System Identification and Finite Element Model Updating of a Multi-Span Railway Bridge with Uncertain Boundary Conditions

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Abstract

This study presents the implementation of a sensitivity-based finite element model updating process on a 48.6 m long, multi-span, reinforced concrete railway bridge located in Stange, Norway. Lack of documentation and uncertainties surrounding the boundary conditions combined with unrealistic dynamic response obtained from dynamic analysis using a finite element model based on the design drawings prompted the need for monitoring of the vibrations on the bridge followed by identification of modal properties and development of an updated finite element model which can more accurately represent the as-built structure.

For this purpose, the railway bridge was instrumented and vibration data from operational conditions was collected. Using the covariance- driven stochastic subspace identification method, the modal properties of the bridge were identified from the recorded vibrations. Comparison of the identified mode shapes with those obtained from the documentation-based initial finite element model showed significant discrepancies depicting the shortcomings of the initial model. A comprehensive sensitivity analysis and iterative finite element model updating was undertaken with a specific focus on the boundary conditions to obtain a FE model that can replicate the observed behaviour. As a result, the correlation between the observed and computed mode shapes were increased to 89% from 61% and the average error in the first four natural frequencies was reduced to 10% from 23%. Comparison of the initial and updated finite element models highlighted the significance of the boundary conditions on the dynamic behaviour of the bridge.

Keywords: Dynamic behaviour; railway bridge; modal identification; monitoring; finite element model updating; boundary conditions

1 Introduction

The assessment and verification of an existing bridge requires detailed finite element analysis of the bridge under generic or specific loading conditions. The accuracy of the analysis results, and thus the assessment of the bridge, is directly impacted by the capability of the developed finite element model to simulate the actual behaviour of the bridge. However, finite element models are generally based on design drawings and material specifications, which may not necessarily reflect the as-built conditions [1]. As a result, the results obtained from a finite element model that is based on these drawings and material properties often fail to match the actual behaviour of the bridge [2]. One of the most widely used methods to overcome this shortcoming is to conduct vibration measurements on the bridge, identify its modal parameters and calibrate the finite element model to match the identified behaviour [1, 2].

Calibrating the finite element model, while useful for every structure, is crucial for structures with high uncertainties regarding its structural properties. This article presents the calibration of a finite element model of such a structure, Stange Overpass in Norway. The article is structured as follows: First a summary of the structure and the conditions that has led to the summarized study is introduced. Then, the measurement campaign undertaken to measure the vibrations occurring on the bridge due to train traffic is summarized. Modal parameters obtained from the recorded vibrations through system identification is discussed. Finally, the calibration process of the finite element model is elaborated. Observations from the conducted study and needs for future work concludes the article.

2 Problem Statement

The bridge used in the study is a three-span, post-tensioned concrete railway overpass with a total length of 48.6 m. The overpass consists of two separate identical bridges that house single railway track. Figure 1 presents the elevation view of the Stange Overpass. The continuous bridge deck is supported by two abutments at the ends and two circular piers with relatively stiff pier caps along the bridge span. Foundations of both abutments and the bridge piers are anchored at the bedrock, which extends to the ground level. The bridge deck is a U-shaped monolithic construction and supports a 0.6m ballast layer along with the continuous, centric steel railway track and the sleepers.

The abutment of the bridge has an unusual detail that separates Stange Overpass from conventional bridges. The detail of the abutment and its connection to the deck is presented in Figure 2. According to the design drawings, the only connection between the deck and the abutment is through the elastomeric bearing that sits 4.4m away from the outer edge of the bridge deck. According to the design drawings, there is no physical connection between the bridge deck and the abutment along the 4.4m long portion of the bridge at the outermost section of the bridge leading to 4.4m cantilevers at both ends of the bridge. The design drawings further indicate that, the abutments at both sides, which have a U-profile, is not filled and remains empty. However, a visual inspection of the bridge by the authors revealed that the abutments have been filled with backfill material topped with a concrete slab, which is not present in the design drawings and documentation. Therefore, the boundary conditions of the bridge present a two-folds challenge. First, the boundary conditions are rather unusual for a bridge construction. Arguably more importantly, the boundary conditions of the as-built bridge deviate significantly from those that are specified in the design drawings and documents.

