

# The Evaluation of the Dynamical Soil-Bridge Interaction

Andrej Strukelj, Matjaz Skrinar  
Faculty of Civil Engineering, University of Maribor  
2000 Maribor, Slovenia

## 1. Introduction

Although the earthquake engineering can be principally considered to be a semiempirical science it is very important to predict the behaviour of the structures within engineering accuracy. As earthquakes cannot be predicted and even much less prevented, we must learn how to live with them. It is an engineering task to design the buildings in the way to prevent the disaster and casualties. For a better understanding of the structure behaviour undergoing various loads, a suitably chosen model, that is simple enough and yet accurate, plays an extremely important role in engineering analysis. As the bridges are earthquake very sensitive structures, many engineering analyses are devoted not only to designing of the bridges, but also to the monitoring of the bridges conditions during their implementation. A wide list of examples is reviewed in paper by Salawu and Williams, 1995.

Besides numerical simulations with more or less complex computational models of the structure, tests on models or even on real structures are performed. Fanous et al. 1996 have fabricated a 1:3 shell-bridge model and tested it with service-load to determine its behaviour when subjected to various load patterns. Similar tests on a laboratory model were performed also by Bensalem et al. 1995 and Bensalem et al. 1995. Kunnath and Sashi 1995 have calibrated analytical model using static lateral load tests, ambient and forced vibrations on a model to simulate the response of the Cypress Viaduct. For the inelastic damage analysis a computer program has been implemented. A numerical study of the effects of soil-structure interaction on seismic response of PC cable-stayed bridges using 2-D FEM model is given by Zheng and Takeda 1995. A comparison of results by 2-D FEM model and mass-spring model is also given. A comparison between the behaviour of a computational model in which the structure is idealised without the foundation system and a model in which both the structure and the supporting foundation are idealised is given by Elassaly et al. 1995.

Determination of equivalent soil spring constants for a simple model with lumped masses, springs and dashpots, and the proposed formulas for calculating such constants for both spread (shallow) footing and pile-supported (deep) foundations is given by Chen 1996.

## 2. The Computational Model of the Bridge

In the case considered the concrete upper part was modelled very precisely using an explicit finite element model. The computational model of the upper construction, like actual construction itself, consisted of the pier, the base of the cantilever and the symmetrically placed segments of the cantilever with the travelling wagon at the end of each side for the in-place concreting. The construction had two planes of symmetry until finishing the twelfth segment. 3-D models of each construction phase followed all of its characteristic geometric properties as much as possible. The travelling wagons on both edges of construction were also modelled since their influence on the dynamic characteristics of the system due to their mass has proved itself to be very high. The pier and the base were divided into 5716, each of the segments into 850 to 1100, and the model of the erection gantry depending on its position into 1320 to 1400 tetraedric finite elements. For the analysis of the

construction with 24 symmetrically placed segments (12 from each side) 31226 elements were used. As the bridge construction was founded by piles through the layer of gravel directly into the underlying rock the pier was in the first approximation calculated as fixed on the rigid half-space.

### 3. The Bridge Data

The project features twin parallel prestressed concrete structures 237 m long with 3 spans. The river crossing is a three-span continuous structure composed of a 71 m back span, a 110 m main span and a 56 m back span. The geometry of the structure includes a vertical curve with a radius of 1000 m and cross slopes of 2 % and 2.5 %, the longitudinal slope being 1.5 %. The horizontal distance between the structures is constant (0.6 m) throughout the entire length. On the left longer city embankment, a large, impressive pier supports the massive part of the structure. The structure continues towards the right slender pier standing in the river. The structures were erected by a double (balanced) cantilever segmental construction using travelling wagons for in-place concreting. An additional stability system was required to support the out-of-balance moment at the slender right pier only. This system consisted of a temporary concrete frame that have been removed after the construction was finished. The superstructure is made of concreted segments with a typical box section 11.70 m wide at the deck. The box girder out-to-out width of 6 m consists of one cell and two web walls. The depth of the boxes varies from 7.50 m at the main pier table at the left embankment to 2.80 m from the mid-span to right pier.

