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Development and application of tuned mass dampers to a historic pedestrian bridge in Switzerland

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ABSTRACT: The more than seventy years old pedestrian bridge *Haggenbrücke* situated close to *St. Gallen*, *Switzerland* is one of the highest footbridges in Europe. The maximum column height is about 90 meters. Although designed for lightweight road traffic, the bridge was only approved for pedestrian traffic due to its vibrational sensitivity recognized during the bridge opening in 1937. Besides the vertical vibrations of the filigree steel truss, pronounced horizontal vibrations can be observed under high pedestrian volume. During a thorough restoration of the bridge in 2009 / 2010, tuned mass dampers were built-in beneath the deck plate. The present paper reproduces the design, the manufacturing, installation and the performed test of the vertical TMDs.

1 INTRODUCTION

The pedestrian bridge *Haggenbrücke* is situated close to *St. Gallen, Switzerland*. It crosses the valley of the river *Sitter* with an overall length of approximately 352 m. The bridge has seven spans with a maximum length of 69 m. The maximum column height is about 90 m. The superstructures as well as the columns are very filigree steel trusses.

1.1 Restoration project

After more than seventy years the bridge showed substantial damages concerning the corrosion protection of the steel structure, the RC deck slab and the foundations. Comprehensive finite element calculation (Fig. 1) revealed capacity overloads of several truss elements.

Beside the renewal of the corrosion protection and the retrofitting of the foundations the concrete deck was replaced by an orthotropic steel plate within the restoration project. The new orthotropic steel plate is lighter than the former RC deck. This led to a decrease of the dead loads and due to the connection with the truss girder to an increase of the superstructure stiffness.

To reduce the vibration sensitivity of the bridge four vertical and two horizontal tuned mass dampers (TMD’s) were provided.

The bridge is a hot spot of suicides. Therefore, a further measurement of the restoration project was therefore the installation of nets beneath the deck plate to prevent future suicide attempts.

1.2 Dynamic behavior of the historic bridge

During the opening in 1937 (Fig. 2) the bridge surprisingly showed very strong horizontal vibrations under the excitations of the crowd. Therefore, car traffic was restricted and comprehensive static verifications were carried out. Additional retrofitting and strengthening of the truss girder could not eliminate the vibration sensitivity.

To identify the vibration behavior of the bridge two different kinds of excitations were investigated: ambient noise and more or less impulsive excitation due to jumping and running on the pavement.

The measured natural frequencies in transverse horizontal and vertical direction are given in Tab. 1. They are compared with the calculated values. Fig. 3 and 4 show the amplitude functions of the excitations due to ambient vibration. The measured frequencies lie in the critical range defined by Swiss Code SIA 260:2003 of 1.6-4.5 Hz for vertical and <1.3 Hz for horizontal vibrations.

The measured maximal amplitudes of the velocities in vertical direction were 17.1 mm/s. They are larger than the limit of 5 mm/s defined by the German code DIN 4150, Part 2. Due to the large mass to be excited, it was difficult for one person to build up the horizontal vibrations. In comparison to the vertical vibrations, the horizontal ones are only important in the rare event of many people on the bridge (e.g. Fig. 2).

The damping of the structure could be measured by the decay of the vibration due to impulsive excitation. With a short-time Fourier transform (Fig. 5) the damping ratios of the first mode for vertical and horizontal vibrations could be identified with 0.3% for 1st horizontal and 0.9% for the 1st vertical mode.

The results of the measurements show the vibrational sensitivity and the typical low damping of the steel truss structure.