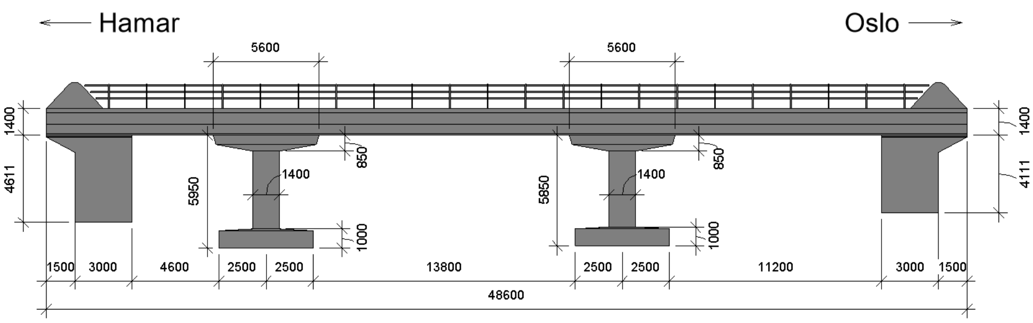


Figure 1. Elevation view of the Stange Overpass

Due to this discrepancy between the drawings and as-built structure, a finite element model that is based solely on design drawings is highly unlikely to provide a satisfactory of the estimate of the behaviour of the bridge leading to the study summarized in this article.

3 Measurement of Vibrations

3.1 Measurement Campaign

A 48-hour measurement campaign was conducted in December 2020 with the overarching goal of identifying the modal parameters of the Stange Overpass from the recorded vibrations. Another objective of the measurement campaign was to evaluate the level of accelerations that are induced by train traffic on the bridge. A total of five accelerometers were used in the measurements. In order to maximize the information about the mode shapes, the instruments were deployed in two different layouts. The placement of the three accelerometers were not changed between the two layouts so that they can serve as the anchor points in establishing the mode shapes. Moving two of the accelerometers allowed us to obtain the modal values at seven points using five accelerometers. Figure 3 shows the placement of the accelerometers for the two layouts. The orange circles in the figure represent the accelerometers and the sensor number is indicated inside the circle. For each layout, the measurements were conducted continuously for 24 hours with a sampling rate of 250Hz.

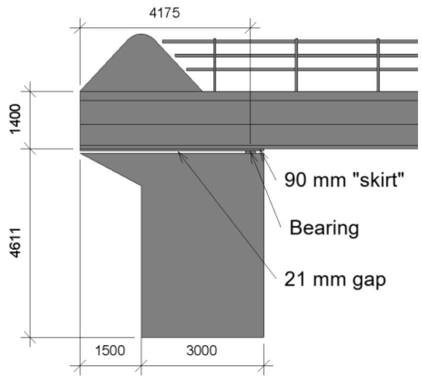


Figure 2. Detail of the abutment-deck connection

3.2 Observed Accelerations

European standards [3] limits the maximum accelerations that can be induced by train traffic on ballasted railway bridges to 3.5 m/s^2 to ensure the stability of the ballast. First, the recorded accelerations were evaluated to check if Stange Overpass conforms to this requirement. Figure 4 presents the distribution of the maximum accelerations observed at each train crossing at each sensor. The red, horizontal lines depict the median maximum acceleration at each sensor, while the edges of the boxes and the whiskers represent 75% and 99% confidence intervals, respectively. The red points indicate the statistical outliers. There are two main observations that can be deducted from the figure. First, the accelerations that are observed on the bridge is within the limits put forth by the Eurocode. Only in one case out of the 98 train crossings recorded, the limit of 3.5 m/s^2 is slightly exceeded in one of the sensors. Secondly, the accelerations recorded at the ends of the bridge is systematically higher than their counterparts at the middle of the bridge. This observation suggests that, despite the presence of the concrete slab under the bridge deck at the ends, the behaviour of the bridge is significantly impacted by the cantilevering action at the bridge ends.