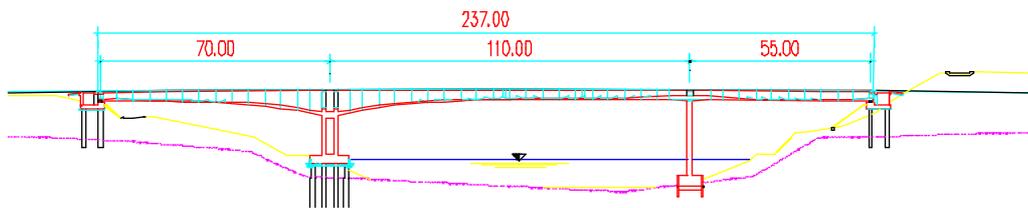


Figure 1. Elevation of the bridge

### 4. Dynamic Testing on the Bridge Structure

Due to the significance of the structure, the design engineer has requested additional controls despite the regular supervising activities and the eigenfrequency behaviour of the structure was chosen as a control parameter. The measurements of the dynamic characteristics have been performed throughout the erection of the down-water left embankment structure. In the case under consideration the ambient vibrations are measured and the primary interest is focused on the eigenfrequency changes caused by the erection of each segment. Two piezoelectric accelerometers were used to measure the bridge response to the ambient excitation in vertical and transverse directions. The accelerometers used were characterised by higher sensitivity for low level measurements. The input signal was afterwards amplified with the charge amplifier. Displacements were obtained by built-in integration function in the charge amplifier. The amplified signal is then forwarded to analogue-digital unit which was controlled by the portable PC computer. To unravel the modal parameters from the vibration data the Fast Fourier Transformation was utilised. The limited vibrational analysis was performed in-situ and the complete analysis was performed afterwards in the laboratory. The initial measurements have clearly shown that the motion was dominated by a single frequency although in the response spectra some other frequencies were detected. Therefore the main attention was paid to the detection of the dominant

eigenfrequency. Together with the erection of the structure eigenfrequencies were decreasing as the result of the increasing mass and the decreasing stiffness of the balanced cantilever structure. This resulted in the fact that more and more eigenfrequencies were detected in the measured frequency range.

## **5. The Comparison of the Calculated and Measured Eigenfrequencies**

In the measured motions of the structure (accelerations, displacements) after the finishing of each segment the eigenfrequencies corresponding to the computed eigenmodes (Fig. 3) were detected. For the eigenmode A (flexural vibrations of the pier structure), which was dominant from the finishing the base of the cantilever to the beginning of the construction of the eighth segment, the eigenfrequencies in all erection phases were identified. For the eigenmodes B and C, which were not dominant, the corresponding eigenfrequencies were identified only after the finishing of the base and the third and eighth segment. Eigenfrequencies corresponding the eigenmodes D and E were constantly identified after the finishing of the ninth and sixth segment respectively. The comparison between the measured and calculated values for the eigenmode A showed reliable agreement as the disagreement of results was about 10%. Even better accuracy has been obtained at the eigenmode that belonged to the transverse motion of the cantilever, since the difference was practically insignificant. The constant disagreement of the eigenfrequencies belonging to the transverse motion of the pier and excellent agreement of the results belonging to the transverse motion of the cantilever has confirmed the assumption that the computational model of the structure should be upgraded by implementation of an appropriate model for the soil-structure interaction.

## **6. Computational model upgrading using measured data**

Since the measurements on the structure have shown that the fixed boundary conditions in the computational model are not the realistic solution, the process of model upgrading was performed. Unlike some approaches, known from the literature, where a relatively large computational model for the implementation of the soil-foundation into the analysis process was used, the whole concentration was to 'plant' a small, compact computational model into the rigid media. Thus the first step was to choose a model that is both, simple enough moreover accurate for the soil-foundation-structure interaction. A simple bloc of elastic material was inserted between the rigid base and the FE model of construction. Figure 2 shows the computational model of the upper structure with the elastic bloc. The segments of the structure are represented with super elements and a quarter of the construction is omitted only for a better representation of the computational model. The elastic bloc was divided into 240 tetraedric finite elements. The characteristics of this elastic bloc should have covered both, the behaviour of the foundations and the soil. It was expected that such simplification would have been possible due to very good soil characteristics. To identify the characteristics of the above mentioned elastic insert, the iterative method based on comparison of the measured results and the results obtained with the FE model was used. These unknown values were identified using the model of the first phase of the construction (only the pier and the base segment with erection gantries on both sides). In a simple iteration process the mechanical properties of the elastic bloc were altered and the results, obtained with modified structure, have been compared against the measured one. The comparison the results served as an indicator for further modification of the mechanical data of the elastic bloc. After obtaining the most representative bloc (the bloc which properties gave the best results) the same elastic

