimated to 240 GNm$^2$ and the mass to 18000 kg/m. This results in a first natural frequency of about 2.5 Hz.

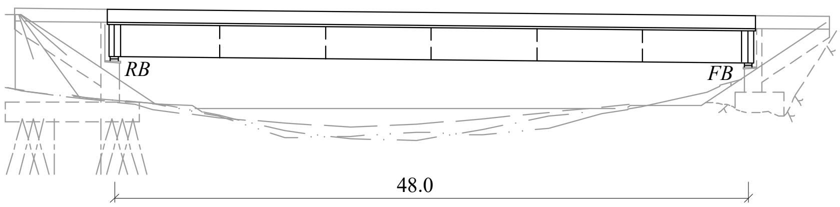


**Fig. 1.** Elevation of the bridge, roller bearings (*RB*) and fixed bearings (*FB*).

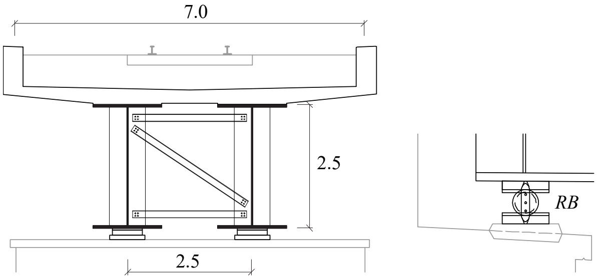


**Fig. 2.** Left: cross-section of the bridge, steel girders and concrete deck, right: detail of the roller bearing.

The bridge is located on a railway line with an allowable speed of 250 km/h, although current trains do not run faster than 180 km/h. The compatibility for future high-speed trains has been assessed using Eurocode EN 1991-2, assuming a lower bound damping of 0.5% and train load models denoted HSLM-A. This, in combination with the relatively low mass and low natural frequency makes the bridge susceptible to resonant loading and a theoretical exceedance of the vertical deck acceleration by more than a factor 2.

This has led to the motivation of installing external viscous dampers with the objective to increase the total damping of the bridge, especially for the first mode of vibration since that will be governing the resonant loading. Viscous dampers rely on a relative motion along the direction of the damper and it is usually not viable to install any obstructing device directly under the bridge. One option can be to connect the damper to an auxiliary beam as in [7-8], but that would likely result in a rather large extra structure due to the long span length. Another option is instead to use the eccentricity between the neutral axis of the bridge cross-section and the roller bearing. For this bridge the eccentricity is 2.3 m, which results in a noticeable longitudinal displacement at the roller bearing. The optimal damper location near the support was studied in [3] but it was found that the most reliable and robust solution would be to install the damper horizontally as close to the roller bearing as possible. This solution was used and the installed dampers is shown in Fig. 3. The dampers were manufactured by Dellner Dampers AB.

**Fig. 3.** View of one of the dampers when mounted at the roller support.

**Fig. 4.** Details of the damper, left: fastening, right: damper with accumulator tank.

The expected displacement at the damper location is only about 2 mm, which puts extra requirements on the damper performance at low amplitudes. The performance at low amplitudes was improved by separate accumulator tanks that creates an over-pressure in the damper fluid, see Fig. 4. In total four dampers were installed, following the plan in Fig. 5. Each damper has a theoretical damping coefficient $c_d$ = 1000 kNs/m and a load capacity of 100 kN. The performance of the dampers were verified by separate laboratory testing, reported in [4].

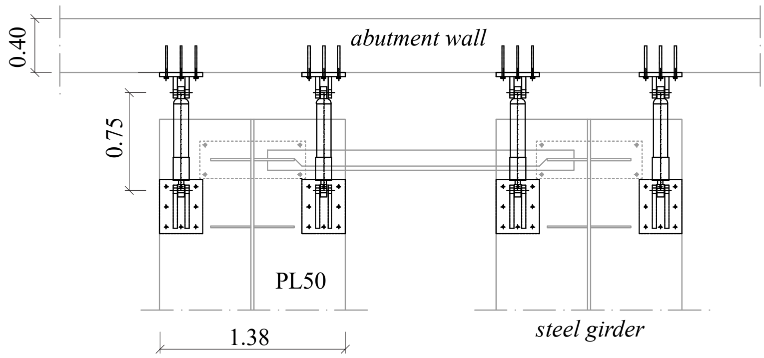


**Fig. 5.** Plan view of the bridge-damper installation.

## 3 Forced vibration tests

Experimental testing was performed in June 2021, using a forced vibration actuator system illustrated in Fig. 6. The actuator system has a load capacity of 20 kN and was placed under the bridge at 3.7 m from the roller bearing. Further details of the actuator system is reported in [6].