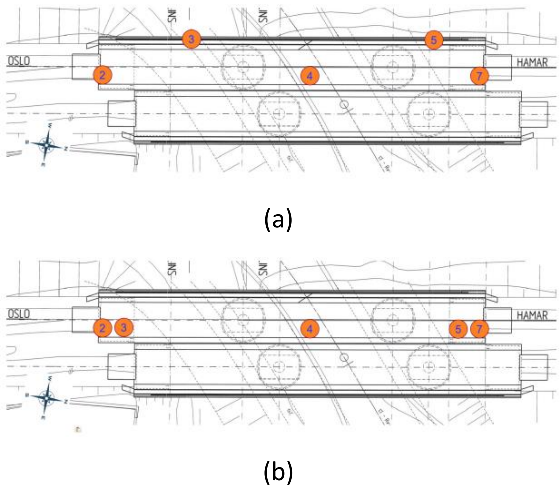


Figure 3. The placement of the accelerometers for (a) layout 1 (b) layout 2

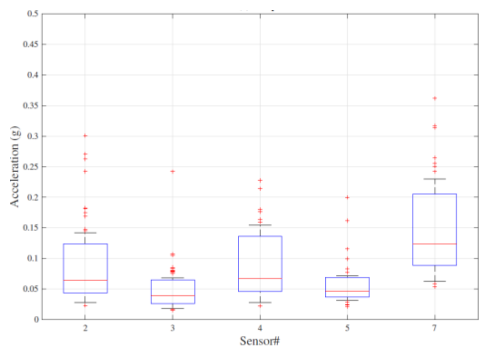


Figure 4. Box and whiskers plot of the maximum accelerations recorded at each sensor

3.3 Identified Modal Parameters

As the next step, the modal parameters of the bridge were identified from the recorded vibrations. For this, accelerations from the free decay part of the accelerograms were used. The free decay data is preferred over the ambient vibrations because they contain much higher energy compared to the latter and provides better information on the mode shapes excited by the train crossings. Figure 5 presents a sample acceleration time history that shows different phases of vibrations on a railway bridge induced by a train crossing. In this figure, the forced vibration part is depicted as phase number one, while the ambient vibrations (phase 3) before and after the crossing of the train are highlighted by green boxes. The free decay phase, i.e. phase number 2, is highlighted by a yellow box and consists of the period between two points in time: The first of these points is the moment that the train leaves the bridge while the second is the moment that the vibrations induced by the train crossing completely dies down. The free decay phase from each of the 98 recorded train crossings was used for modal identification. The duration of the free-decay phase of each record that will be used in the system identification was decided based on the methodology developed by Ülker-Kaustell and Karoumi [4]

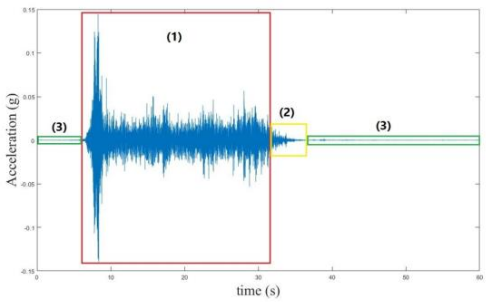


Figure 5. Acceleration time history that shows the different stages of loading induced on a railway bridge by a train crossing.

Covariance-driven stochastic subspace identification (SSI-COV) method [5] was used to identify the vibration frequencies and mode shapes from the vibration records.Application of the SSI-COV algorithm on the vibration data from the 98 train crossings revealed four distinct, recurring dominant modes. Presented in Figure 6 are the identified mode shapes and modal parameters. The depicted shapes and parameters are the mean values of the mode shapes and modal properties from the 98 train-crossings. Here, it should be noted that, the first 50 train-crossings were recorded from the first sensor layout while the remaining 48 were recorded using the second sensor layout. In order to combine the mode shapes identified using two separate sensor layouts, we first took computed the mean value of the mode shapes identified from the first and second layouts, separately. These two mode shapes were then combined by using the stationary sensors as the anchoring points. As a result, the mode shapes presented in Figure 6 are based on seven sensor locations that are depicted in Figure 3.