bloc was afterwards inserted in the FE models of all other construction phases for the reasons of the verification.

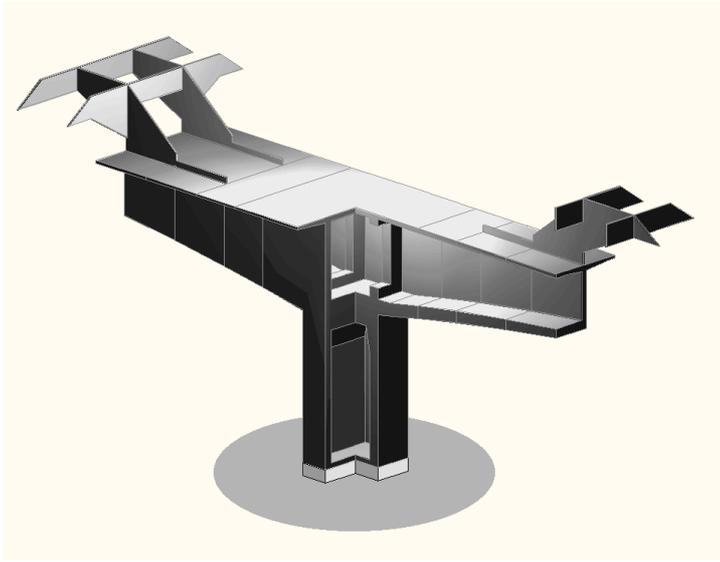


Figure 2. The schematic representation of the computational model using super elements

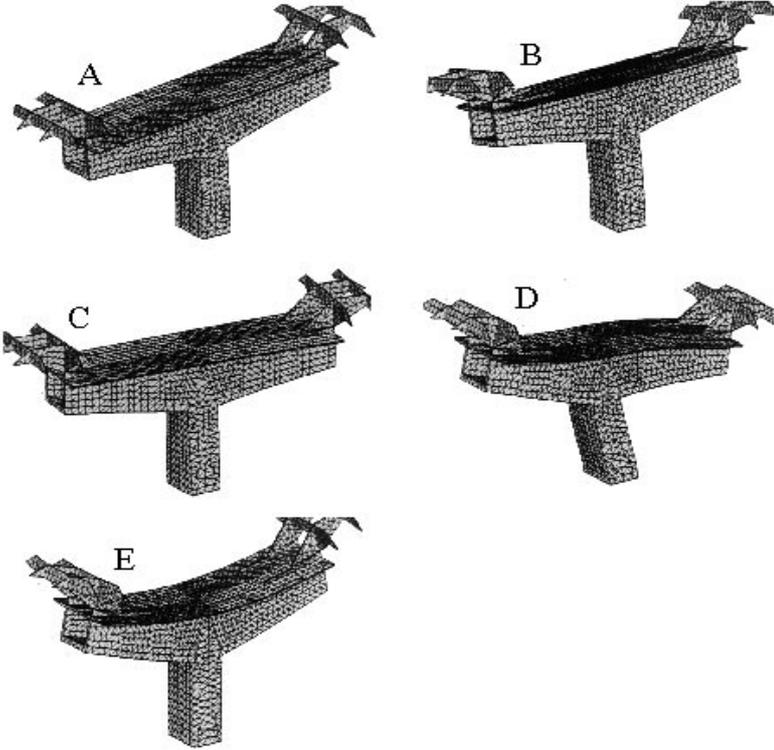


Figure 3. The five characteristic eigenmodes belonging to two most characteristic modes