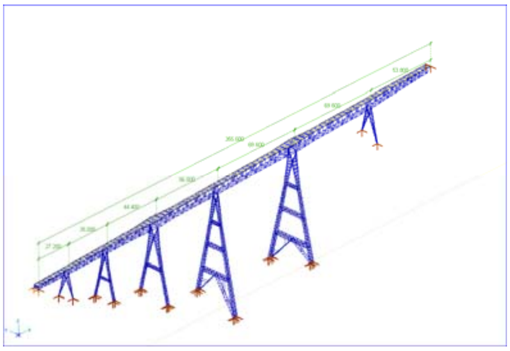

Figure 1, FE-model of the complete bridge

Figure 2, Bridge opening in 1937

Table 1. Comparison of measured and calculated frequencies

	horizontal transverse direction		vertical direction	
Mode	Measurement	Calculation	Measurement	Calculation
1st	0.66 Hz	0.68 Hz	1.8-2.05 Hz	1.74 Hz
2nd	0.88 Hz	0.87 Hz	2.35 Hz	2.11 Hz
3rd	1.10 Hz	1.09 Hz	2.75 Hz	2.54 Hz

1.3 Dynamic behavior of the bridge with new orthotropic steel deck plate

A small increase of the natural horizontal frequencies from 0.66 to 0.7 Hz could be observed after the retrofitting of the new steel deck plate. Furthermore the lower vertical frequencies could no longer be measured. The 1st and 2nd natural vertical frequencies lie now at 2.45 and 2.7 Hz. There is almost no change of the damping of the structure.

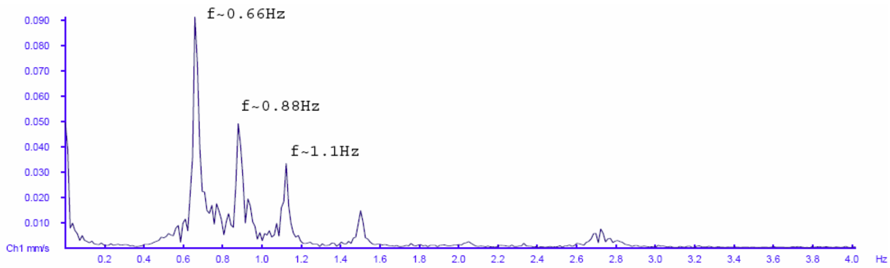

Figure 3, Measured amplitude function, horizontal transverse direction

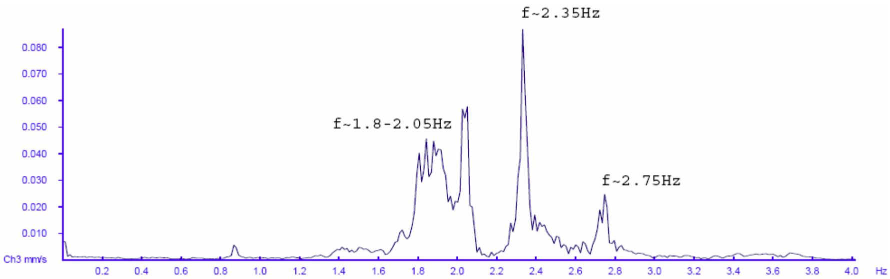

Figure 4, Measured amplitude function, vertical direction

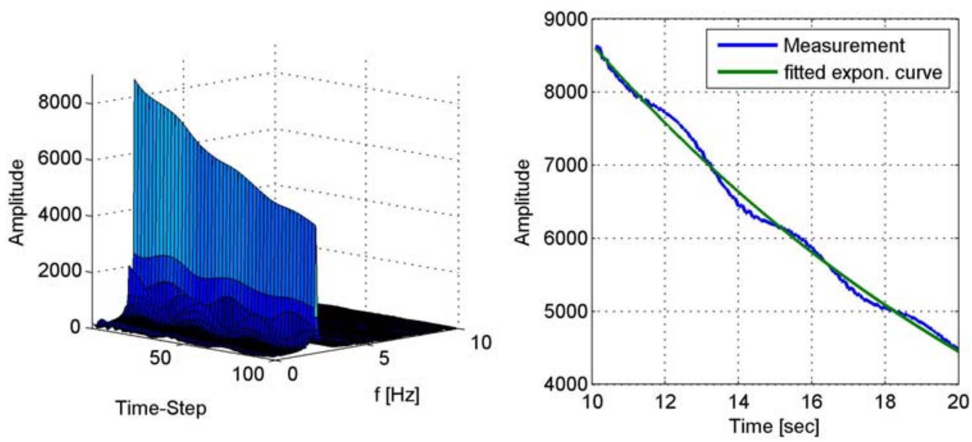

Figure 5, Short time Fourier transform of the velocity (left) and exponential decay (damping 0.9%, right).