**Fig. 6.** Left: view of the bridge during train passage, right: the actuator during forced vibration testing.

During the forced vibration testing, the instrumentation according to Fig. 7 was used. Ten uniaxial accelerometers A1-A10 (PCB 393A03) was installed vertically on the lower flange of the steel girders. Four LVDTs D1-D4 was installed longitudinally between the abutment wall and the steel girders at the roller bearing, D1 and D3 at the upper flange, D2 and D4 the lower flange. Each damper was installed with strain gauges in a full-bridge configuration, measuring the damper force F1-F4.

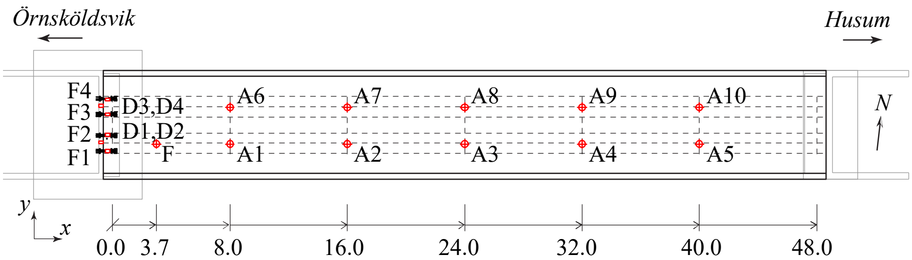


**Fig. 7.** Instrumentation during the forced vibration testing.

Forced vibration tests were performed for the case of no dampers, two dampers and four dampers. For each case, a load amplitude of 1, 5, 10 and 20 kN was used. The bridge has two closely spaced modes corresponding to the first vertical bending mode at about 2.4 – 2.6 Hz and the first torsional mode at about 2.7 – 2.8 Hz. This is illustrated by the frequency response function (FRF) in Fig. 8 for the case of 20 kN load amplitude. The modes are separated by subtracting the response from each side of the bridge, in this case A3-A8. The FRF is then fitted to a 2DOF-system using the function *modalfit* in MATLAB. This is done for all combinations of load amplitudes and number of dampers and the estimated modal properties are presented in Table 1.

For the first bending mode, the natural frequency does not change due to the number of dampers but changes slightly due to the load amplitude. The corresponding damping does increase with the number of dampers, at most from 1.0% to 1.6% for the 20 kN load amplitude. This load corresponds to a peak acceleration of about 0.5 m/s$^2$ and a support displacement of about 0.5 mm for the case without dampers and 0.4 m/s$^2$ and 0.35 mm displacement for the case of four dampers. Similar trends in damping ratios are found for the cases of 10 kN and 5 kN load amplitude, but the damping ratio is lower for the 1 kN load amplitude, ranging from about 0.6% to 1.0% as the number of dampers increases.

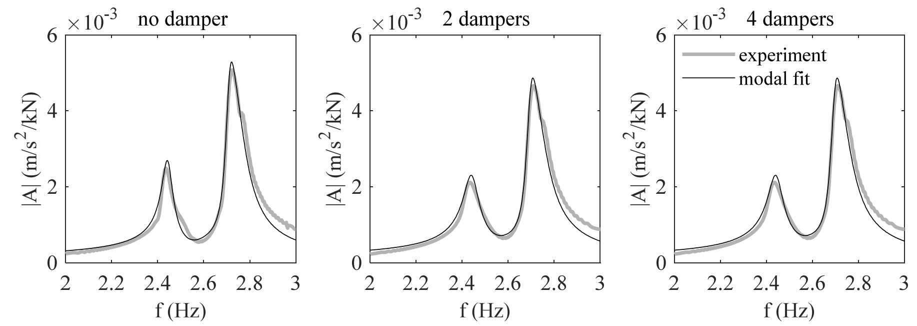


**Fig. 8.** Frequency response function at mid-span (A3-A8) for the first bending and first torsion mode, 20 kN load amplitude.