The linear independence of the identified mode shapes was then evaluated using Modal Assurance Criteria (MAC). The MAC values for the identified first four mode shapes plotted in Figure 7 show that there is no significant cross-correlation between the identified modes ensuring that the identified mode shapes are linearly independent.

4 Finite Element Model

4.1 Initial Finite Element Model

The initial finite element (FE) model was created in the CSI Bridge Environment based on the design drawings and specified material properties. The deck and the piers of the bridge was modelled using elastic beam – column elements. In order to ensure that the mass of the bridge in the vertical direction is taken into account correctly, the bridge deck was meshed with a specified nodal distance of 1m. The foundations at the bottom of the piers were modelled as fixed because the foundation are anchored to the bedrock. The modulus of elasticity of concrete was specified as 38 GPa. The stiffness of the springs that emulate the elastomeric bearings are computed based on the product specification of the bearings. The ends of the deck were modelled as cantilevering from the elastomeric bearings as indicated in the design drawings. Modal analysis was carried out on the initial finite element model to compute the modal properties. The first two mode shapes computed using the initial finite element model are plotted in Figure 8 together with the identified mode shapes. Figure 8 clearly shows that initial finite element fails to capture dynamic behaviour of the bridge as evidenced by the significant discrepancy between the computed and identified mode shapes. Owing to the cantilevering parts at both ends of the bridge deck, the mode shapes from the initial FE model is dominated by the cantilevering behaviour of these parts. On the other hand, the identified mode shapes have much lower modal values at the ends of the bridge indicating the significant support provided by the concrete slab at the top of the abutments and the underlying soil layer. In order to reflect this effect on the finite element model, linear springs that span from the elastomeric bearings to the ends of the bridge was added to the FE model.