2 DESIGN OF THE VERTICAL DAMPERS

2.1 Predesign of the vertical dampers

The installations of four vertical TMDs in the middle of the largest spans (span 4, 5, 6 and 7) were planned to reduce the vibration amplitudes. Two TMDs should damp the lower and the higher resonance frequency. The dampers were predesigned according to the criterias of *Den Hartog*, see Butz et al. (2008). The mass of the new superstructure with orthogonal steel plate and mastic asphalt is 142’000 kg for the largest span. With a mass ratio damper to bridge of 3% and a mode factor of 0.5 for each span the damper masses are 2’200 kg. The optimal frequency and damping of the TMDs are 2.62 Hz and 10% for the higher bridge natural mode of 2.7 Hz. For the lower natural mode of 2.42 Hz the mass was increased to 2’700 kg in order to use the same spring elements.

2.2 Design of the spring elements

Each vertical TMD contains four compression springs beneath the mass block. From the damper mass of 2'200 kg and the damper natural frequency of 2.62 Hz a spring rate of 145 N/mm results. The springs are designed against buckling under static as well as dynamic loads and with endurance strength. The static deflection is found to be 40 mm and smaller than the predicted dynamic deformation of about 20 mm.

The dynamic deformation is calculated by using the Fourier series load model given in Butz et al. (2008) for runners with the Fourier coefficients of Bachmann et al. (1995). A group of ten runners is assumed with an average weight of 75 kg and a contact time of 0.2 s. The frequency of the runner's steps is assumed to be the same as the natural frequency of the bridge deck.

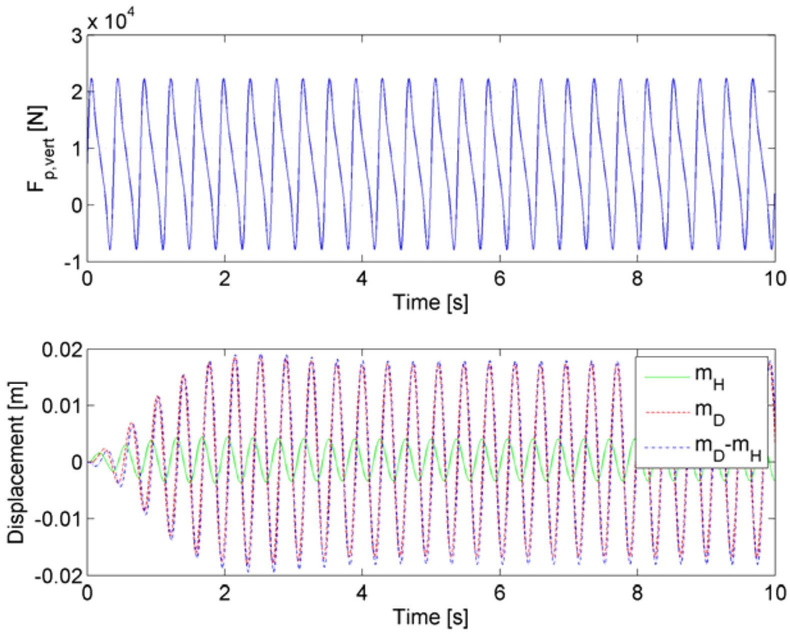

Figure 6, Runner's excitation for 10 second (top) and response of the TMD and the bridge deck (bottom), m_H: mass of bridge, m_D: damper mass

2.3 Design of the damper elements

Ready-to-install dampers were not available for this project and it was necessary to develop an own design. We chose a viscous fluid damper with vertical moving steel tubes in silicon oil (Fig. 7). The steel tubes generate the surface for the friction of the viscous liquid. It is possible to remove or add tubes in order to adjust the damping ratio beside the change of fluid viscosity. The damping properties were estimated assuming the flow of the viscous fluid through the annular gaps of the tubes under pressure. A *Newtonian fluid* was assumed for the viscosity of the oil.