**Table 1.** Estimated natural frequency and damping, first bending and first torsional mode.

| $F_{amp}$ (kN) | *dampers* | $f_1$ (Hz) | $\zeta_1$ (%) | $f_{1,t}$ (Hz) | $\zeta_{1,t}$ (%) |
|---|---|---|---|---|---|
| 20 | 0 | 2.44 | 0.99 | 2.71 | 0.83 |
| | 2 | 2.44 | 1.43 | 2.69 | 0.92 |
| | 4 | 2.46 | 1.59 | 2.71 | 0.84 |
| 10 | 0 | 2.49 | 1.04 | 2.76 | 0.72 |
| | 2 | 2.48 | 1.35 | 2.74 | 1.21 |
| | 4 | 2.49 | 1.53 | 2.76 | 1.50 |
| 5 | 0 | 2.52 | 0.92 | 2.78 | 0.53 |
| | 2 | 2.51 | 1.22 | 2.76 | 0.66 |
| | 4 | 2.53 | 1.46 | 2.79 | 1.17 |
| 1 | 0 | 2.59 | 0.63 | 2.79 | 0.33 |
| | 2 | 2.58 | 0.80 | 2.78 | 0.43 |
| | 4 | 2.60 | 1.02 | 2.82 | 0.47 |

## 4 Long-term monitoring system

After completing the forced vibration testing, a reduced instrumentation according to Fig. 9 was performed and a system for long-term monitoring was installed. D1 measures the displacement at the upper flange and D2 at the lower flange. BW1 and BW2 are Bridge-Weigh-In-Motion (BWIM) sensors that measure the transverse bending at the soffit of the concrete deck. BW1 and BW2 are spaced 8.00 m apart and are primarily used to determine the type, speed and direction of passing trains. Sensors s1 and s2 are uniaxial strain gauges installed in the longitudinal direction at the lower and upper flange of the steel beam. The temperature at the bridge is recorded by sensor T under the bridge. The long-term monitoring system has a trigger that only saves data for passing trains. Each file is sampled with 150 Hz and contains 50 seconds of recorded data, including 10 seconds before the train enters the bridge.

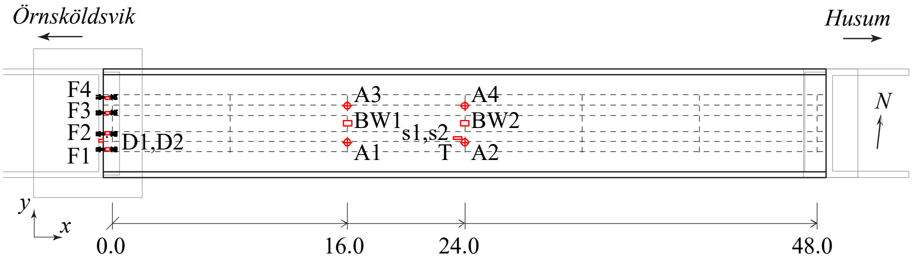


**Fig. 9.** Instrumentation during the long-term monitoring programme.

## 4.1 Peak response from passing trains

The bridge is traversed by mixed train traffic, but one of the most common train types is the X62 passenger train, with geometry according to Fig. 11 and an axle load of about 170 kN. During the period from June 18 to October 26 a total of 3741 trains were recorded, of which 2008 trains were detected as type X62.

The peak response in terms of vertical deck acceleration and total damper force is shown in Fig. 10. The speed is estimated based on the BWIM-sensors. Given a natural frequency of 2.6 Hz and a centre bogie spacing of 16.4 m results in an estimated critical speed of 152 km/h. This is close to the experimental results for trains bound to Övik. For the X62 trains in the opposite direction however, the critical speed is about 135 km/h. The reason for this change is not readily known, but would for the same load frequency correspond to a natural frequency of 2.3 Hz. The total damping force capacity for all four dampers is 400 kN, which is significantly higher than the peak force of merely 25 kN for the X62 train. It should be noted however that the X62 train is merely 66 m long, compared to the design load models that are about 400 m long.