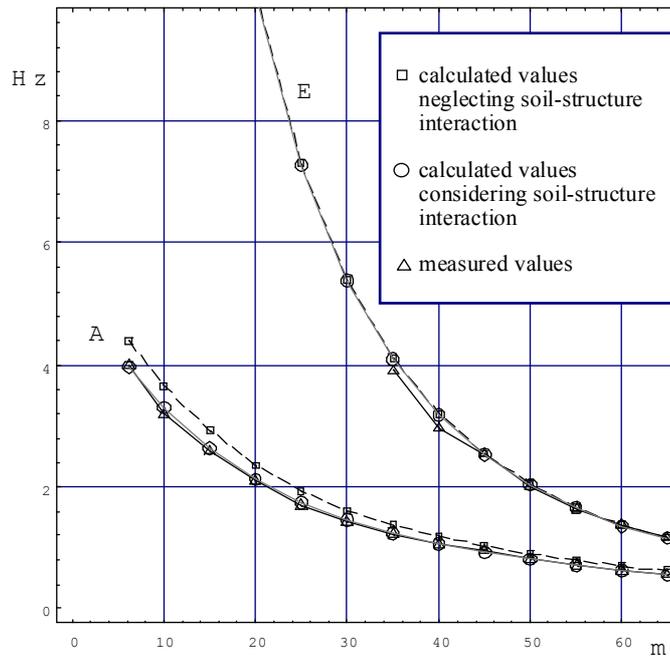


Figure 4. The comparison of eigenfrequencies of the bridge structure

The accuracy of the results in comparison with measurements was about 99% for all phases. The agreement of the eigenfrequencies corresponding to the eigenmode transverse motion of cantilevers (becoming increasingly dominant after finishing the sixth segment) remained unchanged; that proves that the reason for the disagreement of the results for the first model was actually caused by unsuitably chosen boundary conditions. Eigenfrequencies obtained using new model have contributed to a better understanding of previously measured eigenfrequencies. Since the matching of the results was very good it was possible to classify the measured eigenfrequencies to the belonging eigenmodes. It became evident that partially also the eigenfrequencies of the eigenmode D were identified, but their agreement with calculated values was worse because their traces in the measured signal were very weak. In the Fig 4. the comparison of the measured and calculated values is shown. Evidently, after the introduction of the simplified interaction model into the calculation the difference between the measured and calculated values almost vanished. The five eigenmodes, that can be identified through all phases of the free cantilever construction are schematically shown in the Fig.3. The eigenmodes A, D and E represent motion in the  $y$ - $z$  plane, eigenmode B represents the motion in the  $x$ - $z$  plane and eigenmode C represents rotation around the  $z$  axis.

## 7. Conclusions

In the design of a structure, the implementation of soil-foundation-structure interaction into the analysis process plays a very important role. Mathematical models of real structures usually involve significant assumptions especially with regard to boundary conditions. The comparison and correlation of theoretical predictions with the measured response have clearly confirmed this fact. To adequately predict the seismic behaviour of the bridge-foundation-soil system, 3-D finite element analysis is one of the best choices. Since the soil might be a material quite different from what a mathematician might choose for tractable analysis, a large variety of models from very simple to very sophisticated could be considered in the computation. The approach implemented differs from the approach used by some investigators where the soil and foundation were modelled by a huge mesh of four-node solid elements

as in the case under consideration relatively small elastic 3D bloc was used to simulate the soil and foundation. It is obvious that an upgrade of the model is much more accurate if it is performed with the dynamic rather than with the static data. This approach has thus proved that full-scale tests can be a very useful tool in engineering to update the numerical model based on the fundamental properties of the materials and components. From the engineering point of view, the obtained model is an optimal one since its complex description of the upper structure offers very detailed information about the bridge structure behaviour during the loads. On the other hand the implemented soil-foundation model is simple and yet accurate enough to avoid unnecessary computational efforts. A similar study will be performed also for the right embankment structure, since the change of eigenfrequencies was monitored during all erection phases of the structure. The corresponding computer simulations will be performed as soon as possible. Finally, with the established properties of the computational model for the whole structure, further tests on the bridge can serve as an independent instrument to control the degree of the structure degradation during its utilisation period.

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