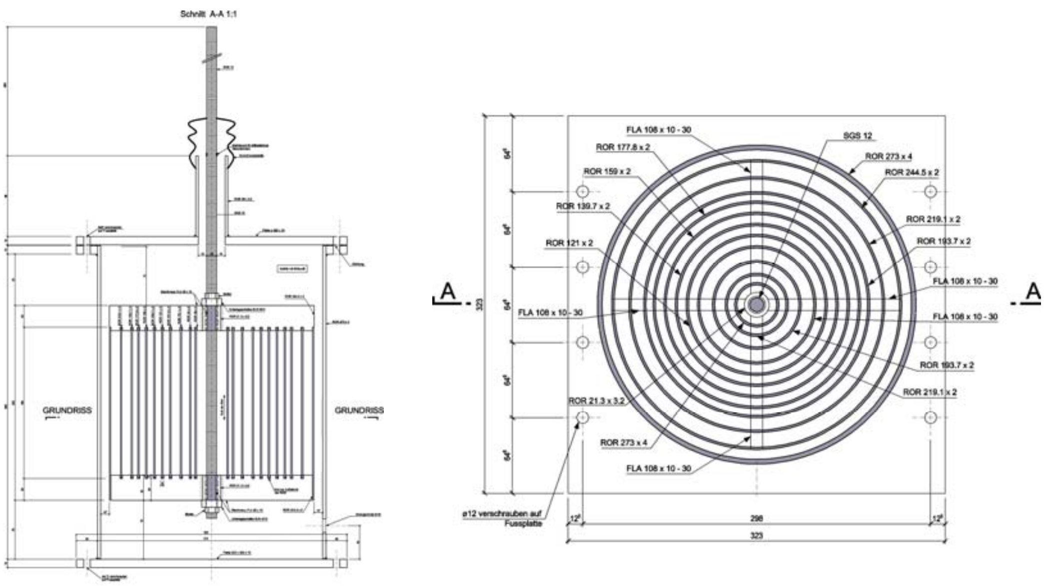

Figure 7, Viscous damper element (detail of the shop drawing).

2.4 Construction of the overall system

The complete damper system is shown in Fig. 8. The TMDs were assembled at the building site and installed in steel cages immediately beneath the steel deck plate. Openings in the deck plate permit the installation and maintenance of the TMDs.

3 PERFORMED TESTS FOR THE VERTICAL DAMPERS

3.1 Workshop tests

3.1.1 Tests of the damper element

Experimental tests were performed for the adjustment of the damper elements with respect to the oil viscosity and number of steel tubes. The damping behaviour was analysed by measuring the downward movement of the inner steel tubes trough the viscous liquid under their own weight. The ratio of weight to velocity determined the damping of the element. Linear extrapolation gives the correct liquid viscosity of the damper.

Two kinds of measurements were performed: (1) velocity sensor at the tubes and (2) displacement measurement. Both measurements yielded to similar results. The test results yielded to a fluid viscosity of 1250 cSt or a mixture of silicon oil of 1000 and 5000 cSt of 74% and 26%, all steel tubes have to be used.

3.1.2 Test of the complete TMD

Two tests were performed prior to the installation of the complete TMDs in the bridge: (1) measurement of the natural frequency of the damper without damping of the silicon oil and (2) measurement of the damping of the TMD itself.

The workshop tests result in a natural frequency of 2.59 Hz and a damping ratio of 9-12% for the complete TMD. The natural frequency of the TMD is very close to the optimal frequency according to the *Den Hartog* criteria. The deviations of the damping ratio from the optimal values are tolerable.