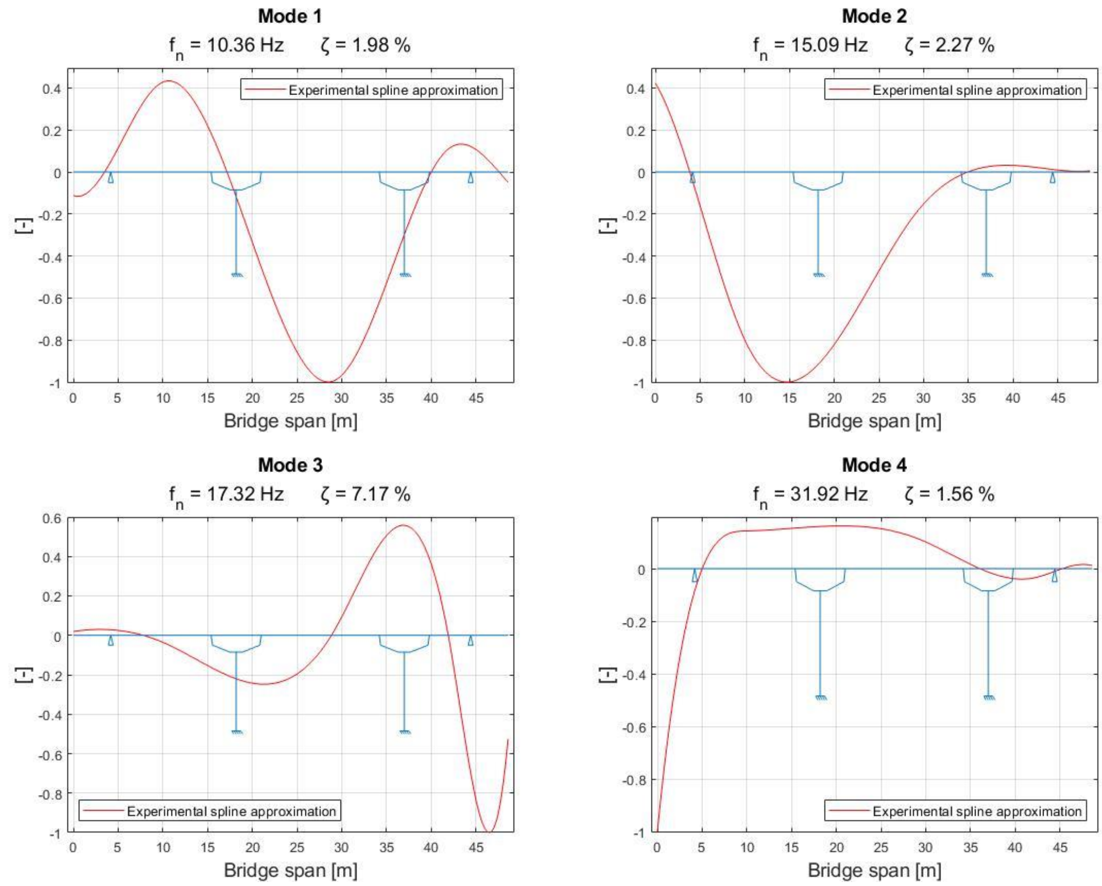


Figure 6. Mode shapes and modal parameters identified from the recorded vibrations

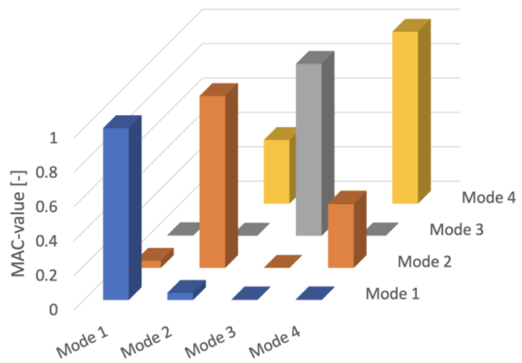


Figure 7. MAC correlation of the extracted mode shapes

4.2 Updating of the FE Model

Inspection of the difference between the identified mode shapes and those computed from the initial FE model (Figure 8) show that the most critical parameter in the updating process is the stiffness of the linear springs that simulate the behaviour of the concrete slab and the underlying soil layer at the abutments. A sensitivity study to establish the values of these parameters were conducted first. The properties of the soil supporting to the concrete slab at the top of the abutment were unknown. In order to cover a wide range of possible soil types and conditions, the soil stiffness value was varied from 5000 kN/m^2/m to 300000 kN/m^2/m. This range of soil stiffness represents a wide range of soil conditions from organic material to crushed stone [6]. The results of this sensitivity study were later used as the initial estimates of the FE model updating process.

Initially, a total of nine parameters were included in the finite element model updating process. These parameters are Young´s modulus and mass density of concrete of the bridge deck, Young´s modulus of concrete of the bridge piers, mass density of the bridge piers, mass density of the ballast, stiffness for the bearings and the stiffness of the spring representing the soil resistance at the abutments.

Sensitivity analysis conducted to quantify the impact of each parameter on the dynamic properties of the bridge showed that mass density of the bridge piers and the mass density of the ballast have minimal impact on the vibration characteristics of the bridge. Therefore, these two parameters were set to their median values and were not included in the FE updating procedure leaving seven parameters.