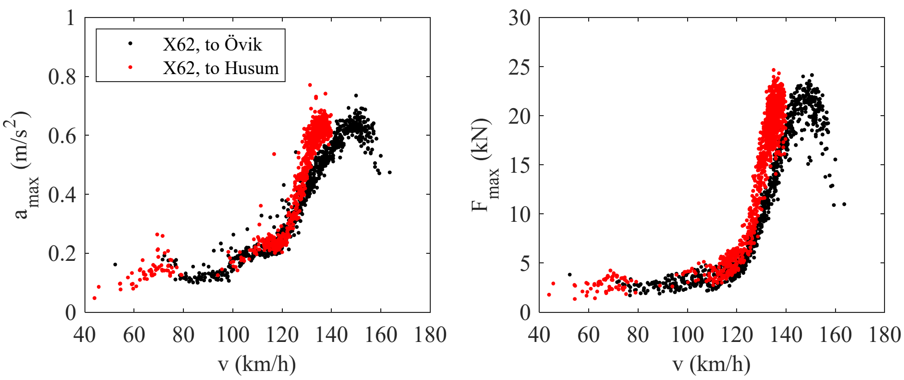


**Fig. 10.** Peak acceleration and peak total damper force during passage of the X62 trains.

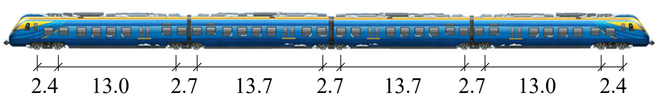


**Fig. 11.** Geometry for the X62 Cordia Nordic manufactured by Ahlstrom, operated by Norrtåg.

### 4.2 Free vibration response

Based on recorded train passages, the free vibration response is used for estimating the natural frequency and damping for the first bending mode. As seen from the forced vibration testing, the damping seems to be amplitude dependent. It is therefore expected that the estimated damping from free vibrations depends on the duration of the signal to be included in the analysis. The start of the free vibration is counted from when the whole train has left the bridge, estimated from the BWIM-sensors. The signal is band-pass-filtered between 2-3 Hz and the analysed signal is the average response from accelerometers A2 and A4 in Fig. 9, to attenuate the influence from the torsional mode. The natural frequency is estimated from the peaks of the free decay and the damping as a regression of the logarithmic decrement. Data with a coefficient of determination $R^2 < 0.9$ is discarded.

The procedure is performed for a duration of 1-6 seconds of the free vibration response and the results are presented in Table 2. The natural frequency is near constant at 2.6 Hz, similar to the results obtained from the forced vibration testing with low load amplitude. The damping does however vary from about 1.5% to 2%, the higher value corresponding to a shorter duration of the signal. It should also be noted that the standard deviation is rather high, indicating a significant scatter in the results, especially for shorter free vibration times. Fig. 12 shows a histogram of the results for a duration of 2 seconds, confirming the large scatter in the estimated damping.

**Table 2.** Estimated values of frequency and damping for the first bending mode, based on free vibrations after train passage.

| $t_{free}$ (s) | $f_1$ (Hz) | | $\zeta_1$ (%) | |
|---|---|---|---|---|
| | mean | std | mean | std |
| 1 | 2.58 | 0.06 | 2.09 | 0.93 |
| 2 | 2.58 | 0.05 | 2.09 | 1.00 |
| 3 | 2.58 | 0.05 | 1.90 | 0.54 |
| 4 | 2.58 | 0.04 | 1.75 | 0.44 |
| 5 | 2.58 | 0.03 | 1.59 | 0.32 |
| 6 | 2.58 | 0.03 | 1.49 | 0.23 |

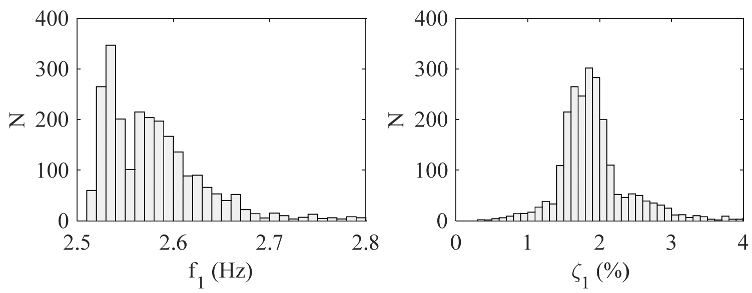


**Fig. 12.** Histogram of the first natural frequency and damping, evaluated from 2 seconds of free vibration after train passage.

## 5 Conclusions

This paper presented experimental testing on a railway bridge retrofitted with external viscous dampers. The objective was to increase the modal damping for the first bending mode during train passages.

The dampers were installed longitudinally at the roller support, relying on the displacement caused by the eccentricity between the neutral axis of the bridge and the roller bearing. To assure good damping performance at low amplitudes, separate accumulator tanks was installed on each damper to provide an over-pressure in the damper fluid. In addition, extra attention was needed to avoid gaps at the fastening points.

From the forced vibration tests, the first natural frequency showed small variation within 2.4 – 2.6 Hz but the corresponding damping ratio increased from 1.0 – 1.6% when installing the dampers.

The peak response from passing trains showed that the resonance speed was about 152 km/h, well in agreement with the expected results. The peak acceleration was 0.8 m/s$^2$ and the peak damper force 25 kN.

Based on the free vibration after train passages, the natural frequency was about 2.6 Hz and the damping 2%. The scatter in damping was however significant.

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