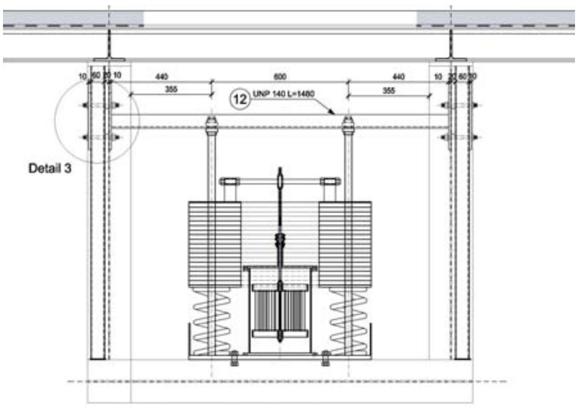

Figure 8, Complete TMD in steel cage beneath the deck plate.

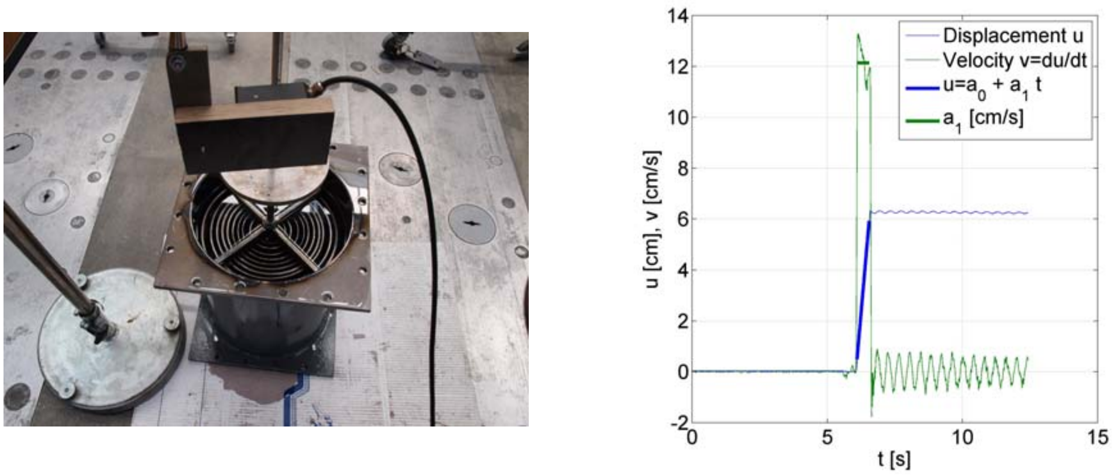

Figure 9, Damper element, test set-up, measured displacement and velocity (right)

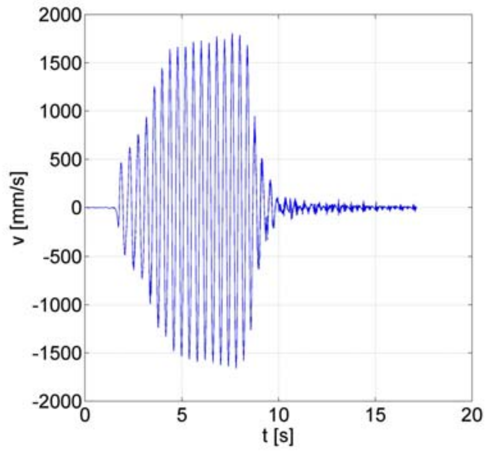

Figure 10, Workshop test, left: complete TMD, right: velocity decay of the damper mass

3.2 Bridge tests after TMD installation

The vertical TMD's were installed in the middle of four spans (see Fig. 11). The efficiency of the vertical TMDs were tested using a harmonic shaker with and without dampers. The frequency of the shaker mass of 31 kg passes through a range of 1.0 to 10 Hz. In Fig. 12 the resulting amplitude curves are compared. The TMD reduces the maximum amplitude by a factor of about 3.