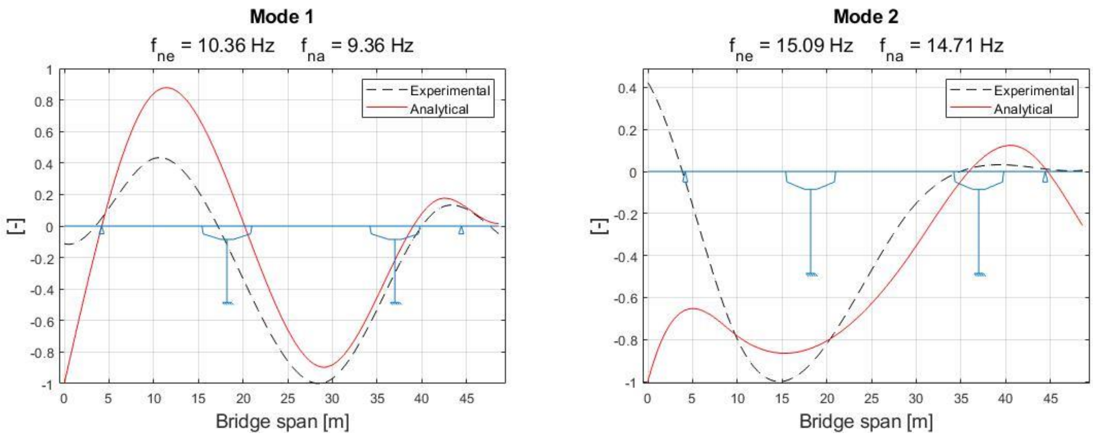


Figure 8. Comparison of the two first identified mode shapes with those computed using the initial FE model

Finally, a series of FE analysis for a wide range of the seven selected parameters were conducted. The optimum set of values for these parameters were then determined through statistical comparison of the mode shapes and vibration frequencies computed from the FE analyses to those obtained from the system identification process. Figure 9 presents the comparison of the first two mode shapes obtained from the calibrated model and those identified from the recorded vibrations. The MAC values presented in Figure 10 and the vibration frequencies shown in Table 1 depict the impact of the FE model updating procedure in estimating the identified mode shapes and vibration frequencies. While the initial FE model based on the design drawings fail to estimate both the mode shapes and the vibration frequencies, the updated model provides much improved results. Table 2 shows the values of the modelling parameters used in the initial and updated FE models. These values clearly show that the behaviour of the bridge is dominated by the boundary conditions of the bridge. The material properties, on the other hand, have relatively low impact on the dynamic properties of the bridge and remain unchanged from the initial model.

5 Concluding Remarks

This article presents the results of the first step of a research project that aims to calibrate the finite element model of a railway bridge based on long term monitoring. It summarizes the initial measurement campaign where the vibrations on the Stange Overpass was measured for 48 hours. The measured vibrations were then used to identify the dynamic properties of the bridge. The identified mode shapes were then compared to their counterparts computed using a FE model based on the design drawings. This comparison showed that the FE model based on the design drawings fail to capture the dynamic behaviour of the bridge.