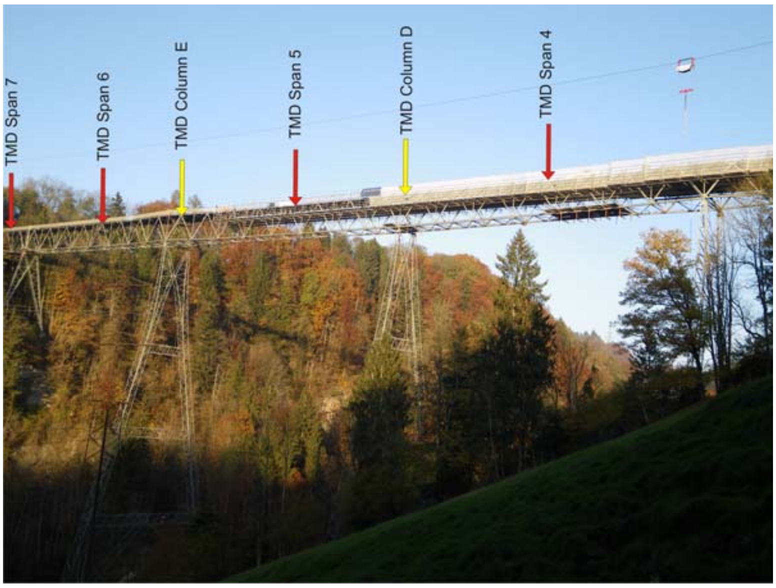

Figure 11, Positions of vertical (red) and horizontal (yellow) TMD's.

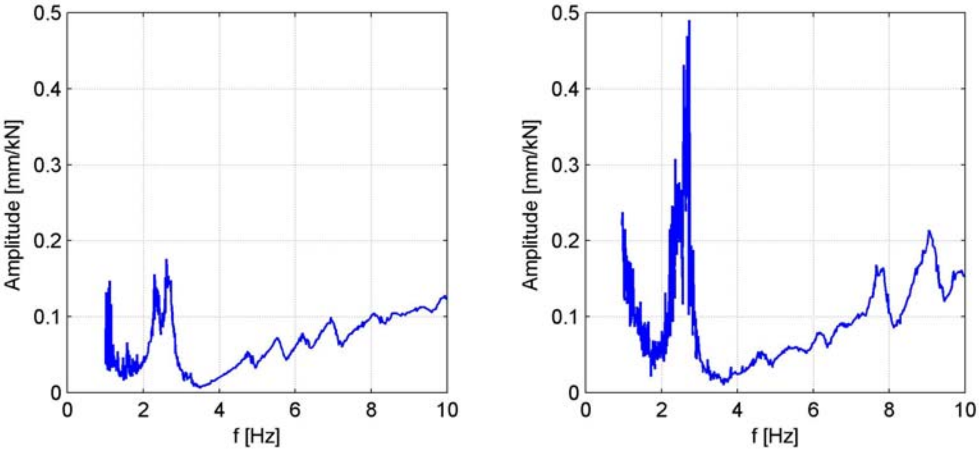

Figure 12, Frequency response of the bridge deck with (left) and without (right) TMD's.

4 CONCLUSIONS

The presented development of tuned mass dampers followed a stepwise procedure of designing, testing and adaptation. This way, the aim of reducing the vertical vibration sensitivity of the bridge *Haggenbrücke* could be achieved through a "normal" engineering design and construction process. The overall cost was smaller than for ready-to-install dampers and an individual and subsequent adaptation is possible. A bottleneck in the production of the dampers is the long delivery period for third party products e. g. the used spring elements. The frictionless free movement of the damper mass is a further important point of the damper functionality. In addition to the design a manufacturing, it should be aimed for an exact manufacturing with close tolerances.

5 REFERENCES

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