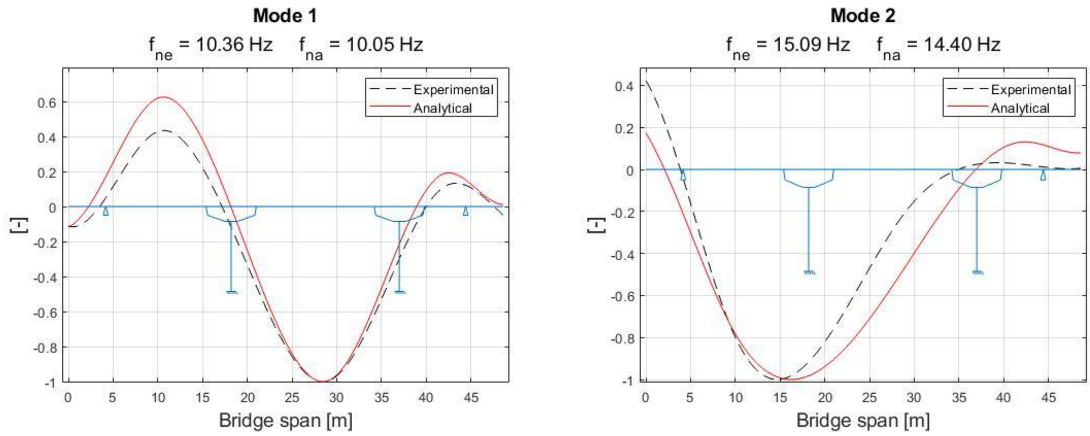
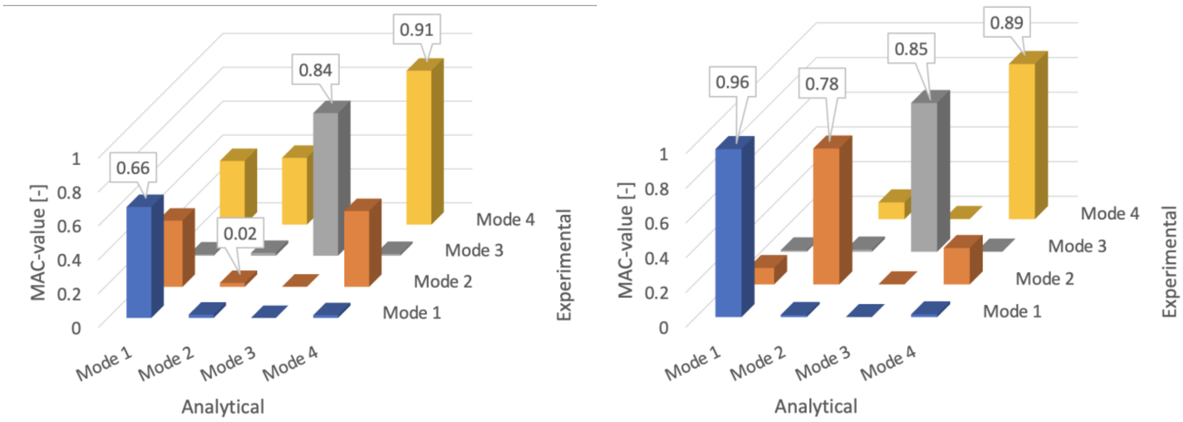


Figure 9. Comparison of the identified first two mode shapes with those computed using the calibrated FE model



(a) (b)

Figure 10. Comparison of MAC values between the identified and computed mode shapes for (a) initial FE mode (b) final numerical model

Table 1. Comparison of vibration frequencies identified from recorded vibrations to those obtained from numerical analysis

Mode #	Identified (Hz)	Initial FE Model (Hz)	Error (%)	Calibrated FE Model (Hz)	Error (%)
1	10.36	9.36	9.7	9.98	3.7
2	15.09	14.71	2.5	14.23	5.7
3	17.32	13.60	21.5	19.00	9.7
4	31.92	12.41	61.1	25.52	20.1

Table 2. Modeling parameters used in the initial and calibrated FE models

Parameter	Initial FE Model	Calibrated FE Model
Young's modulus of concrete bridge deck and girders [GPa]	36.0	36.0
Young's modulus of concrete columns [GPa]	36.0	36.0
Mass density of concrete bridge deck and girders [kg/m3]	2 548	2 548
Mass density of concrete columns [kg/m3]	2 548	2 548
Mass density of ballast [kg/m3]	1 800	1 800
Stiffness of bearing spring Hamar [kN/m]	1 629 630	1 475 000
Stiffness of bearing spring Oslo [kN/m]	1 629 630	1 475 000
Stiffness of soil-spring Hamar [kN/m/m]	-	126 614
Stiffness of soil-spring Oslo [kN/m/m]	-	383 680

The FE element model was then calibrated to minimize the discrepancy between the computed and identified modal parameters. The comparison of the initial and calibrated FE models highlights that the parameter governing the behaviour of the bridge is the boundary conditions that are highly uncertain due to the construction of the bridge.

The presented study lays out the foundations of the next phase of the project. In this phase, the vibrations on the bridge will be monitored for at least a year. The long-term monitoring will be used to evaluate the impact of the environmental conditions on the dynamic behaviour of the bridge, and particularly, on the boundary conditions. The FE model will then be calibrated further using the long-term monitoring data to reflect these effects.

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7 References

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